Summary paper of:


J. Christer Andersson  Svensk Kärnbränslehantering AB
1 Introduction

The construction of nuclear waste repositories and prediction of their long-term behaviour require a more detailed understanding of certain rock mechanics problems than has previously been needed by the industry. The nuclear waste repository will be located at great depth. The stresses at that depth increase the risk of stress-induced yielding in parts of the repository. Due to the stringent safety requirements, this may cause a problem in designing the facility. The actual initiation point for yielding should therefore be determined as precisely as is practically possible.

The first well documented full-scale experimental approach to studying the yielding of a rock mass was performed at the Underground Research Laboratory (URL), Pinawa, Manitoba, Canada where the Mine-by Experiment (MBE) was conducted between 1989 and 1995. In the MBE excavation-induced stress initiated spalling, and v-shaped notches formed at the locations of the highest stress concentration. Martin (1997) summarized the observation in four stages during the process of notch development.

To verify that the findings from the URL, where the rock is in principle fracture-free, could be applied to the rock in the Fennoscandian shield, which includes fractures, SKB conducted the Äspö Pillar Stability Experiment (APSE) in order to: 1) demonstrate our current capability to predict yielding in a rock mass containing fractures, 2) demonstrate the effect of a confining pressure, and 3) compare the 2D and 3D mechanical and thermal predicting capabilities of existing numerical codes.

Mainly based on the observations and interpretations of the Mine-by Experiment, the following hypotheses were derived. The hypotheses served as a basis for the design work.

1. Äspö diorite is expected to yield in a manner similar to the Lac du Bonnet granite at the URL, despite the fact that the diorite is fractured. The yield strength should therefore be approximately 0.6 \( \sigma_c \), and the depth of the breakout should be between 1.1a and 1.4 a (9 to 35 cm), where “a” is the radius of the large hole).

2. Elastically calculated temperature-induced stresses can be superimposed on the excavation-induced stresses at locations where the rock has not yielded. A good approximation of the actual stress state at those locations can then be derived.

3. Small confining pressures significantly influence the yield strength of rock. A clear difference in the behaviour of the rock mass in the two holes was therefore expected to be observed.
2 Design of experiment

Rock mass yielding has only been associated with a few locations in the deeper parts of the Äspö HRL. The findings from the URL were, as previously described, used to guide the design of the Äspö Pillar Stability Experiment. It should be noted, however, that while the mean uniaxial laboratory compressive strength (UCS) of Äspö diorite is approximately equal to the UCS of Lac du Bonnet granite, the magnitude of the maximum principal stress at the 450 m level of the Äspö HRL is only 50% (approximately 30 MPa) of the maximum principal stress in the AECL’s URL (60 MPa). Hence, a major challenge for the experiment was to develop a design that would increase the excavation-induced stresses in a controlled manner, similar to that used in a laboratory environment. To achieve this the experiment contained the following major steps:

1) Determine a suitable site for the experiment on the 450 m level of the Äspö Hard Rock laboratory (Figure 2-1)

2) Determine the largest practical tunnel geometry for elevating the stresses in the floor of the tunnel to an optimum magnitude

3) Conduct scoping calculations and predict the pillar response

4) Excavate the tunnel and carry out site characterization.

5) Determine the optimum width of the deposition-hole pillar based on scoping calculations and site characterization

6) Design and test the confining system that was to be installed after the drilling of the first hole

7) Select the location for the pillar, and install the instrumentation

8) Excavate the first 1.75-m-diameter hole to a depth of 6.5 m and install the confining system

9) Excavate the second 1.75-m-diameter hole to form the pillar

10) Install additional instrumentation in the open hole and install the heaters

11) Heat the pillar gradually so that the onset of failure could be easily detected and monitored

12) At the end of the heating phase, gradually release the confinement pressure

13) Conduct post-experimental characterization of the pillar to document the extent of damage

The tunnel trends perpendicular to the direction of the major principal stress with the pillar located approximately 15 m behind the tunnel face. The top portion of tunnel has straight walls and an arched roof, while the invert had a rounded tunnel floor with a ~2-m radius. This provided uniform concentration of excavation-induced stresses in the floor. The total height of the tunnel is approximately 7.5 m and the width 5 m. Because of the different geometry of the top and bottom of the tunnel, the tunnel was excavated in two stages, a top heading and a bench (Figure 2-2). The major advantage of this strategy was to minimize the damaged zone in the floor. It was important to ensure that the rock mass response near tunnel and pillar boundaries was essentially elastic and not too damaged by the blasting.
Figure 2-1. Location of APSE in relation to the deeper parts of Äspö HRL. The arrow indicates the direction of the major principal stress.

