Measured Rock Responses in the ONKALO - Underground Rock Characterization Facility

Erik Johansson
Matti Hakala
Topias Siren
Jouni Saari, Marianne Malm

June 2015
Measured Rock Responses in the ONKALO - Underground Rock Characterization Facility

Erik Johansson
Saanio & Riekkola Oy

Matti Hakala
KMS Hakala Oy

Topias Siren
Posiva Oy

Jouni Saari, Marianne Malm
ÅF-Consult Oy

June 2015

Working Reports contain information on work in progress or pending completion.
MEASURED ROCK RESPONSES IN THE ONKALO – UNDERGROUND ROCK CHARACTERIZATION FACILITY

ABSTRACT

Several separate campaigns to measure the rock responses have been conducted at the different locations and different depths in the ONKALO underground rock characterization facility during its excavation works in 2007 - 2014. The rock response measurements have included microseismic and acoustic emission monitoring, rock displacement measurements with convergence pins and extensometers, strain gauge measurements and video camera measurements. The objectives of these campaigns were to better understand the rock properties (deformation-strength properties and in situ stress) and to develop a predictive capability for the design purposes and improve and refine the ability to predict the mechanical conditions ahead of excavation. Most of the rock response measurements have included predictions made beforehand as part of the ONKALO Prediction-Outcome campaign set in the early phase of the ONKALO construction. The predictions have been mostly conducted using advanced computer codes. The measurement campaign was also a learning process to know how the instrumentation is implemented and how they work in the ONKALO rock conditions. The campaigns were co-ordinated so that they have all been conducted at the same general location by the same rock mechanics team. All of the executed rock response measurements have been collected and are summarized in this report.

In summary it can be said that most of rock response tests are considered successful. The rock mass behaviour (response) can be predicted indicating that rock properties and the in situ stress field are understood regarding the natural variability of the Olkiluoto rock mass even with its heterogeneity and anisotropy. Although certain simplifications must be used e.g. in the numerical predictions.

Some technical problems in the instrumentation have been found and that experience will be utilized in the coming test. Some of the instruments are still in the rock to serve the long term monitoring of the ONKALO rock behaviour.

Keywords: Olkiluoto, ONKALO, rock mechanics, rock response, microseismicity, acoustic emission, convergence measurements, extensometer measurements, strain gauge measurements, video camera measurements.
KALLION VASTEEN MITTAUKSET MAANALAISESSA TUTKIMUSTILASSA ONKALOSSA

TIIVISTELMÄ


Mittauksista on myös opittu, miten erilaiset mittalaitteet toimivat ONKALOn olosuhteissa, ja miten niitä tulisi asentaa. Mittausten suunnittelusta, koordinoinnista, ja toteutuksesta on vastannut sama ONKALOn kalliomekaniikkaryhmä. Tähän raporttiin on koottu kaikki suoritettut kallion vastemittaukset ja esitetty yhteenveto niiden tuloksista.

Tulokset osoittavat, että mittaukset ovat pääosin onnistuneet hyvin. Kallion käyttäytymistä on osattu riittävästi ennustaa, mikä kertoo kallion lähtötietojen (kallion ominaisuuudet ja jännitystila) olevan kohdallaan siitäkin huolimatta, että Olkiluodon kivi on heterogeenista ja anisotrooppista. Ennustelaskelmissakin on osin jouduttu tämän takia käyttämään yksinkertaistuksia.

Eräitä teknisiä ongelmia on mittalaitteissa esiintynyt, ja tätä kokemusta tullaan hyödyntämään tulevissa mittauksissa. Osa mittalaitteista on vastemittausten jälkeen jätetty kallioon, ja ne toimivat edelleen palvellen ONKALOn pitkäaikaista monitoreointia.

Avainsanat: Olkiluoto, ONKALO, kalliomekaniikka, kallion vaste, mikroseismiikka, akustinen emissio, konvergenssimittaus, ekstensometrimittaus, venymälialuskamittaus, videokuvaus.
TABLE OF CONTENTS

ABSTRACT

TIIVISTELMÄ

TABLE OF CONTENTS.................................................................................................. 1

1 INTRODUCTION................................................................................................... 3
  1.1 Objectives and Prediction-Outcome work.................................................... 3
  1.2 ONKALO ...................................................................................................... 3

2 MICROSEISMIC AND ACOUSTIC EMISSION MONITORING............................. 5
  2.1 Method ......................................................................................................... 5
  2.2 Shaft section -290 m ... -450 m................................................................. 8
  2.3 POSE pillar experiment at -345 m.............................................................. 10
  2.4 POSE single hole experiment at -345 m (AE)............................................ 14

3 CONVERGENCE MEASUREMENTS ................................................................. 21
  3.1 Method ....................................................................................................... 21
  3.2 Shaft at -180 m ........................................................................................... 23
  3.3 Shaft at -290 m ........................................................................................... 27
  3.4 POSE niche at -345 m ............................................................................... 29

4 EXTENSOMETER MEASUREMENTS ............................................................... 33
  4.1 Method ....................................................................................................... 33
  4.2 POSE niche at -345 m ............................................................................... 33
  4.3 Technical facilities/parking cavern at -437 m ............................................. 36

5 STRAIN GAUGE MEASUREMENTS .................................................................. 41
  5.1 Method ....................................................................................................... 41
  5.2 Experimental holes at the POSE niche at -345 m........................................ 41

6 VIDEO CAMERA MEASUREMENTS................................................................. 47
  6.1 Method ....................................................................................................... 47
  6.2 POSE pillar experiment at -345 m.............................................................. 47

7 SUMMARY AND CONCLUSIONS ...................................................................... 51

8 FUTURE WORK.................................................................................................. 53

REFERENCES ............................................................................................................. 55


1 INTRODUCTION

1.1 Objectives and Prediction-Outcome work

In line with the necessity to develop a predictive capability for design purposes of the rock mass behavior, the Prediction-Outcome campaign, which was conducted during the ONKALO (Posiva Oy’s Underground Characterisation Facility) excavation and in anticipation of the future repository, was developed by the OMTF (Olkiluoto Modelling Task Force). The associated principles are outlined in Andersson et al. (2005).

