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In-situ experiment concerning thermally induced spalling of circular shotcreted shafts in deep crystalline rock

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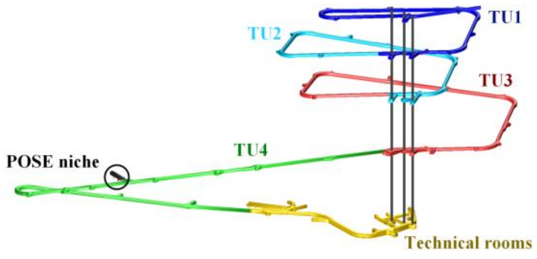
ABSTRACT: A thermally induced shotcrete spalling in-situ experiment will be carried out during 2013 in the ONKALO rock characterization facility on the west coast of Finland. In the technical areas of the planned spent nuclear fuel repository there will be shotcreted rock spaces which will be subjected to thermally induced deformations and stresses. In this heating experiment, we are attempting to establish the failure strength of shotcrete on a pre-stressed rock surface, when the stress state in both the rock and shotcrete are increased by heating. The in-situ strength can be used to evaluate the safety of the shotcreted structures. The interaction between the shotcrete and the rock mass will be observed as the rock approaches its strength limit. Posiva's Olkiluoto Spalling Experiment niche is ideal for such an experiment due to the amount of supplementary data from preceding experiments. The area is already built, instrumented and contains running experiments. The test arrangement is described and a prediction of the expected results is provided. Similar research has been carried out using fire-induced thermal actions, but this is the first test to use heaters in the surrounding rock mass in the sub-boiling thermal range.

1 Introduction

The intent of the experiment is to define the failure strength of shotcrete on a pre-stressed rock surface, when the stress state of both rock and shotcrete are increased by heating. The experiment will be carried out in the ONKALO rock characterization facility on the west coast of Finland. The location is within Posiva's Olkiluoto Spalling Experiment (POSE) niche at a depth of -345 m (Fig. 1a). The third hole, (POSE-EH3) with a diameter of 1.524 m (radius 0.762 m) and depth 7.2 m, will be used (Fig. 1b). In the preceding experiment, the POSE-EH3 is heated from inside to study thermally induced damage caused by the excess heat, as a simulation of the heat from a spent nuclear fuel canister. After the experiment, the hole is allowed to cool down for 20 weeks and any damaged rock is removed. After this, eight new holes (Ø76 mm, L = 7.5 m) for the heaters will be drilled outside the main hole and the surface will be shotcreted. The central distance to the heater holes is 1.762 m (heater hole centres are 1 m away from the hole surface). This will simulate shotcreted shafts (diameter 3.5 m for ventilation and 4.5 m for personnel) in the technical areas heated by the surrounding panels of arrays of spent nuclear fuel canisters. The heating power will be increased in steps: 8 x 1000 W for 3 weeks, 8 x 1500 W for 2 weeks, 8 x 2000 W for 4 weeks and 8 x 0 W for 7 weeks (total duration 16 weeks). For the inside shotcrete, this will lead to a maximum temperature of just below 130° C, which is the maximum temperature tolerance of the strain gauges. During the experiment, strain gauges will record the stress and thermocouples will record the temperature increase. The acoustic emission (AE) sensor array from the preceding experiment will be reused. The temperature and corresponding stress to induce damage will be recorded. In particular, the damage suppressing effect of shotcrete is expected to be observed and recorded.



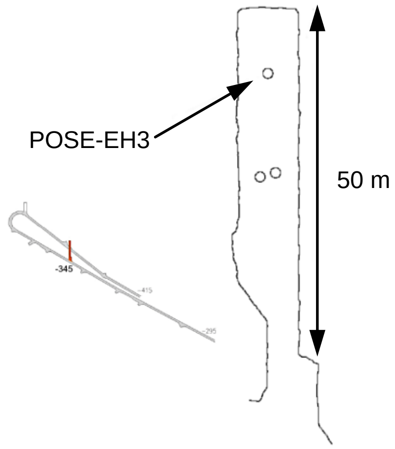


Figure 1a. Location of the POSE niche (Siren 2011)

Figure 1b. Location of the POSE-EH3 hole

2 Previous shaft reinforcement thermomechanical modelling results

Thermomechanical modelling has been carried out for the existing shafts in the ONKALO to simulate the long term temperature increase due to the excess heat produced by the spent nuclear fuel canisters. Based on thermal conductance modeling, the access ramp (Fig. 1a) will experience higher loads than the shafts which are further away from the panels. These shafts were chosen for more precise study because they have an even circular shape which concentrates stresses and because they are difficult to maintain and have a long service life. The ventilation contribution has not been considered.

