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### THERMALLY INDUCED ROCK STRESS INCREMENT AND ROCK REINFORCEMENT RESPONSE

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### ABSTRACT

The thermal heating caused by the deposition of spent nuclear fuel containers increases the *in situ* rock stress during disposal time. The thermal stress increase was modelled using thermo-mechanical modelling. The numerical codes used to establish the effects of heating on the in situ stress field are outlined, together with the rock mass parameters, in situ stress values, radiogenic temperatures and reinforcement structures. This is followed by a study of the temperature and stress evolution during the repository's operational period and the effect of the heating on the reinforcement structures. It is found that, during excavation, the maximum principal stress is concentrated at the transition areas where the excavation profile changes and that, due to the heating from the deposition of spent nuclear fuel, the maximum principal stress rises significantly in the tunnel arch area of NW/SW oriented central tunnels. However, it is predicted that the rock's crack damage (CD, short term strength) value of 99 MPa will not be exceeded anywhere within the model. An additional study of the radiogenic heating effect on the brittle deformation zones is included. The main conclusion is that, despite deep reaching damage potential in all the load cases studied the currently designed and used reinforcement types and configurations (rock bolts, shotcrete) are capable of handling the dead weight of the damaged rock should this occur, with damage occurring on the shotcrete liner. The long term safety and stability of the repository during its lifetime can be guaranteed by perceiving the reinforcement strategy in two stages. Firstly, by installing the rock reinforcement to sustain the initial stresses and short term increases from the start of deposition with a monitoring programme in place. Secondly, by installing additional reinforcement, if found necessary through monitoring and observation of the underground facilities. In this way, the effect of any time dependent rock stress increase affecting the reinforcement structures can be observed, in addition to creep based damage, thus providing a better level of safety than a single stage design.

#### **KEYWORDS**

Olkiluoto repository, radiogenic heating, rock reinforcement, rock damage modelling, design, monitoring

#### INTRODUCTION

The nuclear waste disposal projects in Finland and Sweden are heading for the implementation phase in the next ten years or so (2015-2025). In Olkiluoto Island, located in Western Finland, Posiva Oy has built ONKALO rock characterization facility in the future nuclear waste disposal site for confirming site studies. Also the ONKALO technical facilities are used during the disposal project for c.a. 120 years. During this time the rock reinforcement structures are vulnerable to failures.

Therefore the rock reinforcement solution needs to be able to withstand the increased stress state due to the thermal output from final deposition of nuclear waste containers. Reports describing the thermal output from the nuclear waste containers, and time-dependent evolution of the temperature within the rock mass, have been published on several occasions (Ikonen, 2003; Ikonen, 2005; Ikonen & Raiko, 2012). However, in this article, the time-dependent evolution of the prevailing *in situ* stress state coupled with the radiogenic thermal stress evolution is presented. This coupled stress state is then used as an input parameter for the design of reinforcement, imposing increased demands on the rock reinforcement structures.

## DISPOSAL AND ROCK REINFORCEMENT STRATEGY

The disposal of the nuclear waste has been planned to be conducted in stages, taking up to 120 years. Within the current frame of the plan, the deposition will advance on a single level at a depth of -420 m. The planned disposal will involve the used fuel rods from the reactors OL1, OL2 and OL3 at Olkiluoto, and from the reactors LO1 and LO2 located at Loviisa nuclear power plant. The current disposal plan, illustrated in Figure 1, will act as a basis for the Thermo-Mechanical modelling of the stress regime at the deposition level. Hence, importing of the heat sources on the large scale 3DEC model for the analysis of the prevailing stress state will be constructed according to the deposition sequence described in Hakala *et al.* (2014). Each of the reactor types produce a different type of spent nuclear fuel rods as a side-product and this has been taken into account in the modelling. The disposal layout and model dimensions are presented in Figure 1.



Figure 1. Illustration of the disposal layout and model dimensions.

The rock reinforcement strategy is based on several key items influencing the necessity, type and amount of required rock reinforcements. The key items affecting the reinforcement quantity and strategy are listed below.