Figure 2-2. Section of the tunnel. The tunnel was blasted as a pilot drift and thereafter a separate bench to reduce the EDZ as much as possible.

Three types of monitoring systems were used to monitor the experiment: temperature, displacement and acoustic emission, Eng & Andersson (2004)

Monitoring of the temperature during the heating phase of the experiment was essential for calibration of the thermal model, the instrument set up is illustrated in Figure 2-3.

The experiment was designed to allow displacements associated with the pillar yielding process to be monitored at various stages of pillar loading. Two types of LVDT (Linear Variable Differential Transformer) type instruments with different measurement range were chosen to monitor these displacements. The LVDT’s were mounted on steel pipes (Figure 2-4) at the depths 2.5, 3.0, 3.5 and 4.1 m.

Acoustic Emission (AE) monitoring was used for tracking brittle damage in the rock (Haycox et al. 2005). The AE monitoring system consisted of twenty-four ultrasonic transducers which responded to the frequency range 35-350 kHz (Figure 2-5).
In the KBS-3 concept the bentonite in the emplacement borehole will generate a swelling pressure of approximately 5-7 MPa. One of the objectives of the experiment was therefore to establish the effect of confining stress on brittle fracturing. This objective required the design and construction of a confining system that could be placed in one of the 1.75-m-diameter boreholes. The confining system was constructed of a rubber bladder approximately 1.75 m in diameter and 6 m long installed in a system consisting of steel plates connected by strong textile straps (Figure 2-6 and Figure 2-7).

The pillar was heated with 4 electrical heaters with a heated length of 6.5 m, which were installed in four vertical boreholes.

![Figure 2-3](Image)

**Figure 2-3.** Plan view of the instrumentation of the experimental volume and a vertical section perpendicular to the tunnel axis showing the symmetrical depth distribution of the thermocouples and the borehole ID codes.

![Figure 2-4](Image)

**Figure 2-4.** LVDTs mounted on a steel pipe in the open hole. The upper LVDT is of the short-range type and the lower of the wide-range type.
Figure 2-5. A schematic of the AE acquisition system and sensor configuration. There are two ultrasonic transducers and four receivers in each instrumented borehole.

Figure 2-6. The specially constructed rubber bladder and the confining equipment. The vertical textile straps connected to the horizontal steel plates kept the bladder in place as it was expanded by a water pressure.

Figure 2-7. Photographs of the textile straps and the top steel lid.
3 Geotechnical setting

The major rock type in the experiment volume consists of slightly fractured Åspö diorite. Parts of the volume are oxidized and a mylonitic shear zone cuts through the upper part of the pillar. Some pegmatite and fine grained granite veins are also included. Examples of the mapping are presented in Figure 3-1 and Figure 3-2.

The mechanical properties of the rock in the experiment volume is well documented (Staub et al. 2004). Eighteen samples were tested using the ISRM-recommended testing procedure (Brown, 1981). To determine the onset of cracking for Åspö diorite, axial and lateral strains were recorded for each test. The number of acoustic emission events was also recorded for the uniaxial compressive tests. In addition to the mechanical properties, a number of tests were carried out to establish the thermal characteristics of Åspö diorite. The results from all the successful laboratory tests on intact samples from the experiment area are given in Table 3-1. In Table 3-2 laboratory determined fracture properties are presented.

![Figure 3-1. Compilation of the geological mapping of the five pillar blocks.](image)
Figure 3-2. Geological mapping of the tunnel floor at the pillar location. The large-diameter holes and the reference points A to F used during the mapping of the hole walls are indicated in the figure.

Table 3-1. Intact rock mechanics parameters derived from laboratory tests on core samples. Modified from Staub et al. (2004).