The modelling was carried out by calculating the temperature increase after 120 years (which is the planned operating time of the repository before sealing). The obtained thermally induced stress was implemented in a 2D-FEM model (Phase2 8.007) in order to assess the possible induced damage both in the shotcrete and in the rock mass. Two diameters have been considered: 3.5 m (ventilation shafts) and 4.5 m (personnel shaft). The calculation was verified using analytical methods. The maximum temperature increase at the shaft distance (approximately 150 m from the closest panel) has been around 4°C at the depth of -415 m, which corresponds to a hydrostatic in-situ stress increase of 2 MPa. The boundary of the shaft is already subject to significant stress, with a tangential stress $\sigma_{\theta\theta}$ up to 58 MPa (initial horizontal principal stresses σ_1 = 24.8 MPa and σ_2 = 16.6 MPa).

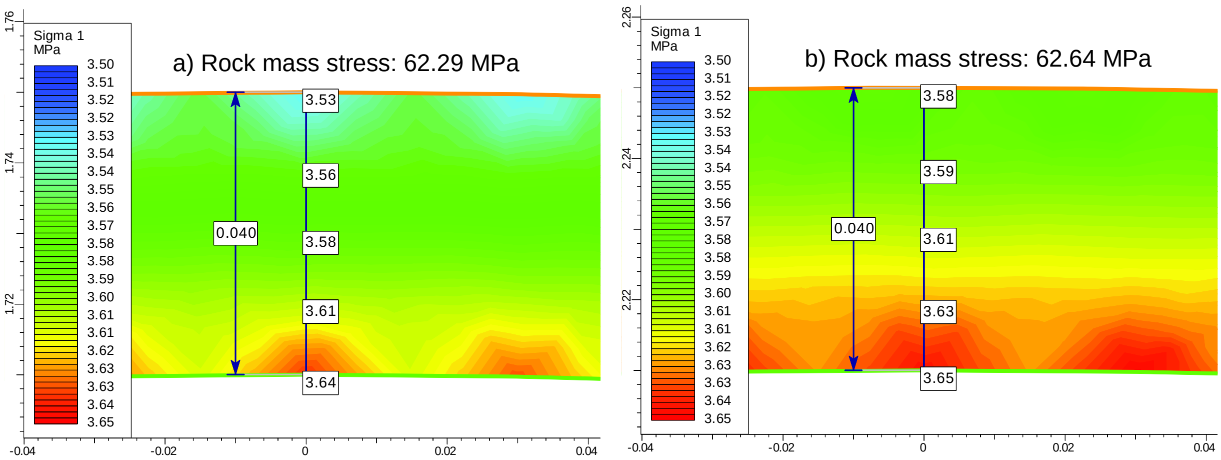


Figure 2. Stress distribution for the shotcrete in the case of the section at -420 m with a 40 mm shotcrete layer and diameter of a) 3.5 m or b) 4.5 m

For a 40 mm layer of shotcrete, the results of the calculation show a low influence of the temperature increase: the maximum increase of the tangential stress in the shotcrete was 3.65 MPa (Fig. 2) at the internal boundary of the shotcrete, where the stress is at its maximum level. The difference between the internal and external boundary of the shotcrete layer is about 3% (less than 0.1 MPa). The difference between two different diameters is negligible. There might be a larger influence because, by increasing the stress, also the level of cracking of the rock mass is being increased, and therefore the shotcrete may lose some adhesion with the shaft boundary, or cracks may be created in it.

Nevertheless, this kind of reinforcement, coupled with systematic bolting in weaker areas, has been considered as the most suitable for the personnel shaft.