- The rock reinforcement planning in ONKALO is based on Eurocode 7, in particular the Observational Method component.
- Repository demands and boundary conditions imposed on reinforcement types, lifetime, and materials are each taken into account within the strategy.
- The effect of *in situ* rock stress, and geological conditions, including groundwater constraints imposed on the materials.
- Extensive modelling of the rock behaviour, while taking into account different types of stress-strain behaviour, the respective failure modes and their developments as a function of the repository lifetime.
- The phenomena noted in the rock behaviour models, and their relation to the long term safety of the repository.
- Experience and methodology developed, in particular during construction of the ONKALO research facility, taking into account the long-time experience in Finland associated with rock reinforcement strategies for similar rock conditions, such as found at the repository site.

# SITE AND BOUNDARY CONDITIONS

Posiva has extensively investigated the *in situ* rock conditions at the ONKALO, and around the future repository site. The rock mechanics parts of the investigations are focused on the conditions relevant for predicting and determining the needs for rock reinforcement in the future repository. The stress state at Olkiluoto is presented in Table 1.

Elastic and thermal parameters of the rock mass are based on Olkiluoto Site Report 2011 (Posiva, 2013) and Posiva's Working Report 2012-56 (Ikonen & Raiko, 2012). The isotropic elastic values, in addition to thermal properties, are presented below in Table 2.

The model size used to calculate radiogenic temperature evolution and induced thermal stresses is 5 km in the E–W direction, 4.5 km in N–S direction, and having a height of 2 km. The horizontal model boundaries are fixed in the horizontal direction, the bottom side of the model is fixed in all directions, and the top side is free, as to represent actual ground level conditions. Boundaries are considered to be far away enough not to interfere with the induced thermal stresses during the simulated time of 120 years of repository lifetime, as presented in Figure 2.

Table 1. Interpreted stress field at Olkiluoto based on *in situ* stress data (99% confidence interval). (Posiva 2013)

Range	σH	σh	σν	Vertical depth range
Mean (MPa)	13.0+0.031z	8.5+0.024z	0.0265z	
Min (MPa)	2.0+0.030z	1.0+0.020z	0.0240z	0 to OL-BFZ020
Max (MPa)	24.0+0.033z	16.0+0.028z	0.0290z	
Orient. (°N)	87 (8–165)	177 (98–255)	-	
Mean (MPa)	10.9+0.033z	5.3+0.027z	0.0265z	
Min (MPa)	0.1+0.032z	-1.7+0.027z	0.0240z	OL-BFZ020 to OL-BFZ099
Max (MPa)	21.7+0.033z	12.3+0.027z	0.0290z	
Orient. (°N)	112 (53–171)	202 (143–261)	-	
Mean (MPa)	10.8+0.033z	2.1+0.026z	0.0265z	
Min (MPa)	3.0+0.030z	-3.4+0.026z	0.0240z	OL-BFZ099 – 900 m
Max (MPa)	18.6+0.036z	7.6+0.026z	0.0290z	
Orient. (°N)	84 (53–115)	174 (143–205)	-	

Stresses interpreted to ranges between major brittle fracture zones

Parameter	Value	Unit
Density (ρ)	2743	kg/m <sup>3</sup>
Young's modulus (E)	55	GPa
Poisson's ratio (v)	0.25	
Uniaxial Compressive strength	108	MPa
Crack Damage strength	99	MPa
Crack Initiation strength	52	MPa
Indirect tensile strength	12.1	MPa
Coefficient of thermal expansion	9.5E-6	1/K
Thermal conductivity	2.82	W/(mK)
Thermal diffusivity	1.34E-6	m²/s
Specific heat	764	J/(kgK)

Table 2. Parameter values used in the thermo-mechanical analyses (Posiva 2013).



Figure 2. Principal stress tensors in the horizontal plane at the time of 120 years after start of deposition in the central part of the model, at a depth level of -422 m. The colour indicates the maximum compression values, ranging from 24 MPa (blue) to 40 MPa (dark red).

#### **TEMPERATURE AND STRESS EVOLUTION**

The maximum temperature increase inside the panel areas at the deposition level is reached in less than a hundred years after the start of deposition. However, the temperatures are still increasing in the central tunnels and technical facilities after 120 years from the start of deposition. The maximum temperature increase of 13 °C takes place in the NE central tunnels; whereas, in the technical facilities, a maximum of 6 °C is observed in the SW parts, as indicated in Figure 3a. These magnitudes of temperature increase are calculated ignoring the cooling effect of ventilation.