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Mean value</th>
<th>Range</th>
<th>Unit</th>
</tr>
</thead>
<tbody>
<tr>
<td>Uniaxial compressive strength</td>
<td>211</td>
<td>187 – 244</td>
<td>MPa</td>
</tr>
<tr>
<td>Young’s modulus, intact rock</td>
<td>76</td>
<td>69 – 79</td>
<td>GPa</td>
</tr>
<tr>
<td>Young’s modulus, rock mass</td>
<td>55</td>
<td>-</td>
<td>GPa</td>
</tr>
<tr>
<td>Poisson’s ratio, intact rock</td>
<td>0.25</td>
<td>0.21 – 0.28</td>
<td>-</td>
</tr>
<tr>
<td>Friction angle, intact rock</td>
<td>49*</td>
<td>-</td>
<td>Degrees</td>
</tr>
<tr>
<td>Cohesion, intact rock</td>
<td>31*</td>
<td>-</td>
<td>MPa</td>
</tr>
<tr>
<td>Tensile strength</td>
<td>14.9</td>
<td>12.9 – 15.9</td>
<td>MPa</td>
</tr>
<tr>
<td>Thermal conductivity</td>
<td>2.60</td>
<td>2.39 – 2.80</td>
<td>W/m, K</td>
</tr>
<tr>
<td>Volume heat capacity</td>
<td>2.10</td>
<td>2.05 – 2.29</td>
<td>MJ/m³, K</td>
</tr>
<tr>
<td>Linear expansion</td>
<td>7.0</td>
<td>6.2 – 8.3</td>
<td>(1/K)×E-06</td>
</tr>
<tr>
<td>Density</td>
<td>2.75</td>
<td>2.74 – 2.76</td>
<td>g/cm³</td>
</tr>
<tr>
<td>Initial temperature of the rock mass</td>
<td>14.5</td>
<td>-</td>
<td>°C</td>
</tr>
<tr>
<td>Crack initiation stress, AE</td>
<td>121</td>
<td>80 - 160</td>
<td>MPa</td>
</tr>
<tr>
<td>Crack initiation stress, strain gauge</td>
<td>95</td>
<td>83 - 112</td>
<td>MPa</td>
</tr>
</tbody>
</table>

*Average data from Åspö HRL. Not tested on the APSE cores.
Table 3-2. Mechanical parameters of laboratory-induced fractures derived from core samples.

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Mean value</th>
<th>Unit</th>
<th>Standard variation</th>
</tr>
</thead>
<tbody>
<tr>
<td>Mode I toughness, $K_{IC}$</td>
<td>3.8</td>
<td>MPa/m$^{1/2}$</td>
<td>0.1 MPa/m$^{1/2}$</td>
</tr>
<tr>
<td>Mode II toughness, $K_{IIC}$</td>
<td>4.4 to 13.5</td>
<td>MPa/m$^{1/2}$</td>
<td></td>
</tr>
<tr>
<td>Initial normal fracture stiffness, $K_{NII}$</td>
<td>175</td>
<td>GPa/m</td>
<td>68</td>
</tr>
<tr>
<td>Normal fracture stiffness, $K_{NH}$</td>
<td>26976</td>
<td>GPa/m</td>
<td>22757</td>
</tr>
<tr>
<td>Shear stiffness</td>
<td>15.7 / 35.5</td>
<td>GPa/m</td>
<td></td>
</tr>
<tr>
<td>Residual angle</td>
<td>31 / 30</td>
<td>Degrees</td>
<td></td>
</tr>
</tbody>
</table>

The in situ stress magnitudes and orientations were key parameters for the design of the experiment. In the vicinity of the APSE Christiansson & Janson (2002) reported stress measurements with three different methods in two orthogonal boreholes and recommended a stress tensor to be used at the 450 m level.

To verify the stress tensor, back-analysis of convergence measurements was performed during the excavation of the experiment drift, (Andersson & Martin 2003; Staub et al. 2004) using the boundary element code Examine3D (Rocscience). The resulting best fit stress tensor was obtained with a rock mass Young’s modulus of 55 GPa and a Poisson’s ratio of 0.26 (Table 3-3). The major difference between the stress tensor in and the tensor in Table 3-3 is related to the plunge and the trend. Christiansson & Janson. An example of measured convergence and modelled results are presented in Figure 3-3.

Table 3-3. Back-calculated and best-estimate stress tensor for the APSE site.

<table>
<thead>
<tr>
<th>Magnitude [MPa]</th>
<th>$\sigma_1$</th>
<th>$\sigma_2$</th>
<th>$\sigma_3$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Trend (Åspö 96)</td>
<td>310</td>
<td>090</td>
<td>220</td>
</tr>
<tr>
<td>Plunge (degrees from horizontal)</td>
<td>0</td>
<td>90</td>
<td>0</td>
</tr>
</tbody>
</table>

Figure 3-3. A comparison of the measured convergence with the computed convergence from Examine3D. When plotting the figure it is adjusted for that 50% of the total displacements took place before the pins were installed.
4 Monitoring and observations

An overview of the monitored results and the visual observations made during the experiment are presented in this section. The monitored results origins from the thermocouples, LVDT’s and the acoustic system, visual observations were made approximately once a week by manually accessing the hole.