3 Numerical modelling

The problem was modelled numerically using the 3D COMSOL Multiphysics 4.3a Thermal Stress (TS) module and 2D fracture mechanics code Fracod 4.11. The initial data used are presented in Tables 1 to 3 and Figure 3. The COMSOL model considers the thermally induced stress only and ignores in-situ stress magnitude and direction. The COMSOL model exploits double symmetry and only 1/16th of the area was modelled (Fig. 3). The modelled rock mass is quite large (radius 10.5 m and height 20 m) to allow for thermal conduction. This creates an error on the sides as the wall is missing and is not restricting upwards movement.

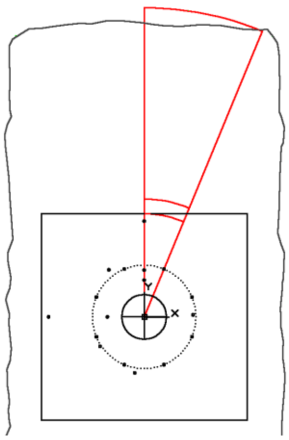


Figure 3a. POSE-EH3, modelled area

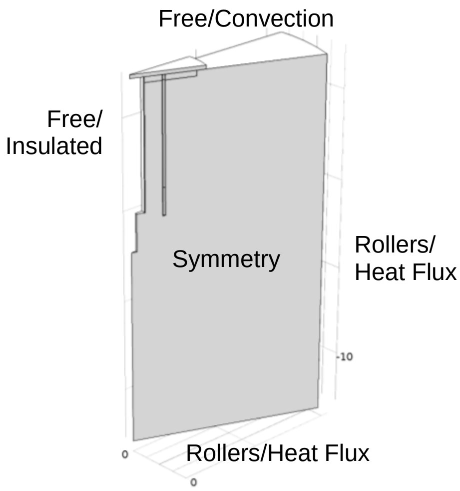


Figure 3b. Boundary conditions

Table 1. Material properties

Property	symbol	pegmatitic granite	shotcrete C35-3/45-1	insulation Paroc Extra XS	unit
Elastic modulus	E	53[1]	34[2]	0.16[6]	GPa
Poisson's ratio	v	0.25[1]	0.20[2]	0.10[6]	
Density	ρ	2635[1]	2200[3]	28.5[7]	kg/m3
Thermal capacity	C_p	716[1]	840[5]	840[7]	J/kgK
Thermal conductivity	k	3.33[1]	1.7[4]	0.0353[7]	W/mK
Linear thermal expansion	α	9.76e-6[1]	10e-6[2]	10e-10[6]	1/K

[1] *Valli & Hakala 2012* [2] *EN 1992-1-1:2004* [3] *based on quality assurance tests* [4] *SRMK C4: 2003*
[5] *Neville 1995* [6] *arbitrary value to prevent the insulation from acting on the rock or the shotcrete mechanically*
[7] *manufacturer quote*

Table 2. Fracture mechanics modelling parameters

Property	symbol	value	unit
Cohesion	c	12.9[1]	MPa
Friction angle	φ	47[1]	°
Tensile strength	$\sigma_{T,}$	12[1]	MPa
Major principal stress	σ_1	23 (305) [2]	MPa (°)
Intermediate principal stress	σ_2	15 (215) [2]	MPa (°)
Minor principal stress	σ_3	3 (vert.) [2]	MPa (°)
Rock tensile fracture toughness	K_{IC}	1.96[3]	MPa$\sqrt{m}$
Rock in-plane shear fracture toughness	K_{IIC}	3.30[3]	MPa$\sqrt{m}$
Concrete tensile fracture toughness	K_{IC}	1.50	MPa$\sqrt{m}$

Concrete in-plane shear fracture toughness	K_{IIC}	3.00	MPa $\sqrt{m}$
Cohesion – tensile & shear	c	$10^{(1)}$	MPa
Friction angle – tensile & shear	φ_t, φ_s	$35^{(4)}$, $35^{(4)}$	°, °
Dilatation angle – tensile & shear	ψ_t, ψ_s	$2.5^{(4)}$, $2.5^{(4)}$	°, °
Normal stiffness – tensile & shear	k_n	$20{,}000^{(1)}$	GPa/m
Shear stiffness – tensile & shear	k_s	$2{,}000^{(1)}$	GPa/m