The thermal expansion of the rock increases the horizontal stresses significantly (Figure 3b). The rock overburden has the height of 422 m, while the horizontal dimensions of the heated rock volume are of 2.5 km and 1 km. However, due to the unconstrained model top (rock surface) and relatively thin rock overburden compared to heated rock mass dimensions as shown in Figure 1, the vertical stress doesn't increase. The horizontal *in situ* stress components at the deposition depth range from 25 MPa to 16.5 MPa, while the vertical component is estimated to be of the order of 11.5 MPa. The maximum stress increase of 17 MPa takes place in the centre of the deposition panel. Examples of the thermally induced evolution of the *in situ* stress are given in Figure 4.



Figure 3. The a) locations of maximum temperature increase (°C) in the central tunnels, and in the technical facility area after 120 years, and the b) Contours presenting the maximum compressive stress ( $\sigma_1$ ), after 120 years from the start of deposition.



Figure 4. Thermally induced evolution of the *in situ* stress field in four locations. The estimated maximum time of use for each tunnel before the installation of backfilling is indicated with orange arrows.

# THERMAL EFFECT ON ROCK STABILITY

In all the models, the maximum principal stress is concentrated at the transition areas where the excavation profile changes, at both sides of the raised profile as shown as an example in Figure 5a. In these areas, the maximum values for the principal stress can vary between 54 and 58 MPa. In the normal tunnel profile areas, the resulting maximum principal stress has a lower magnitude, between 50 and 54 MPa.

Due to the heating from the deposition of spent nuclear fuel, the maximum principal stress rises significantly in the tunnel arch area (Figure 5b). The maximum values of the principal stresses are still located in the profile transition areas, varying between 75 and 78 MPa. In the normal profile areas, the maximum principal stress magnitudes have reached up to 71 to 74 MPa. When compared against the crack initiation (CI) value of 52 MPa, the middle point of the arch is susceptible to suffering from stress induced damage in all locations, and from all the lengths under observation in the model. The maximum depth of damage is 900 mm, and it is located in the arch corner of several profile changing areas. In the normal profile, the estimated damage depth is around 750 - 800 mm. However, the crack damage (CD) value of 99 MPa is not exceeded anywhere within the model.



Figure 5. The differential stresses exceeding 52 MPa limit marked in red a) after excavation and b) during the disposal period.

## LOAD ON REINFORCEMENT STRUCTURES

The main load on the reinforcement structures comes from the damaged and loosened rock mass. In the elastic modelling approach, the loads imposed on the reinforced structures are considered to result only from damaged rock volume. This can be regarded as very conservative approach, as the crack initiation threshold does not necessarily result in an actual release of rock volume hence resulting in extra load for the reinforcement.

The load from the damaged rock mass on the middle bolt row and shotcrete liner can be conservatively calculated as shown in Table 3 and presented in Figure 6 below. If the damaged rock mass is assumed as a continuous wedge with an apex height of 900 mm and an apex angle of 100°, the load caused by the wedge is 18.3 kN/m using a partial safety factor of 1.35. The design tensile capacity for a Ø25 mm rebar bolt (EN 1.4301 steel) is 85 kN and the design shear capacity for 40 mm of fibre reinforced shotcrete is 64 kN/m. From a strength point of view, both reinforcement types are able to cope with the additional load of stress damaged rock.

Area	Maximum depth of	Load from damaged	Safety factor of rock	Safety factor of fibre
	damage (mm)	rock mass, kN	bolt (Ø25 mm) 1 bolt	reinforced shotcrete
			every 2 m along	(40 mm)
			tunnel	
А	900	18.3	2.3	3.6
В	900	18.3	2.3	3.6
С	850	16.3	2.6	4.0

Table 3. Safety	/ factors of	reinforcement against load f	rom damaged rock.
		5	9



Figure 6. Calculation example for stress-induced damage load on reinforcement. A released wedge is indicated by the red triangle. The coloured area represents the exceeding of the CI threshold.