The graphs in Figure 4-1 to Figure 4-3 present the temperatures at both the pillar walls and between the heaters on the left side of the experiment.

**Figure 4-1.** Measured temperatures in DQ0066G01 (the confined hole) at position “C”. Water leaking into the hole reduces the temperature compared to DQ0063G01. The temperature anomalies from day 61 depend on the release of the confinement pressure.

**Figure 4-2.** Measured temperatures in DQ0063G01 (the open hole) at position “C”.

The displacement graph in Figure 4-4 originates from the 3.0 m instrument level. All monitored displacements occur, as indicated in the figure, stepwise.

The rock at the centre instrument positions at depths of 3 and 3.5 m exhibits a radial expansion before the dilatation is initiated. The reason for this is probably a 3-dimensional stress re-distribution caused by the propagating notch’s effect on geometry. Figure 4-5 illustrates the radial expansion at the 3.0m instrument level.

Efforts were made to back-calculate the radial expansion measured using Phase2D and Examine3D. The 2D and 3D modelling of the pillar centre indicates deformations in the same direction as the radial expansion. However, the deformations are only 10% of those measured in situ. The models hence indicated but did not predicted the radial expansion very well.

The acoustic records from the heating phase of the pillar is presented in Figure 4-6. AE data after day 60 which include the release of the confining pressure and the cooling of the experimental volume are not included. Almost all events are located at the open hole’s pillar wall. The very few events in the central part of the pillar indicates that it can be considered to respond elastically during the heating phase. The records in Figure 4-6 are compiled in Figure 4-7.
Figure 4-4. Deformations of the short-range sensors at the 3.0 m instrument level

Figure 4-5. Contraction of rock before dilatation is initiated at the 3.0 m level.
Figure 4-6. Location of acoustic emission events during the heating phase of the experiment. Grey shaded areas are steel plates used to cover the irregular zones created during the drilling of the first hole and provide good contact between the water-filled bladder that provided the confining stress and the rock wall. Data modified from Haycox et al. (2005).
Figure 4-7. All source-located AE events during excavation and heating of the pillar. Acoustics from the release of the confining pressure are not included.
5 Yielding observations

Documentation of the propagation of fracturing and yielding were made on 15 different occasions during the experiment. When the heaters had been turned off and the rock mass had cooled down, the slabs created by the spalling were carefully removed (approximately 0.1 m³). Laser scanning of the pillar wall in the open hole was performed before and after the removal of the slabs.

As the v-shaped notch was formed and migrated down the borehole, the behaviour of the rock can be described in four different stages (below) which are summarized in Figure 5-1.

1. As the notch tip approached an instrumented level, the radial expansion was measured.
2. Below the notch tip, small rock slabs (chips) started to form tangentially to the borehole wall, the first indication of dilation. These chips were roughly a quarter of a fingernail in size and very thin. After a while, larger thin chips of coin size were observed.
3. Discrete fractures started to form. The fractures became the boundaries of the slabs and ranged in length from a few centimetres to approximately 10 cm.
4. More fractures formed, creating larger slabs beneath the ones already formed. As the fractures grew deeper into the pillar, their angle increased from being tangential to the hole wall for the first superficial chips.

All observations indicate that the initiation of brittle failure is purely tensile. Only in the deepest part of the notch was there evidence that shearing had also occurred.

To establish the full three dimensional geometry of the overstressed region, i.e., the failed zone, the pillar wall was scanned using laser technology. The data set was used to assess the width and the depth of the failed area as well as the location of the maximum depth. A visualization of the survey is presented in Figure 5-2 where the width and depth of the spalled zone is indicated. The width of the spalled volume presented as the total breakout angle is plotted together with the notch depth in Figure 5-3.
Figure 5-1. Illustration of the yielding process in an overstressed granitic rock.

Stage II. When the yielding stress is reached, thin finger nail sized chips are formed tangentially to the hole wall. The thin rough edges of the chips and inspection of the chip surfaces, suggests these thin slabs were formed by tensile stresses.