(1) Siren 2011 (2) Situation below the tunnel floor including in-situ and tunnel effect at depth of -3 m after Valli & Hakala 2012 (3) Modified after Siren 2012 (4) Posiva 2009, table 5-6

Table 3. Shotcrete strength for C35-3/45-1

Property	symbol	compressive	tensile	unit
Design strength	f_{cd}, f_{ctd}	$22^{(1)}$	$1.6^{(1)}$	MPa, MPa
Characteristic compressive strength	f_{ck}, f_{ctk}	$35^{(2)}$	$2.2^{(2)}$	MPa, MPa
Mean strength	f_{cm}, f_{cm}	$43^{(2)}$	$3.2^{(2)}$	MPa, MPa

(1) EN 1992-1-1:2004 and NA SFS-EN1992-1-1-YM:20074 (2) EN 1992-1-1:2004

4 Instrumentation plan

After the third phase of the main POSE experiment, the ONK-EH3 hole will be scaled manually and shotcreted using the same method, equipment and materials as in the shafts—to the extent that this is feasible. Eight heater holes (Ø = 76 mm, L = 7.5 m) will be drilled symmetrically around the hole and 6 m long heaters with maximum capacity of 4000 W are installed in them. Thermal conduction is improved (and convection effects reduced) by using Tabular Alumina (Aluminum Oxide) in the heater holes. The thermal sensors and strain gauges on the rock surface from the previous experiment are reused if functional or replaced. Additional thermal sensors and strain gauges will be installed on the surface of the shotcrete at levels -3 m and -6 m. The 120° dual pattern of tangential strain gauges and strain gauge rosettes is used from the preceding POSE 3rd phase experiment. The acoustic emission system from the preceding experiment is reused without modifications. The heating period lasts for 9 weeks and the monitors are read at each 15 minutes to produce a time series.

Figure 4 illustrates the test instrumentation: the red triangles (8) are new heater holes for this experiment, the blue dots (24) are AE sensors, the red dots (44) are thermal sensors (thermoelement monitors), the green dots (6) are tangential strain gauges, the yellow dots (9) are rosette strain gauges

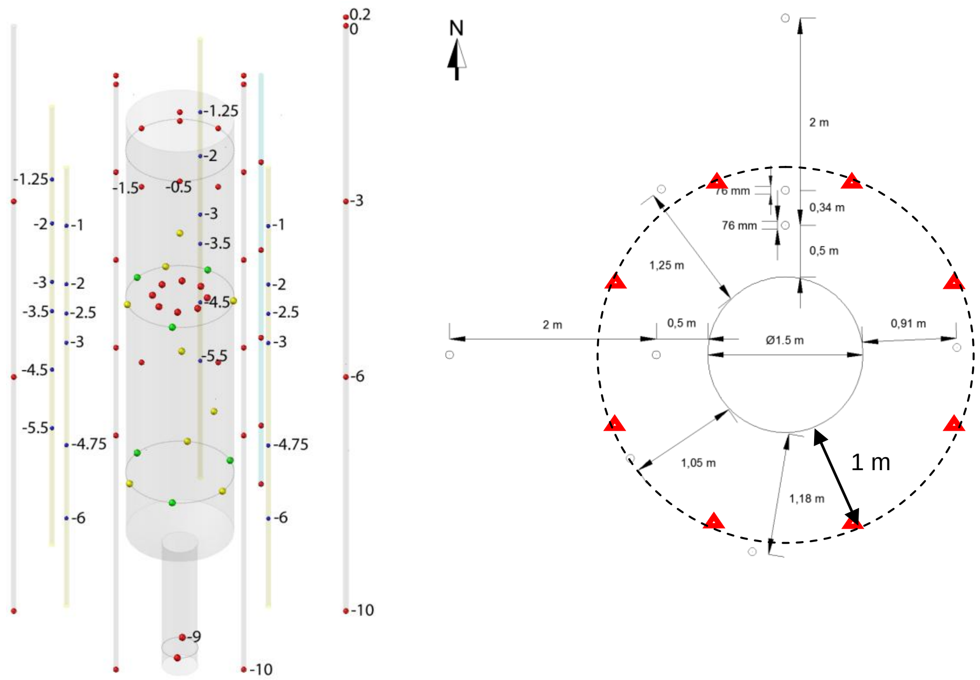


Figure 4. The instrumentation plan around the experiment hole (modified after Valli & Hakala 2012)