The Prediction-Outcome (P-O) work is considered to be one of the most important campaigns – because it will progressively improve and refine the ability to predict the mechanical rock mass conditions ahead of excavation. The aim of the P-O work is:

- to enhance confidence in the ability to predict rock conditions in general – and especially for the repository volumes,
- to allow testing and verification of repository design rules (as it will not be possible to make many additional drillholes in the repository volume), and
- to support the construction work and assist in the co-ordination of engineering, investigation and construction.

Four types of predictions have been made during the ONKALO construction:

- the Type A prediction, for a particular increment of chainage ahead of the tunnel face, is made before construction and only uses the latest version of the Site Model and the ONKALO model,
- the Type B prediction is made before the construction of a tunnel section, but should make use of all available data, mapping of previous tunnel sections, etc.,
- the Type B_pilot hole prediction is made before the construction of a tunnel section, and should make use of all available data, mapping of previous tunnel sections etc, but also including pilot hole data, and
- the Type C prediction – or outcome assessment – is made after construction (to see whether the prediction techniques can in fact predict the outcome).

The objective of this rock mechanics report has been to collect all the ONKALO sites where the rock responses have been measured during the ONKALO excavations. Most of the ONKALO sites also included predictions (mostly type B). Outcomes have also been compared to predictions and the results concluded and summarized in the report.

1.2 ONKALO

The ONKALO underground rock characterization facility in Olkiluoto on the west coast of Finland has now been completely excavated to the anticipated repository depth i.e. -430...–450 m. The ONKALO consists of the access tunnel and three shafts (Figure 1-1). The access ramp has been excavated by using drill and blast (D&B) method to a depth level of about -450 m. Three shafts (one personnel shaft Ø4.5 m and two ventilation shafts Ø3.5 m) have been raise bored to the depth of 450 m. Technical facilities are located at the depth of -437 m. In the ONKALO, one of the main focuses of
investigations is currently in the demonstration tunnels at a depth of -420 m, where, for example, technical demonstrations and full scale tests are carried out and the methodology for locating suitable rock volumes is demonstrated.

Figure 1-1. The layout of the ONKALO underground rock characterisation facility.
2 MICROSEISMIC AND ACOUSTIC EMISSION MONITORING

2.1 Method

Since 2002, Posiva Oy has carried out microseismic (MS) monitoring on the island of Olkiluoto. The number of seismic stations has increased gradually and the communication, hardware and software facilities have developed over ten-year period. Currently, Posiva’s permanent seismic network consists of 17 seismic stations and 21 triaxial sensors (Saari & Malm 2014). The purpose of the microseismic monitoring is to improve the understanding of the structure, behaviour and long term stability of the bedrock in the area.

The special arrangement of this microseismic monitoring system was used to monitor the raise boring of the two shafts in the ONKALO, i.e. shaft sections between -290 m and 450 m in the ventilation shaft (⌀3.5 m) and personnel shaft (⌀4.5 m) (Saari & Malm 2015) (see Section 2.2). One of the objectives of these measurements was to observe possible rock damage around the two shaft sections. The network consisted of 16 sensors and their locations are shown in Figure 2-1. The network was designed to monitor very small seismic events, with good location accuracy and within a small rock volume. According to the simulations, the detection threshold around the shafts, at depths between -290 m and -437 m, was about \( M_L = -3.0 \) and the location error ranging 10 to 15 m.
Figure 2-1. Locations of microseismic sensors in the shaft monitoring campaign in the ONKALO (Saari & Malm 2015).
The same special arrangement of this MS monitoring system was used for the POSE pillar in situ experiment (called POSE 1&2 test) at -345 m in the ONKALO (Johansson et al. 2014) (see Section 2.3). The network consisted of one triaxial and three uniaxial 25 kHz high sensitivity accelerometers. The sampling rate of the sensors was 48 kHz. The locations of the sensors (P1 – P4, ONK-PP256...258, ONK-PP272) are shown in Figure 2-2. The objective of these measurements was to observe possible rock damage around two near full scale deposition holes (Ø1.5 m) during the heating phase of the experiment.

The acoustic emission (AE) and ultrasonic monitoring system were used to monitor the POSE single hole experiment (called POSE 3 test) at -345 m (Valli et al. 2014) (see Section 2.4). The system was composed of 24 piezoelectric AE transducers with a resonant frequency of ~50 kHz. The full frequency range of the sensors was in the range of ~35 to 100 kHz (Reyes-Montes et al. 2014). The monitoring system consisted of four 7 m deep drillholes (ONK-PP342…344 and ONK-PP347) around the near full scale deposition hole (Ø1.5 m) (Figure 2-3). Six transducers were mounted in each drillhole. Similar to the above but with better accuracy than the MS measurements, the objective of the AE measurements was to detect possible rock damage around the POSE single hole during the heating phase.

Figure 2-2. Locations of the microseismic sensors (P1–P4) in the POSE pillar experiment (Johansson et al. 2014).
2.2 Shaft section -290 m ... -450 m

Outcome

MS monitoring indicated that only five possible raise boring induced seismic events were recorded in the ONKALO during the shaft raise boring on 17 January – 2 April 2014. This result was expected because even the bigger blasts related to the excavation of the ONKALO have rarely induced seismic events. The events listed in Table 2-1 occurred during the breaks of the raise boring activity. It is also possible and likely that more induced events have occurred during the actual raise boring. However it is impossible to distinguish those among the raise boring signal. The two first events listed in Table 2-1 were clearly separate in time from other raise boring related recordings. It is highly unlikely that these two events were related to moving the raise boring machinery or similar activities. The locations of these events are shown in Figure 2-4.
Table 2-1. Possible induced seismic events during the breaks in raise boring (17 January – 2 April 2014).