Stage III. As the stress increases larger chips are formed. Also these are tangential to the hole wall and have thin edges but they are thicker in the centre. The chips appears to have been formed by tensile stresses.

Stage IV. The chips increase in thickness and aerial size as the V-shaped notch gets deeper. At some point, crushing and/or shearing at the notch tip occurs and is concentrated in a relatively small area (several cm²), suggesting that at the notch tip the small chip forming process observed in stages II to III is suppressed. The process at the notch tip appears to extend into the pillar at this stage the large chips appear to form on the flanks of the notch, which also appears to be formed by tensile stresses. The wall surfaces of the notch are quite smooth.

These fractures are only present in parts of the notch. They appear in the upper part of the photograph but not in the lower.
Figure 5-2. Result of the laser scan. The left-hand figure shows the approximate geometry of the notch. The right-hand figure shows the depth of the notch and the dashed line indicates the location of the borehole wall before yielding was initiated.

Figure 5-3. Left: notch depth and breakout angle for hole depths in 50 mm increments. Right: Breakout angle is defined as the angle for a sector with the centre point at the hole’s centre point which intersects the scanned contour as it coincides with the original contour. Notch depth is the longest distance perpendicular to the original contour of the spalled contour, i.e. depth of yielding.
Yield strength

To determine the yield strength of the rock the total tangential stress has to be determined. The total stress consists of the excavation and the thermal induced stresses superimposed on each other. The thermal induced stresses were back-calculated using the finite element program Code_Bright (CIMNE 2000). The modelling is described in detail in Andersson et al. (2006) and Fälth et al. (2005). The temperature monitoring results from the centre of the pillar wall were used as thermal boundary conditions. The thermal properties of the rock, and to a lesser extent the output from the heaters, were adjusted to keep the temperatures between the heaters as close to the monitored temperatures as possible. The difference between the monitored temperatures and the back calculated was only a few degrees.

Examples of the calculated thermally induced tangential stress at four different depths are given in Figure 6-1 together with the accumulated number of acoustic events recorded for each day. Figure 6-1 clearly shows that the daily number of acoustic emission events increases when the stress increases.

![Figure 6-1. Thermally induced tangential stress close the four instrument levels and the associated daily acoustic events. The induced thermal stress is very similar between 2.46 m and 4.10 m. Note that the number of daily acoustic events increases when the stress increases. The reduction of the confining pressure began at day 61 and that part of the data set is not included.

The excavation of the boreholes forming the pillar caused the stresses to redistribute. Because of this stress redistribution, stresses near the boundary of the excavation can change significantly depending on its geometry in relation to the orientation and magnitude of the far-field stress tensor. Examine3D was used to track the stress path for point A located 0.1 m to the left of the centre of the hole at a depth of 2 m and 3 mm into the pillar wall during the excavation phases. The stress path, which is plotted in Figure 6-3 in the $\sigma_1$, $\sigma_2$ space, was determined by excavating the first hole in three stages and then continuing with the second hole in five stages.

With the excavation of the second hole Sigma 3 decreases, and Sigma 2 and Sigma 1 increase. Hence, the deviator stresses are increasing while the mean stress is decreasing. From Figure 6-3 it is clear that there is significant unloading (sigma 3) and loading (Sigma 1) as the second hole is excavated.
Figure 6-2. Excavation induced tangential stress along the most stressed part of the pillar. The dashed line indicates where the stress is modelled slightly to the left of the pillar centre in the second hole, DQ0063G01.

Figure 6-3. Stress path for point A located at a depth of 2 m close to the boundary of the second hole. Circles indicate the stress at the modelled excavation stages. Sigma 2 (S2 in figure) stresses are also provided for each excavation stage.

The yielding time for the different locations of the pillar wall was determined using the LVDT displacement graphs and visual observations. The onset of dilation indicated by the LVDTs is defined as the time the transducer is compressed more than its initial setting.

For the visual observations yielding was defined to have occurred when small (fingernail-sized) rock chips formed on the surface of the hole. The exact time of the occurrence of yielding could obviously not be determined from the visual observations. However, the rate of temperature increase was quite low and the error is assessed to be less than 2% of the maximum tangential stress (~2 MPa).
The locations of yielding determined by the LVDTs and the visual inspections are numbered sequentially in Figure 6-4 on top of the geological mapping. The numbers were assigned in the order spalling took place. For example, number 10 in Figure 6-4 shows the location of the tenth observed spalling area.