5 Predicted results: thermomechanical finite element method

The heat increases at a relatively steady rate of 12° C / week, slowing down towards the end (Fig. 5). 100° C is reached during week 7, which should be enough time for all the water to evaporate before being converted into steam. At the end of week 9, the temperature will peak out at 125° C. The seven week cooling period brings the temperature back down to 35° C. Heater holes will reach a maximum temperature of 160° C at the hole top and 140° C along the top three metres.

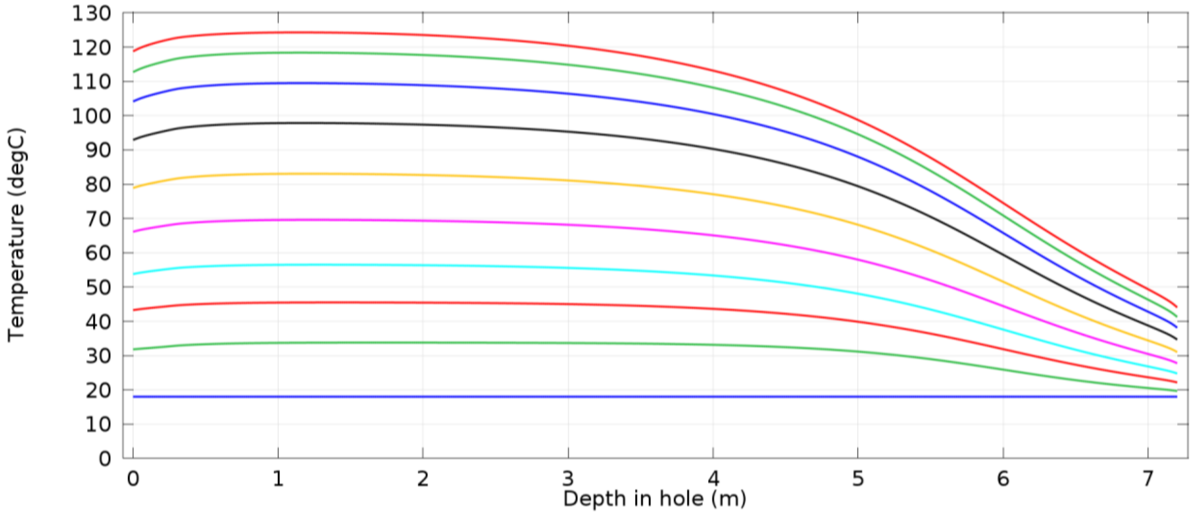


Figure 5. Temperatures measured from the centre of the shotcrete layer (weeks 0–9)

No damage in the shotcrete is expected during the first three weeks (Fig. 6). After five weeks, there is a 5% chance of damage and after seven weeks a 50% chance of damage. After nine weeks of heating, the stress will peak at 53 MPa, which is 23% higher than the mean strength of the shotcrete. The highest loaded region is around -3.5 m which is close to the -3 m monitoring level. The second monitoring level at -6 will peak at a much lower stress of 35 MPa, which is still higher than the design strength of the shotcrete.