For more detailed plasticity studies 2D models were also calculated with the actual surveyed tunnel excavation geometry. Unfortunately the modelling software used (Phase<sup>2</sup>) is not capable of modelling Thermo-Mechanical coupling of the stress state involving time dependency on the stress increase. Consequently, the models were calculated ignoring time dependency affecting the long-term evolution of the stress state, i.e., the stress increase was assumed to be increased in an instant. As the time dependency of the thermal stress evolution was ignored, only the highest stresses acting in the rock mass were being accounted for in the models, as shown in Figure 7.



the sprayed concrete

stress of the liner, occurring on the walls, is

Light blue parts of the bolts are considered to yield due to shearing.

compressive stress for the sprayed concrete

The maximum tension stress of the liner, occurring on the walls, is

Light blue parts of the bolts are considered to yield due to shearing.

Figure 7. Peak forces for the sprayed concrete liner for the theoretical tunnel design and actual surveyed tunnel geometry.

The current quantity of the analysed rock reinforcement, Ø 25 mm rockbolts with a length of 3 m and 40 mm layer of sprayed concrete, is strong enough to provide a stable tunnel opening during the peak of the long term stress state, with minor damage. However, the long term stability and safety can be improved through the implementation of the principles of the Observational Method.

Results from all the modelled locations are very similar, if not almost identical. This can be expected because all the access tunnels are similarly orientated and have the same profiles. There are no dramatic differences in the heat induced component of stress at different locations, especially when compared to the magnitude of the stress components before the heating. It can be concluded, however, that the additional heat-induced stress does extend the depth of the potential stress damage considerably. The stress-induced damage can be quite extensive when

considering the lower damage boundary (crack initiation value), reaching up to 900 mm in depth. This said, it must be noted that the damage depths can be considered to be on the conservative side as the crack damage strength is not exceeded at any location, nor is it likely that a fully-released wedge would be formed in the tunnel roof. In all the load cases studied, the currently designed and used reinforcement types and configurations, rock bolts and shotcrete are capable of handling the additional load of the damaged rock.

Thermally induced stresses are function of temperature increase, thermal expansion coefficient of rock and rock mass deformability. It is assumed that temperature increase is well known, but at least minor uncertainties are related to upscaling of thermal expansion coefficient. However, the rock mass deformability is currently under highest uncertainty. Both the temperature increase and deformation modulus have a 1:1 effect on thermal stresses, whereas the effect of thermal expansion is higher. The values used can be considered conservative.

### CONCLUSIONS

During the operation period the temperature increases in the central tunnels have a maximum of 14 °C and in the technical facilities 6 °C without considering the effect of ventilation. The horizontal thermal background stress increase in the central tunnels before backfilling is 8 MPa, and in the technical facilities area less than 2 MPa. The vertical thermal background stress remains about the same as before heating. The effect of ventilation is favourable and neglected in the analysis.

Based on elastic analysis, the stress-induced damage can be quite extensive when considering the lower damage boundary (CI), reaching up to 900 mm in depth. This said, it must be noted that the damage depths can be considered to be very much on the conservative side as the crack damage strength is not exceeded in any location. Also, there is no guarantee that any such cracking would form discrete and detachable rock blocks. Despite deep reaching damage potential, in all the load cases studied the currently designed and used reinforcement types and configurations (rock bolts, shotcrete) are capable of handling the dead weight of the damaged rock should this occur. Up to a limit of 100 °C, no reductions to the material parameters are required.

When bolts, shotcrete lining and rock mass are considered as a combined structure, the rockbolts and the associated grouting can reach the point of yielding but not ultimate failing according to 2D plastic analysis. The shotcrete structure suffers bending tensile failure on the sidewalls. Parts of the bolts will yield through shearing, especially in the crown portion of the tunnel where rock mass damage is concentrated. However, the critical strain limit of 0.05 is not exceeded in the yielded part of the bolts, and they will continue to act as active rock support. The tensile capacity of the sprayed concrete is rather low, before reaching the plastic limit, and failure of the sprayed concrete liner is predicted to occur to some extent on the walls of the central tunnels. However, the fibres within the concrete matrix activate after yielding in the concrete layer has started to occur.

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