During the heating phase the yielding occurred as the tangential stresses increased. Often the yielding would occur by a start/stop manner. For example, nothing would happen for a while until and then suddenly spalling would occur and last for a few minutes. The intermittent spalling resembled the build-up and release of stress. Very few acoustic emission events were recorded in between the spalling occasions. The observations made during the excavations and the heating of the pillar clearly indicate that there is no significant time dependency involved in the spalling process. It is not enough that the stress is close to the spalling strength for a long period of time for the rock to spall.

Figure 6-4. Illustration of the yielding locations numbered in the order they occurred placed on top of the geological mapping. Circles indicate LVDT positions and squares visual observations.
The rock mass yield strength is defined as the tangential stress 3 mm into the pillar at a point where yielding of the rock mass was clearly identified. The total tangential stresses in the pillar for points 2, 10, 17 and 20 (Figure 6-4) are shown in Figure 6-5. The complete yield stress data is presented in Table 6-1.

![Figure 6-5. Total tangential stress for four instrument locations in the pillar. Filled grey circles indicate the time of yielding and hence the yield strength for that point.](image)

<table>
<thead>
<tr>
<th>Point #</th>
<th>Depth (m)</th>
<th>Yield stress (MPa)</th>
<th>Yield stress in % of $\sigma_c$</th>
<th>Geology</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>1.96</td>
<td>127</td>
<td>60</td>
<td>Fresh</td>
</tr>
<tr>
<td>2</td>
<td>2.46</td>
<td>120</td>
<td>57</td>
<td>Fresh</td>
</tr>
<tr>
<td>3</td>
<td>2.52</td>
<td>114</td>
<td>54</td>
<td>Fresh, end pegmatite vein</td>
</tr>
<tr>
<td>4</td>
<td>2.46</td>
<td>116</td>
<td>55</td>
<td>Fresh, pegmatite vein</td>
</tr>
<tr>
<td>5</td>
<td>2.52</td>
<td>124</td>
<td>59</td>
<td>Fresh</td>
</tr>
<tr>
<td>6</td>
<td>2.9</td>
<td>128</td>
<td>61</td>
<td>Fresh</td>
</tr>
<tr>
<td>7</td>
<td>2.99</td>
<td>119</td>
<td>56</td>
<td>Fresh, touches mylonite</td>
</tr>
<tr>
<td>8</td>
<td>2.95</td>
<td>129</td>
<td>61</td>
<td>Fresh, little pegmatite</td>
</tr>
<tr>
<td>9</td>
<td>3.07</td>
<td>119</td>
<td>57</td>
<td>Fresh</td>
</tr>
<tr>
<td>10</td>
<td>2.99</td>
<td>129</td>
<td>61</td>
<td>Fresh</td>
</tr>
<tr>
<td>11</td>
<td>3.20</td>
<td>128</td>
<td>61</td>
<td>Fresh, little pegmatite</td>
</tr>
<tr>
<td>12</td>
<td>3.07</td>
<td>128</td>
<td>61</td>
<td>Fresh</td>
</tr>
<tr>
<td>13*</td>
<td>3.95</td>
<td>117</td>
<td>56</td>
<td>Fresh, close to thin mylonite. Close to bolt</td>
</tr>
<tr>
<td>14*</td>
<td>4.50</td>
<td>111</td>
<td>53</td>
<td>Greenstone inclusion,</td>
</tr>
<tr>
<td>15*</td>
<td>4.80</td>
<td>93</td>
<td>44</td>
<td>Oxidized, mylonitic, close to pre-existing fracture</td>
</tr>
<tr>
<td>16</td>
<td>3.50</td>
<td>124</td>
<td>59</td>
<td>Fresh</td>
</tr>
<tr>
<td>17</td>
<td>3.50</td>
<td>133</td>
<td>63</td>
<td>Fresh</td>
</tr>
<tr>
<td>18</td>
<td>3.58</td>
<td>125</td>
<td>59</td>
<td>Fresh</td>
</tr>
<tr>
<td>19</td>
<td>3.58</td>
<td>133</td>
<td>63</td>
<td>Fresh</td>
</tr>
<tr>
<td>20</td>
<td>4.10</td>
<td>129</td>
<td>61</td>
<td>Fresh</td>
</tr>
<tr>
<td>21</td>
<td>4.10</td>
<td>119</td>
<td>57</td>
<td>Fresh, close to vertical pegmatite vein</td>
</tr>
</tbody>
</table>

Mean values +/-Std

* Point is not used when the mean value of the spalling strength is calculated.
Seven uniaxial compressive strength tests and eight triaxial compressive strength tests were carried out on samples taken from the experiment volume using the procedures recommended by the ISRM, Brown (1981) and Hakala and Heikkilä (1996). The stress magnitudes associated with crack initiation and volumetric strain reversal plus the peak strength were determined for each sample using the methodology first proposed by Brace et al. (1966). The crack initiation stress was determined using both strain gauges and counting of acoustic emission events. Five of the UCS samples were taken within 1 to 2 m of the pillar location.