<table>
<thead>
<tr>
<th>Date</th>
<th>Time</th>
<th>X</th>
<th>Y</th>
<th>Z</th>
<th>Local magnitude</th>
<th>Location details</th>
</tr>
</thead>
<tbody>
<tr>
<td>22.1.2014</td>
<td>18:05:00 (UTC)</td>
<td>6791972.0</td>
<td>1525913.7</td>
<td>-401.6</td>
<td>-3.5</td>
<td>near the ventilation shaft for incoming air</td>
</tr>
<tr>
<td>18.3.2014</td>
<td>18:19:45 (UTC)</td>
<td>6791992.7</td>
<td>1525918.9</td>
<td>-431.3</td>
<td>-3.4</td>
<td>near the personnel shaft</td>
</tr>
<tr>
<td>14.3.2014</td>
<td>12:45:09 (UTC)</td>
<td>location could not be identified due to too few stations</td>
<td>one event</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>14.3.2014</td>
<td>12:47:49 (UTC)</td>
<td>location could not be identified due to too few stations</td>
<td>two events</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Figure 2-4. Possible induced seismic events during the breaks in shaft raise boring in the ONKALO. View from south. Blue sphere: 22 January 2014 at 18:05:00 (UTC). Red sphere: 18 March 2014 at 18:19:45 (UTC) (Saari & Malm 2015).

Prediction and conclusion

Preliminary rock damage predictions with numerical codes 3D FEM MIDAS/GTS and Examine\textsuperscript{3D} have been made for the shaft area at -437 m by Siren et al. 2011. They concluded that spalling type damage might exist at the bottom of the shafts as shown in Figure 2-5. The predictions seem to fit with the two observations made by the MS-monitoring, although the location accuracy of MS-system is only 10 to 15 m. Later scanning results of the shafts conducted after the raise boring could not indicate any major damage.
Figure 2-5. Predicted spalling/damage (in red) in the crossing area at -437 m level using average rock stress conditions, a=inlet air ventilation shaft and b=personnel shaft (Siren et al. 2011).

2.3 POSE pillar experiment at -345 m

Outcome

MS monitoring recorded some events during the heating phase: 199 events during the heating phase and 114 events during the cooling phase (Johansson et al. 2014). The total number of the located events was then 313, which on average was 4.2 events per day during the heating and cooling phases. According to the simulations, the detection threshold inside the POSE microseismic subnetwork was from $M_L = -5.0$ to $M_L = -5.5$. The observed magnitudes varied between $M_L = -2.3$…-6.1. Presumably, there were other smaller events.

The first peak of events (12) recorded on the third day of the heating was correlated with the first damage observed in the camera images. Two weeks later there was the next peak of events, which seems to correlate with the time when the change was noticed in the strain gauges and also further damage in the video monitoring. The maximum number of events per day was 20, which seems to correlate with the time when further damage was noticed by the video monitoring. A few other peaks in number of events per day can be recognised, but they do not seem to have any special correlation with the temperatures.

Events were found around the holes but the majority of the events were located on the N and S sides of the experimental holes and in the pillar between the holes—confirming the visual observations (Figure 2-6). In terms of the vertical direction, the events were located in the middle and upper levels of the experimental holes. The first events related to the heating experiment were mostly located in the top part of the pillar and later towards the central and bottom parts. The events during the cooling phase were mostly located in the top parts. It should be noted that the location accuracy is about 1 m, which is not good enough for more specific analysis.
Figure 2-6. MS-events (313) during the heating experiment of the POSE pillar test. The size of the spheres is indicative of the magnitude. Blue events were located with three triggers and red events with four triggers. View towards north (top left diagram) and east (top right) and from above (bottom) (Johansson et al. 2014).

Prediction

Before the test execution two types of predictions were made to estimate the damage around the holes (Johansson et al. 2014). One was made using continuum thermo-mechanical analyses with the 3DEC code (Hakala & Valli 2014) and the other using the fracture mechanics approach with Fracod2D (Siren 2011). For the 3DEC analysis the rock mass was assumed to be continuous, homogeneous, isotropic and linearly elastic with the intact rock parameter values based on the mean values defined for gneissic rock types (Posiva 2013) and on preliminary biaxial test results related to in situ stress measurements in the former excavation damage zone (EDZ) niche and at the access tunnel chainage. The in situ stress, which was applied in the prediction simulations, was based on two LVDT cell stress measurements that were conducted in the access tunnel at chainage 3620 m and in the
EDZ-niche. The combined preliminary stress measurement results indicated a trend of 336° and a magnitude of 25.7 MPa for the maximum principal stress (Figure 2-7). Hence the highest stress concentrations should form on the pillar side walls.

Both numerical simulations estimated damage at the pillar walls with a maximum depth of around 80 mm (3DEC) or 110 mm (Fracod, anisotropic model) already after drilling the second hole (Figure 2-8). After two weeks heating, the stresses were estimated to be increased in the pillar area to the level of the laboratory peak strength and further damage to take place around the holes.

Figure 2-7. The trend of the maximum principal stress according to stress measurements from the ONKALO access tunnel and the EDZ niche (Hakala & Valli 2014).
Figure 2-8. Predictions for the rock damage in the POSE pillar experiment; 3DEC analysis (top) showing the rock damage in red and Fracod2D analysis after hole drilling (bottom) (Johansson et al. 2014).

Conclusion

No damage was observed during the hole boring but clear damage occurred due to the heating. The damage was not however located on the pillar side (Figure 2-9) (Johansson et al. 2014). The major damage was localized and controlled by the foliation and rock type contacts, which were known to be weak. Most visual damage seemed to occur on the S- and N-side walls, which was quite consistent with the results of the MS-monitoring. This would indicate rather E-W stress direction than SE-NW direction.
Although, as noted earlier, the MS-monitoring was not accurate enough to exactly localize the damage.