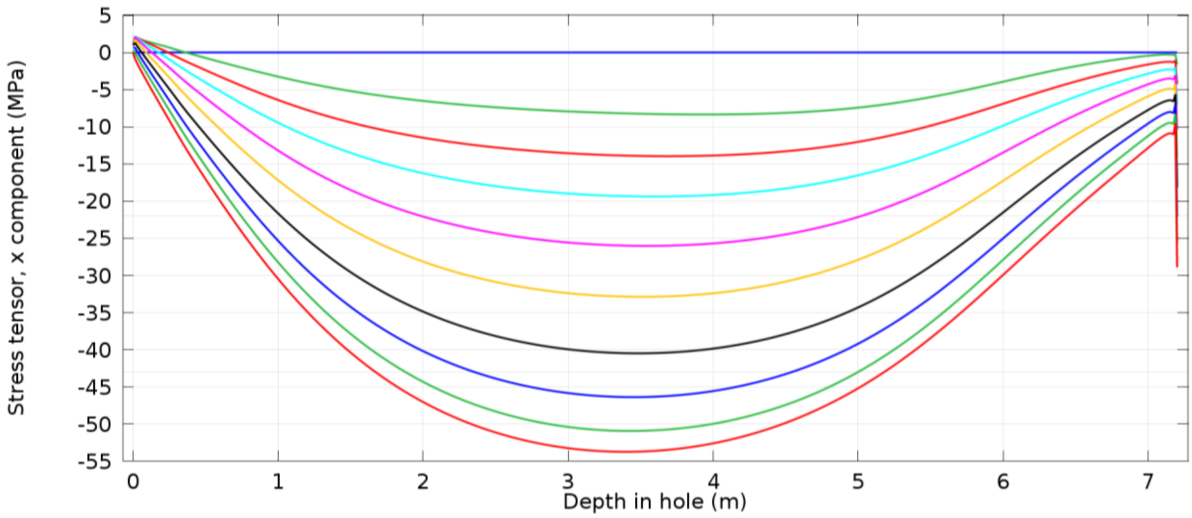


Figure 6. Tangential stresses measured from the inner surface of the shotcrete layer (weeks 0–9)

The stresses in the rock are 1.56 times higher due to the difference in elastic modulus (Fig. 7). The peak tangential stress increase of 80 MPa is reached at the end of week 9. This stress acts together with the in-situ stress (Table 2). The initial tangential stress around the hole is estimated to be 46…54 MPa and the combined stress should be 126…134 MPa. It is possible that the rock wall may sustain damage as the estimated damage strength is 58...102 MPa. The shotcrete produces a support pressure which may reduce the extent of the damage (Glamheden et al. 2010). It should be noted that most likely the rock wall will have already been damaged by the preceding POSE 3rd phase experiment which can have creates an asymmetrical loading situation causing localized damage.

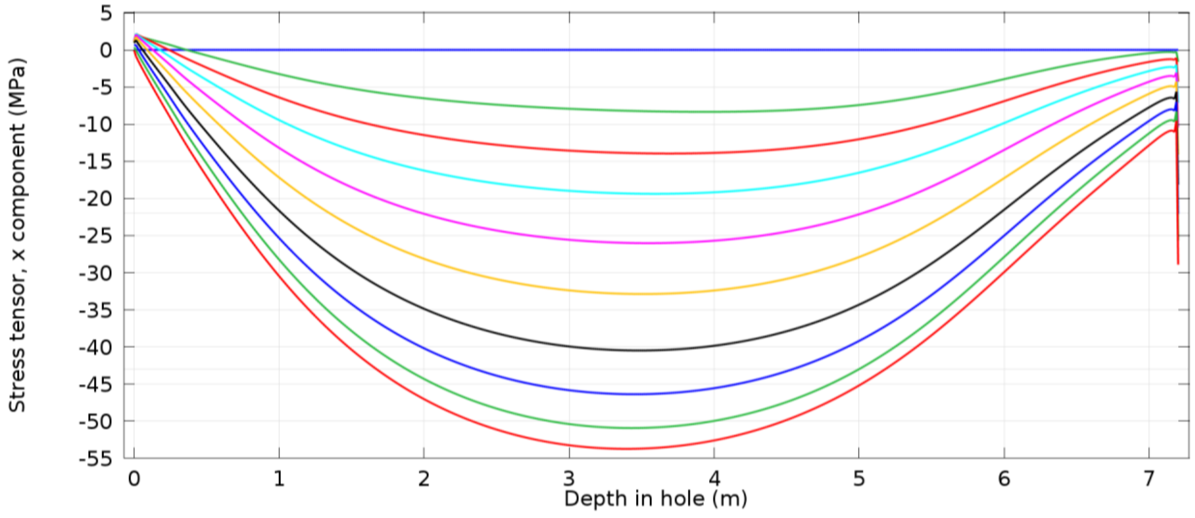


Figure 7. Tangential stresses measured from the inner surface of the rock mass (weeks 0-9)

The hole moves upwards and outwards at top which causes tensile stresses to the topmost 1.5 metres of the hole. The calculated stresses exceed the design tensile strength after week 5, but peak out at 2.1 MPa and never reach the characteristic tensile strength level. Some tensile cracks may be observed in the top part of the hole.