Using the methodology suggested by Martin and Chandler (1994), crack volumetric strain reversal was used as an indicator of the crack initiation stress. To calculate the crack volumetric strain, the elastic volumetric strains, \( \varepsilon_{v,e} \) (Eq. 2) are subtracted from the total measured volumetric strains \( \varepsilon_v \) (Eq. 1). \( \nu \) corresponds to Poisson’s ratio, \( E \) to Young’s modulus and \( \sigma_i \) to the axial stress. Since only uniaxial compressive tests are compared the \( \sigma_3 \) term is 0.

\[
\varepsilon_v = \frac{\Delta V}{V} \approx \varepsilon_{axial} + 2\varepsilon_{lateral} \quad (1)
\]

\[
\varepsilon_{v,e} = \frac{\Delta V}{V_{elastic}} = \frac{1 - 2\nu}{E} (\sigma_i - \sigma_3) \quad (2)
\]

The crack initiation stress has also been determined for triaxial tests, and the slope of this best-fit crack initiation stress locus has been determined by a least-squares approximation to be:

\[
\sigma_i = 0.44\sigma_c + 2.17\sigma_3 \quad (3)
\]

Back calculations of the observations of spalling during the heating phase indicated that the tangential stress required to initiate spalling had to exceed 0.58\( \sigma_c \). These results are in close agreement with findings from the Mine-by Experiment reported by Martin & Read (1996). The fact that the findings from the Mine-by test tunnel in massive unfractured Lac du Bonnet granite are similar to findings from the Äspö Pillar Stability Experiment suggests that local moderate fracturing and heterogeneity does not significantly affect the spalling strength.

A correlation between notch depth and the ratio of the maximum tangential stress and the uniaxial compressive strength for granitic rock is given in Martin et al. (1997). APSE data are plotted together with these results in Figure 6-6, and the APSE data fit the relationship derived by Martin quite well.

![Figure 6-6. Relationship between depth of yielding and maximal tangential stress at the boundary of the opening. Modified from Martin et al. (1997).](image)
7 Effect of confinement

At the planning stage of the experiment it was decided that when the spalling had propagated far enough down the hole and the temperature and AE frequency had reached a steady state the confinement pressure should be released in 50 kPa decrements.

The first 50 kPa decrement of the confinement pressure was done at around noon at July 14. There was no response in the rock that was detected by the acoustic system and there were no displacements.

The majority of all AE events and displacements during the release of the confinement occurred when the pressure was reduced to 150 kPa. During a 4-hour period after that pressure drop, displacements and AEs occurred at an increased level, Figure 7-1.

The last pressure drop for the day should reduce the pressure to 100 kPa. Unfortunately, the pressure transducer malfunctioned during that drop and transmitted a constant value. This was not noticed until it was too late and all the pressure had been released. The actual pressure in the graph should therefore be zero after 21.30 h. Note that the hole was still filled with water.

It is interesting to note that after 21.30 h, when the pressure was reduced from 150 kPa to 0, the acoustic response was rather limited with small accompanying deformations. One might expect that such a large pressure drop would induce a great deal of acoustic activity, which was obviously not the case.

The acoustic activity quickly died off after the total release of the confinement. In the afternoon of July 16 the hole was totally evacuated of water, but that did not cause any additional AE.

The effect of the confining pressure was clearly demonstrated. During the heating phase of the experiment, very few AEs originated from the confined side of the pillar. AE frequency and yielding damage during release of the confinement pressure were also very low at that side. Instead the vast majority of the AEs and the dilatation were concentrated at the notch tip location in the open hole. The fact that yielding initiated here is perfectly consistent with the observations made of spalling in general and in this experiment in particular. The part of the notch located deepest into the pillar, and particularly the notch tip, is in a highly instable equilibrium. Extremely small stress changes trigger the yielding process. The reduction of the confinement is a good example of this. A pressure reduction of 450 kPa in the confined hole was enough to initiate the spalling process again in the open hole. The change of stress in the open hole due to the release of the confinement pressure was intuitively perceived to be very small but has not been quantified. The rock then responded to each of the remaining 50 kPa reductions.