![Observed damage in the POSE pillar experiment, isometric view from north-northeast (Johansson et al 2014).](image)

**Figure 2-9.** Observed damage in the POSE pillar experiment, isometric view from north-northeast (Johansson et al 2014).

### 2.4 POSE single hole experiment at -345 m (AE)

**Outcome**

A total of 609 AE events were located during the monitoring period. Lowest observed magnitudes around $M_L=-2.0$ took place at early stages of heating and were interpreted as corresponding to the induction of small-sized thermal induced cracks (Reyes-Montes et al. 2014). Peak activity (ranging $M_L=-1.5...-1.25$) was recorded during the heating phase of the experiment and was extending to the end of the heating period. Mean location magnitudes increased during the monitoring period (Valli et al. 2014). The relatively larger magnitude events observed were interpreted as associated coalescence and development of microcracks. The processing method and parameters used to locate AEs are described in detail in Reyes-Montes et al. (2014).
The total 609 AE events produced robust locations around the experimental hole. The highest concentration seemed to follow the band of migmatitic gneiss observed in the experiment hole. The locations of these events are shown in Figure 2-10.

Figure 2-10. Located acoustic emission (AE) events during the monitoring period along the POSE single hole experiment (Valli et al. 2014).
Changes in the seismic velocities closely followed the evolution of the temperature profile in the hole wall. An increase in both P- and S-wave velocities was observed at all depth levels and surveyed ray paths during the heating phase. The highest changes were observed in the ray paths skimming the hole surface at depths between 2.3 and 3.7 metres. This observation indicated the closure of the \textit{in situ} stress and excavation-induced micro cracks due to the stress increase (‘thermal stress’). After turning the heaters off, P-wave velocities showed a decrease, reaching values below those measured at the start of the monitoring of the experiment. This happened approximately four weeks after switching off the heaters (Valli \textit{et al.} 2014).

\textbf{Prediction}

Before the test execution, two types of predictions were carried out to estimate the damage and related AE events around the single hole (Valli \textit{et al.} 2014). One was made using continuum thermo-mechanical analysis with 3DEC code (Hakala & Valli 2013) and the other using the fracture mechanics approach with Fracod2D. The same input values for rock were used as in the POSE pillar model described in Section 2.3.

3DEC simulations indicated that, after 12 weeks of heating, the maximum principal stress around the hole will reach approximately 100 MPa and that the extent of damage will be limited to an approximate maximum distance of 200 mm into the rock (Figure 2-11). The damage extent after three weeks of heating would be less than 100 mm. Fracod2D predictions indicated, using two slightly different stress interpretations that the majority of the damage will be subsurface and the maximum depth is about 130–140 mm (Figure 2-12). The predicted acoustic emission (AE) patterns are presented in Figure 2-13 and they follow the development of fractures and indicate no clear concentration of AE events around the experiment hole.

\textit{Figure 2-11}. Predictions for the rock damage in the POSE single hole experiment; 3DEC analysis showing the differential stress required to cause damage/spalling, crack initiation value of pegmatite granite is 57 MPa (Hakala & Valli 2013).
Figure 2-12. Predictions for the rock damage/fracture propagation in the POSE single hole experiment; fracture mechanics analyses during a 12 week heating period and assuming the stress interpretation based on the stress measurements in the POSE hole ONK-EH3 (Valli et al. 2014).
Figure 2-13. Predicted cumulative acoustic emission patterns after the experiment for the EDZ & VT1 stress interpretation (left) and for the ONK-EH3 stress interpretation (right) (Valli et al. 2014).

Conclusions

The damage observed in the POSE single hole after the experiment was minimal (Valli et al. 2014). Open fractures were observed only near the contact between schistose mica gneiss and the pegmatitic granite. All of the other fractures that were seen to develop as a result of heating and thus due to the increase in the *in situ* stress state were closed. A total of 17 induced fractures was mapped from the hole wall with undulating and rough profiles with no alteration evident (Figure 2-14). The global weighted mean trend of the fractures was 172°/34°, fairly close to the orientation and dip of the foliation. It is also worth noting that the fractures observed in the upper part of the hole are mostly horizontal, whilst fractures in the lower part of the hole are near vertical.

The predicted rock damage and AE-events seemed to happen around the single hole with no clear concentration, which was more or less the case in the visual observations, although the highest concentration seemed to follow the band of migmatitic gneiss.
Figure 2-14. Observed damage in the POSE single hole experiment, viewpoints outside the hole (Valli et al. 2014).
3 CONVERGENCE MEASUREMENTS

3.1 Method

The convergence measurement is a method where the distance between the measuring pins or bolts installed at the excavation boundary is measured manually with a special measuring tape. In the ONKALO measurements, the type of the convergence instrument was the Distometer by Solexperts (Lahti & Hakala 2010) (Figure 3-1).

Optical deformation sensors (SOFO) were used around the shaft location at -180 m, but the results were not satisfactory and included too many measurement errors to produce any rock response estimation.

Convergence measurements have been performed in six locations around the shafts at the levels -180 m and -290 m and in one cross-section in the POSE niche at -345 m (Figure 3-2).

![Figure 3-1. Convergence measurement device Distometer used in the ONKALO.](image)
Figure 3-2. Location of the convergence measurements in the ONKALO; shaft measurements shown as circles 1-6 (top) and POSE niche shown as red dashed line at -345 m (bottom).
3.2 Shaft at -180 m

Outcome

In convergence measurements at -180 m, 24 (two shafts) or 12 pins (shaft KU2) short bolts, were installed around the designed shaft perimeter so that the horizontal distance from pin to shaft perimeter was 25 cm (Figure 3-3). The anchoring length of the pins was approximately 15–20 cm. Around each shaft, a total of 24 diametric and 48 secant lines were measured. The objective of the measurements was to measure immediate shaft excavation (raise boring) responses. Each measurement was replicated at least twice, in order to achieve better repeatability than 0.3 mm. The measurement was made three times, once before raise boring and twice after raise boring.