6 Predicted results: thermomechanical fracture code

The input parameters are determined by mainly using existing test results (Siren 2011, 2012) for pegmatitic rock (PGR), which is assumed to be continuous, isotropic, homogeneous and linearly elastic. The input parameters, as listed in Table 2, are used. In the fracture mechanics code, the thermal evolution calculated using constant timesteps and the thermal strength is an average of the beginning value and the end value of this time step. Therefore, the thermal evolution is simplified as presented in Fig. 8. On the hole surface, a heat flux is set to zero corresponding to the fully insulated condition. The experience from previous experiments is that modelling with zero thermal flux corresponds well with observations.

In the prediction, the fracturing initiates at the rock surface (Fig. 9 on left) and expands to form a spalling type failure behind the shotcrete surface (Fig. 9 in middle). After the spalling in rock, the stresses in the shotcrete exceed the failure criterion. After that, there is a clear shotcrete failure and rockburst of spalled rock behind the failed shotcrete layer (Fig. 10 in red) where 1–2 mm displacements (Fig. 9 on right) move towards the experiment hole due to tensile stresses. After the failure, even higher compressive stresses are present but the tensile stresses remain close to 1 MPa. The stresses in the fracture mechanics code become locally extremely high after the shotcrete failure and these high stresses are not described in this paper because of the fracture tip interaction within the complex fracture geometry.

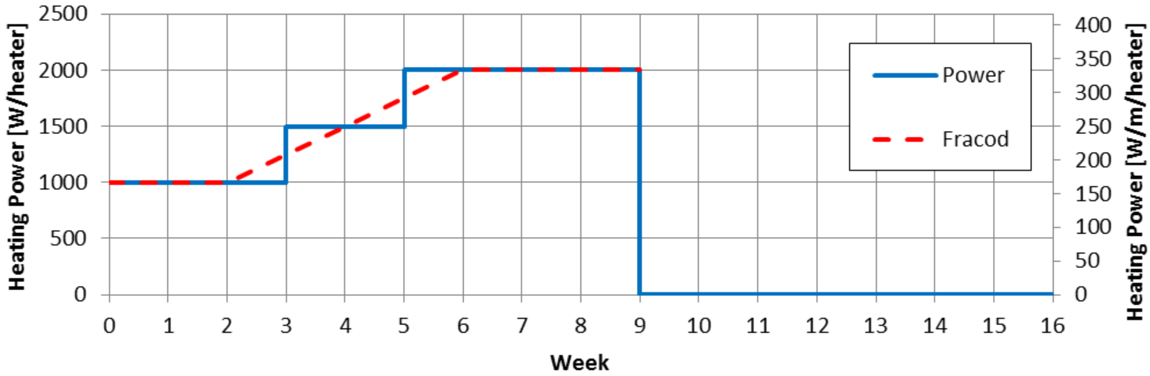


Figure 8. The heating pattern of each heater hole (left scale) and distributed power (right scale)

Glamheden et al. (2010) stated that a support pressure created will prevent spalling, even at small support pressures. In the modelling, the shotcrete layer was removed to test this hypothesis and it increased the depth of spalling significantly. In the model without shotcrete after the main spalling in the minor principal stress direction, reduced spalling will also initiate in other directions as well. This is not observed at such a large scale in the model with shotcrete.