![Figure 7-1. Confinement pressure and acoustic frequency in counts per hour during the release of the confinement.](image-url)
To be able to make as good a geological characterization of the pillar as possible and to study the fracture pattern located inside the pillar caused by the yielding, it was decided to completely remove the pillar from the site. This was done by sawing it into five blocks, each with a height of approximately 1m. To be able to saw the blocks the pillar had to be de-stressed. This was done by the drilling of a slot on the left side of the pillar.

Predictive numerical modelling with Phase2 indicated that the rock was likely to fail during de-stress drilling. A hole visit after drilling approximately half the slot reviled that extensive yielding had taken place as is indicated in Figure 8-1. Before de-stress drilling was initiated, the notch wall was smooth and no slabs were loose.

When the first block was removed it became evident that a certain amount of failure had taken place. The failure was limited to the left part of the pillar, which is closest to the slot, and was visible as sub-vertical failed zones close to the edges of the blocks. It appears as if the stresses punched the pillar against the slot and that failure then occurred.

The failed zones are clearly visible when looking at the tops of the blocks. When they are studied, the fracturing can be seen to end at the approximate centre of the blocks. Tensile fractures have developed in a horsetail like pattern on the right-hand side of the blocks. The pattern is visible on blocks 2 and 3 but is practically disappeared at block 4. Photographs of the top of blocks 2, 3 and 4 are shown in Figure 8-2.

Numerical modelling of the de-stress drilling was made using the 2D FEM code Phase2D. Both the Mohr-Coulomb and the Hoek - Brown failure criteria were used. The modelling of the de-stress drilling clearly illustrates that neither the Mohr – Coulomb or the Hoek – Brown failure criteria represent the behaviour of the rock at low compressive or tensile stresses accurately enough. The Hoek – Brown failure criterion greatly underestimates the rock mass strength and the Mohr – Coulomb criterion indicates a failed zone that not was observed in situ.
Figure 8-2. Photographs of the tops of block 2, 3 and 4 with the general fracture pattern. The colours have been inverted to better illustrate the fracturing. The white traces of the failed zones are clearly visible above the centre line of the tunnel.


9 Discussion and conclusions

The rock mass yielding (spalling) strength for the Äspö diorite was calculated at 18 discrete points at the pillar wall and determined to be $(0.58\pm0.04)\sigma_c$. This value does though not apply to mylonitized or highly oxidized granitic rocks where the yielding strength was slightly lower than the yielding strength for fresh Äspö Diorite. The onset of crack initiation in uniaxial laboratory tests, determined from strain gauge data, was found to occur at approximately $(0.45\pm0.03)\sigma_c$ and with the AE method it was assessed to $(0.56\pm0.16)\sigma_c$.

The visual observation showed that the extent of spalling is sensitive to small changes in the stress magnitudes. It was determined using three dimensional modelling that changes in the tangential stress magnitude of only approximately 1 MPa was sufficient to cause yielding of the pillar to propagate. However, observations suggest that without this stress change yielding of the rock mass would not occur. In other words, there was essentially no evidence for time-dependent processes over the monitoring period used in the experiment.

The fractures formed during the yielding first forms as small chips tangential to the borehole wall underneath larger and larger chips are formed. The observations of the yielding process indicate that except from in the deepest part of the notch all fractures are formed in tension.

The slabs formed during the experiment are likely initiated at flaws located inside the pillar wall at the location of the maximum tangential stress and then grows towards the boundary of the hole as extension fractures. The extremely thin chips/slabs that were the first ones to develop are an indication that there is no evidence of an excavation damaged zone that has re-distributed the stress at the boundary of the hole.

The APSE results are confirmed by other experiments and observations but there are also cases where the results not apply. The results can hence not be applied to any granitic rock without studying the properties of the particular rock mass in detail.

It has been shown that the initiation of yielding is highly sensitive to low confinement pressures, a few hundred kPa.
10 References


Martin, C.D., Read, R.S. 1996. AECL’s Mine-by Experiment: A test tunnel in brittle rock. In the proceedings of NARMS 1996, Quebec, Canada.