The measured convergences around the shafts are shown in Figure 3-4. The maximum convergences at -180 m were between 0.25 and 2.2 mm. The ONKALO shaft KU3 showed low convergence values (0.25 mm), but these measurements had also the best repeatability. In other shafts, the measurement repeatability was quite poor, especially, after the raise boring (see Figure 3-4).

Figure 3-3. Layout of ONKALO shaft convergence measurements with 24 pins and one example of a measurement triangle (Lahti & Hakala 2010).
Figure 3-4. Measured convergences around the shafts at -180 m. Red arrows show the direction of the maximum convergence, and the estimated direction of the maximum in situ rock stress.
**Prediction**

2D- and 3D-simulations were performed to predict the displacements induced by shaft raise boring at -180 m. The 3D-model consisted of five excavation phases, as shown in Figure 3-5. It was assumed that the major principal (horizontal) *in situ* stress was oriented east-west and the ratio of the horizontal principal stresses, $\sigma_H/\sigma_h$, was \( \sim 1.8 \).

![Figure 3-5. 3D-model and excavation phases to predict displacements caused by shaft raise boring.](image)

The results indicate that the maximum displacement at 15 cm out from the shaft perimeter was around 0.8 mm (convergence 1.6 mm) in the 3D-model, excluding the effect of the shaft drift excavation, (Figure 3-6), and around 1.0 mm (convergence 2.0 mm) in the 2D-model (Figure 3-7); whereas, the maximum measured convergence was 2.2 mm (see Figure 3-4 for comparison). Note that excavation of the shaft drift will slightly change the orientation of the maximum rock stress and hence the orientation of the maximum convergence. Analyses showed also that the foliation may also change the orientation of the convergence.
Figure 3-6. Predicted displacements caused by shaft raise boring at 180 m, 3D-model.

Figure 3-7. Predicted displacements caused by shaft raise boring at 180 m, 2D-model.
Conclusions

The measured maximum convergences of 2.2 mm seemed to correspond quite well with the predicted ones of 1.6-2.0 mm. However, during the measurements it was found that the measuring device, the Distometer, was not suitable for tunnel conditions with unclean dripping water (Lahti & Hakala 2010). Normally the device worked well in dry conditions before the shaft raise boring, but it became stuck or gave unreliable values after shaft raise boring when there was dripping water. In the majority of the measurements, it was impossible to achieve the preset measurement repeatability of 0.3 mm. It was also found that the device can produce an unexplained variance of almost a millimeter. As the expected raise boring induced convergences should be less than 2 mm, the method includes too much uncertainty for reliable rock response results to be obtained.

3.3 Shaft at -290 m

Outcome

The measuring layout was similar as that, at -180 m except that all three shafts had 24 pins. Readings were also taken as at the -180 m level.

The measured convergences around the shafts are show in Figure 3-8. The maximum convergences at 290 m were 0.25–0.8 mm. As at 180 m, the ONKALO shaft KU3 showed the same, low convergence values (0.25 mm), and these measurements had again the best repeatability. In other shafts, the measurement repeatability was again quite poor, especially after the raise boring (see Figure 3-8).

Prediction

Predicted displacements for the -290 m shafts are similar as for the -180 m level since no major stress changes are expected over that depth range. The new LVDT stress measurements have indicated that there are two stress regimes i.e. above and below the major fracture zone BFZ20, green BFZ099 zone in Figure 3-2 (Posiva 2013). The shaft measurements are above BFZ20 where the stress components are close to each other and the stress field is not fully clear. It is estimated that above BFZ20 $\sigma_{H}=16.3$ MPa, $\sigma_{H}=15.4$ MPa and $\sigma_{V}=14.8$ MPa. This would give approximately 1 mm displacements (convergence 2 mm) around the shaft (Figure 3-9).
**Figure 3-8.** Measured convergences around the shafts at -290 m. The red arrows show the direction of maximum convergence.

**KU1 Level: -280 m**
- Measured max: **-0.7 mm**
- Repeatability better than 0.3 mm: before 17/36 and after 8/36

**KU2 Level: -280 m**
- Measured max: **-0.8 mm**
- Repeatability better than 0.3 mm: before 36/36 and after 0/36

**KU3 Level: -280 m**
- Measured max: **-0.25 mm**
- Repeatability better than 0.3 mm: before 35/36 and after 35/38
Figure 3-9. Predicted displacements caused by shaft raise boring assuming an almost isotropic stress state, 2D-model.

Conclusion

The measured maximum convergences were only less than 1 mm which was about half of the expected values. The reason for this is probably due to the problems in the measuring device mentioned in the -180 m shaft measurements above.

3.4 POSE niche at -345 m

Outcome

One cross-section in the POSE niche was chosen for convergence measurements with the objective of measuring the excavation induced rock responses (deformations) when the 4.5 m wide and 5.0 m high EDZ-niche was reshaped to be the 9 m wide and 7 m high POSE-niche (Lahti & Siren 2011). Six convergence bolts were installed 0.5 m from the tunnel head, two on both walls and two in the roof (Figure 3-10).

The convergence bolts were placed in the bottom of short holes (diameter 127 mm) in order to protect the bolt heads during the advancing blasting. Convergences were measured manually with Posiva’s Distometer at least twice after installation, or as many times as necessary so that repeatability better than 0.2 mm was achieved. Further measurements were performed after every top heading blast which were closer than 20 m from the measurement section and after every second bench cut. At each time, 12 lines were measured with repeatability better than 0.2 mm. The repeatability of manual convergence measurement was better than 0.14 mm, i.e. below 0.2 mm (Lahti & Siren 2011).
Figure 3-10. Convergence measurement pins (CB1-CB6) in the POSE niche at -345 m (Lahti & Siren 2011).