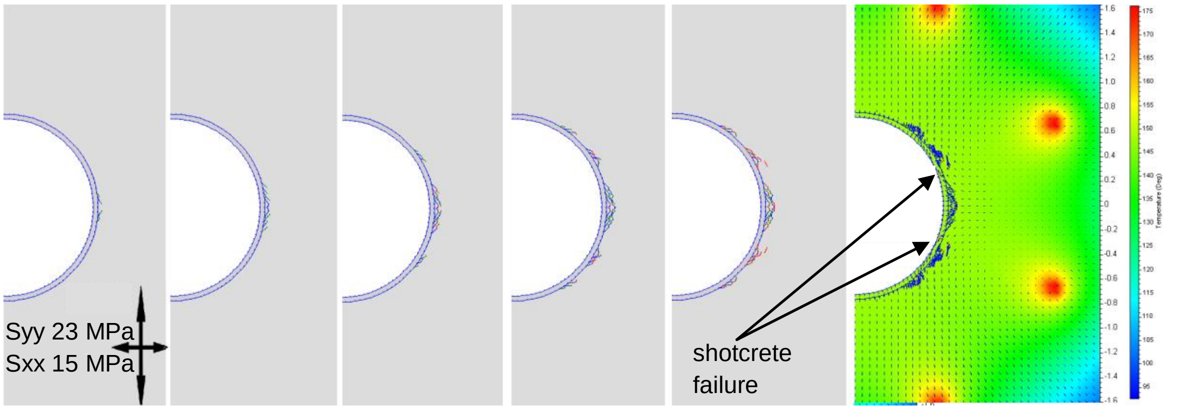


Figure 9. Predicted development of fracture initiation and propagation during nine weeks of heating. The temperature distribution, displacement vectors and predicted fractures after nine weeks are shown on the right. The shotcrete failure is indicated directly.

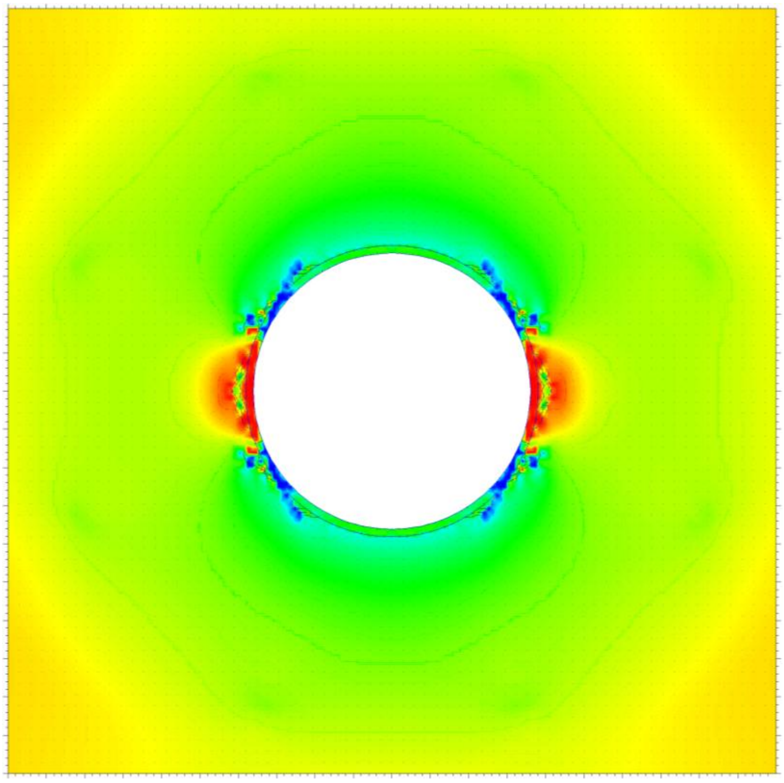


Figure 10. Principal major stress immediately after shotcrete failure. The shotcrete fails in the minor principal stress direction causing the tensile area (red) and spalling.

7 Conclusions

The shotcrete spalling experiment will use significantly higher temperatures and thermal stresses than those expected during the operation of the spent nuclear fuel repository. FEM and fracture mechanics codes suggest that the rock mass is damaged first. The experiment will generate information on how much support pressure the shotcrete can produce and whether it is enough to retain the damaged rock. The stresses will eventually reach the mean strength of the shotcrete with the fracture mechanics code suggesting high stresses and failure in the shotcrete.

The stresses concentrated around the shafts are high and it is useful to establish the in-situ limit strength of the structure and the rock mass. Calculations for the ONKALO shaft shotcrete liner stability during the 120 years operation time indicated that the shotcrete will not suffer significant damage. However, it is unclear what effect the loss of adhesion at the shotcrete-rock interface will have.

8 Acknowledgements

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