The results from the measurements are shown in Figure 3-11. The horizontal lines (red in Figure 3-10), i.e. change in the span of the POSE niche, indicated negative values (shortening) but others positive values (lengthening) (Figure 3-11).

Figure 3-11. Results from convergence measurements in the POSE niche at -345 m (Lahti & Siren 2011).
Prediction

Prediction calculations for the POSE niche convergence measurements were made using the 3DEC code and the average ONKALO rock properties, as described in Posiva 2013, and rock stress data based on two LVDT cell stress measurements that were conducted in the access tunnel at chainage 3620 m and in the former excavation damage zone (EDZ) niche (Hakala & Valli 2014).

The predictions indicated a total horizontal and west-side inclined convergence of about 0.9 mm and the inclined convergence on the eastern side is approximately half of those observed on the opposite side (Figure 3-12). Comparing these results to the total displacements caused by the POSE excavations, it can be said that convergence bolts installed close to the end of the round can detect about 45% of the total displacements (Hakala & Valli 2014).

![Figure 3-12. Prediction for convergence measurements in the POSE niche at -345 m (Hakala & Valli 2014).](image-url)
Conclusion

The rock mass response could be measured in spite of the fact that they were made close to the blasting. The prediction corresponded well with the measured values of the niche span (wall pins, horizontally), but the measured values between the roof pins and wall pins gave positive values instead of negative values (Figure 3-13). The reason for this is unknown but this may indicate some strange behaviour in the roof pins (e.g. due to the blasting).

Figure 3-13. Comparison between predicted convergence values (solid lines) and measured values (dashed lines) in the POSE niche at -345 m.
4 EXTENSOMETER MEASUREMENTS

4.1 Method

Extensometer measurement is a borehole method where relative displacements between several points at different depths in the boreholes are measured, typically using an automatic datalogger reading or manual reading.

4.2 POSE niche at -345 m

Outcome

Two horizontal extensometers were installed in the west side of the niche so that the horizontal angle between the niche axis and the extensometer hole was about 60 degrees (Figure 4-1) (Lahti & Siren 2011). One of the extensometers was optical with very high resolution (SOFO, Smartec SA), the other one was a conventional three-point rod extensometer (MPBX, Interfels) with an electrical reading unit. The rod extensometer in drillhole ONK-PP225 (trend 59.6°, plunge +11.5° and length 34 m) had three anchors at the depths of 14.8 m, 26.2 m and 29.7 m. The last anchor was about 0.5 m from the POSE-niche wall. Both extensometers were fully grouted into the drillhole. Unfortunately, the optical extensometer was permanently damaged during the installation. After installation, the rod extensometer was read continuously after every 30 min and the data were automatically stored in a datalogger.

Figure 4-1. Extensometers to measure rock responses of the POSE niche excavation at -345 m (Lahti & Siren 2011).
The results from the extensometer measurements are shown in Figure 4-2. It seemed that two weeks hardening and settling period before the start of the POSE excavation wasn’t long enough to obtain stable initial readings. During the excavation, readings were reasonable and measured the rock response. After excavation, the readings stabilized as expected but after one to two months later, drifting in the results was initiated. The reason for the fluctuations is unsolved, but possible sources are temperature changes, moisture problems in the measurement system or anchor grouting problems. Later, temperature measurements in the extensometer anchor locations became unstable which suggested moisture problems.

**Prediction**

Predictions for the POSE niche extensometer measurements were conducted using the same 3DEC code as for the POSE convergence measurements (Hakala & Valli 2014). Extensometer predictions indicated that, upon initial expansion of the POSE niche (Full round, -1, as shown in Figure 4-3), the rock mass closest to the excavation perimeter experiences negative displacements (i.e. extensometer shortening). This is due to displacements observed towards the excavated space and the alignment of the extensometer, after which further excavations naturally cause lengthening due to displacements trending towards the expanded POSE niche (Figure 4-3).

![Figure 4-2. Results from the rod extensometer measurements in the POSE niche at -345 m (Lahti & Siren 2011).](image-url)
Conclusion

The rock mass response could be measured and the readings were reasonable in timing and order, but the magnitudes of the measured values were in general about half of the predicted values (Figure 4-4). The reason for this is unclear, but it may indicate the rock mass being stiffer than anticipated.
4.3 Technical facilities/parking cavern at -437 m

Outcome

Two rod extensometers were installed in the technical rooms of the ONKALO at a depth of -437 m in order to measure the rock response due to the excavation of the larger (parking) cavern (Figure 4-5). The extensometers were three point devices with anchor depths of 7.0 m, 12.8 m and 13.5 m. The last anchor was about 0.5 m from the wall of the large cavern. Both extensometers were fully grouted into the drillhole. During excavation, readings were continuously taken, first after every minute and later once per hour, by a datalogger.
Figure 4-5. Extensometers to measure rock responses of the large cavern excavation at -437 m (Alterio & Siren 2013).

The results from the extensometer measurements are shown in Figure 4-6. Both extensometers showed around 2 mm (max. 2.2 mm) displacement response (anchor points 12.8 m and 13.5 m) due to the excavations. The displacements at 7.0 m were around 0.1 mm. After completion of the excavations, the values stabilized and no significant time dependent behaviour was noticed.

Figure 4-6. Results from the rod extensometer measurements in the technical facilities at -437 m (Alterio & Siren 2013).
Prediction

Predictions for the extensometer measurements at -437 m were calculated using the 2D-model (Phase2 code) (Figure 4-7). The predictions used two different stress regime assumptions (E–W and N–S stress orientations) and showed that the maximum displacement was expected to be around 1.3 mm for the east-west stress direction and 1.9 mm for the north-south stress direction. For the anchor in the middle of the pillar at the depth of 7.0 m, the predicted displacement was about 0.2 mm.

Later a more detailed 2D-model utilizing laser scanned profiles from the excavated tunnels and drillhole measurements was created to recalculate the displacements (Figure 4-8). Two stress regimes (orientation) were also used in these simulations. They, as expected, gave slightly smaller displacements, as shown in Table 4-1, than the pre-excavation predictions.

![Figure 4-7. Prediction 2D-model for extensometer measurements in the technical facilities at -437 m (E–W stress orientation).](image)

![Table 4-1. Calculated displacements in mm for the extensometers at -437 m using the actual tunnel geometry (see Figure 35) (Alterio & Siren 2013).](table)

<table>
<thead>
<tr>
<th>Calculated displacement</th>
<th>East-west stress direction</th>
<th>North-south stress direction</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>EW, 7.0 m</td>
<td>EW, 12.8 m</td>
</tr>
<tr>
<td>E1 (eastward) (mm)</td>
<td>0.2</td>
<td>0.8</td>
</tr>
<tr>
<td>E2 (westward) (mm)</td>
<td>0.2</td>
<td>0.6</td>
</tr>
</tbody>
</table>

Input data: \(\sigma_H = 25.1\) MPa, \(\sigma_h = 13.3\) MPa, \(\sigma_V = 11.4\) MPa
Young’s modulus 55 GPa, Poisson’s ratio 0.25
Conclusion

The outcome shows that the north-south stress direction predictions (1.3–1.7 mm) are closer to the measured results (around 2 mm) for the anchors at 12.8 m and 13.5 m depths. However, the measured results from the anchors at 7.0 m depth seem to be the same (0.1 mm vs. 0.2 mm) as in the calculations for the east-west stress direction. Although there are large percentage differences, it is demonstrated that the observations are of the same order as the calculations and the rock mass responses are as expected. The calculations also depend heavily, for example, on the stress magnitudes, rock mass Young's modulus and exact measurement points—which contribute to uncertainties in the predictions.

Figure 4-8. Detailed 2D-model for extensometer measurements in the technical facilities at -437 m (Alterio & Siren 2013).
5 STRAIN GAUGE MEASUREMENTS

5.1 Method

Strain monitoring is based on gluing strain gauge rosettes on the rock walls and monitoring the changes during excavation. The rosettes are typically of the $0/45/90^\circ$ type consisting of three individual strain gauges located one on top of the other.

5.2 Experimental holes at the POSE niche at -345 m

Outcome

In the POSE pillar experiment, strain monitoring was carried out by gluing eight strain gauge rosettes, R1–R8, in the middle of the pillar on the wall of ONK-EH1, as shown in Figure 5-1. The rosettes were of the $0/45/90^\circ$ type with gauge resistance of 120 ohms and they were protected against moisture with a triple layer sealing system. The objective of this monitoring was to measure the rock responses (strains or microstrains i.e. strain expressed in terms of parts per million) due to the boring of the hole ONK-EH2 plus the heating.

Figure 5-1. Location of the strain rosettes in the POSE hole ONK-EH1 (Johansson et al. 2014).
All the strain gauges worked well and responded to the boring. The maximum recorded strain changes during the boring were in the tangential gauges Sx00 (-160…-281 microstrains) and the lowest values in the axial gauges Sx90. The maximum value from the start of boring the pilot hole to the end of the boring of ONK-EH2 was -281 microstrains in the rosette R2 and the minimum in the rosette R7. The strain value responses in the rosette R2 during the boring are shown in Figure 5-2. The heating phase also responded well and caused around 1000 microstrains responses at the maximum (Figure 5-5). Only the rosette R4 behaved differently, indicating possible detachment from the rock surface.

Similarly, in the POSE single hole experiment, strain gauges (tangential, inclined and vertical gauges) were also installed on the wall of ONK-EH3 as either rosettes or single tangential strain gauges. Unfortunately, strain measurements indicated an unrealistically dramatic change during the early phase of heating. This was believed either to be due to moisture during installation or failure of the adhesives used when gluing the gauges, Thus they failed to function properly and did not produce any reasonable results (Valli et al. 2014).

![Figure 5-2. An example of the strain gauge results during the hole boring, rosette R2. Number 1 refers to when the boring of pilot hole of ONK-EH2 starts; number 2 when the final pilot hole depth has been reached; number 3 when reaming of ONK-EH2 starts; and number 4 when boring of ONK-EH2 has finished (Johansson et al. 2014).]
**Prediction**

The same model as for the damage prediction (see Section 2.3) was used to predict strains for the POSE pillar experiment. They were largely obtained from the elastic stress evolutions observed on the hole surfaces and were calculated in the cylinder coordinate system of each hole and in the direction of each strain gauge.

Tangential strains were largely predicted to be over 250 microstrains by the end of the boring (Figure 5-3). During the heating, predictions indicated that the minimum amount of strain would be observed close to the top of the hole, whereas maximums strains (>1000 microstrains after one month heating) in the tangential strain gauge would occur near the mid-point of the pillar (Figure 5-4).

**Figure 5-3.** Strain predictions for rosette R5 after the hole boring from a depth of 3.24 m, tangential gauge strains in red, inclined in green and axial in blue (Hakala & Valli 2014).
Figure 5-4. Strain predictions for rosette R5 during the heating phase from a depth of 3.47 m, tangential gauge strains in red, inclined in green and axial in blue.

Outcome

Most of the strain measurements indicated clear responses and the measured values, especially the tangential strains, seemed to follow the predicted elastic responses well, as seen in Figure 5-5. All the strain gauge values at the mid-hole depth (5.2 m) seemed to match best including the axial strain values. A sudden change was noticed in almost all the strain gauges after around two weeks heating. At the same time, the first rock damage was also noticed in the video monitoring (see Section 6.2).
Figure 5-5. Strain readings at four depth levels, i.e. 0.75 m, 2.0 m, 3.2 m and 5.2 m. The continuous lines are the measured values and the individual symbols are the predicted values (Johansson et al. 2014).
6 VIDEO CAMERA MEASUREMENTS

6.1 Method

The video monitoring system used in the ONKALO for the POSE experiment was developed as part of Master's thesis work at Tampere University of Technology. The method is described in Kumpumäki 2011.

6.2 POSE pillar experiment at -345 m

Outcome

In the POSE pillar experiment, the video monitoring system was arranged to cover a 6 m high area in a 120° sector on the opposite wall to the pillar wall of the hole ONK-EH1 (Johansson et al. 2014) (Figure 6-1). The system consisted of lights, five cameras (1-5) and USB hubs, and it was assembled as a package that was lowered as a unit into the hole. The frame of the package was an aluminium truss which was fixed to the bottom and sidewall of the POSE hole ONK-EH1. The objective of this method was to visually monitor and in real time the possible rock damage on the pillar wall during the boring and the heating phase of the POSE experiment.

Figure 6-1. Location of five video cameras (camera 1 is the uppermost) in the POSE hole ONK-EH1 (Johansson et al. 2014).
Video images were recorded every 60 seconds during the execution periods and once per hour during the break before the heating phase. The imaging system operated during the experiment as planned. The recording was interrupted only once due to several days blackout of the electricity supply.

No damage was observed during the boring phase. During the heating phase some minor damage, fracture forming and fracture shearing/opening was observed. No surface type damage was seen. The video monitoring indicated the first damage occurrence was already in the early phase of heating, i.e., about after a few days of heating. Damage was then further developed at the mid-hole depth, both in the gneiss and in the pegmatite-granite (Figure 6-2). At the same time, the strain gauges also responded (see Figure 5-5). Some other damage, mainly in the upper section of the hole, was also recorded. No significant changes were noticed after that on the pillar sidewall.

Figure 6-2. Rock damage (fracturing) in the pillar wall of the ONK-EH1 hole due to the heating and recorded by video monitoring at the hole depth 3–4.5 m (camera 3) (Johansson et al. 2014).
Prediction

The same model as described in Section 2.3 was used to predict rock damage for the POSE pillar experiment.

Conclusions

Video monitoring could not enable detection of any surface damage as predicted, but some minor fracture shearing/opening type damage could be seen on the pillar wall during the heating. This is partly due to the fact that major damage took place outside the camera coverage area (Johansson et al. 2014). Locations of the observed damage were due, not only because of the rock heterogeneity, but also because the major stress orientation was (after the testing) found to be different to that assumed in the predictions. Back analyses (type C prediction) will be performed later to see whether they can in fact predict the outcome.

Some observed fracture damage seemed though to correlate quite well with the observed strain gauges changes.


7 SUMMARY AND CONCLUSIONS

All the rock response measurements conducted in the ONKALO during 2009 – 2014 have been summarized in Table 7-1.

**Table 7-1.** Rock response measurements conducted in the ONKALO during 2009 – 2014.

<table>
<thead>
<tr>
<th>Test site/location</th>
<th>Methods</th>
<th>Prediction Outcome</th>
<th>Prediction</th>
<th>Outcome</th>
<th>P/O-comparison and comments</th>
</tr>
</thead>
</table>
| Ventilation and personnel shafts -290…-450 m | Numerical codes: MIDAS/ GTS and Examine
| Microseismic (MS) measurements and laser scanning | Stress-induced damage at bottom level of shafts | Two seismic events at bottom level of shafts (-401 & -431 m) | Quite good - recorded events in general at right locations, location accuracy of MS-system was 10–15 m |
| POSE-niche, pillar experiment -345 m | Numerical codes: 3DEC and Fracod2D | MS-measurements, visual mapping | Damage mainly on pillar side walls | 313 events, around holes, mostly on N and S sides of holes | Fair - majority of events not at locations as predicted with numerical codes. |
| POSE-niche, single hole experiment -345 m | Numerical codes: 3DEC and Fracod2D | AE-measurements, visual mapping | Damage at hole walls, AE-events around hole | 609 events, around hole | Quite good – predicted damage and AE-events around hole as observed. |
| Shafts -180 m | Numerical codes: Examine and 3D | Convergence measurements | Maximum convergence -1.6…-2.0 mm | Maximum convergence -2.2 mm | Good - regarding measurement accuracy (±0.3 mm) |
| Shafts -290 m | Numerical codes: Examine and 3D | Convergence measurements | Maximum convergence -1.6…-2.0 mm | Maximum convergence -0.25…-0.8 mm | Fair - measurement device didn’t work as expected |
| POSE-niche, -345 m | Numerical code: 3DEC | Convergence measurements | Wall-to-wall convergence -0.7…-0.8 mm, wall-to-roof -0.3…0.9 mm | Wall-to-wall convergence -0.9 mm, wall-to-roof 0.25…1.0 mm | Good - (wall-to-wall) Poor - (wall-to-roof) |
| POSE-niche, -345 m | Numerical code: 3DEC | Extensometer measurements | Rock displacement 0.1–1.0 mm | Rock displacement 0.05–0.45 mm | Fair-good – displacements half of the predicted |
| Technical facilities, -437 m | Numerical codes: 3DEC, Examine | Extensometer measurements | Rock displacement 0.2–1.9 mm | Rock displacement 0.1–2.2 mm | Good – rock response as expected |
| POSE, pillar experiment -345 m | Numerical code: 3DEC | Strain gauge measurements | Rock strains 100…1200 µstrains | Rock strains -200…-1000 µstrains | Quite good – rock response as expected |
| POSE, pillar experiment -345 m | Numerical code: 3DEC and Fracod2D | Video monitoring | Damage/ spalling on pillar side wall | Only minor damage was observed | Fair – major damage took place outside the camera coverage area, but corresponded quite well with strain gauge changes |
Several separate campaigns to measure the rock responses have been conducted in the ONKALO during the excavation works. The objective of these campaigns has been to develop a predictive capability for the design purposes and improve and refine the ability to predict the mechanical conditions ahead of excavation. Most of the rock response measurements campaigns have included predictions made beforehand being part of the ONKALO Prediction-Outcome campaign set in the early phase of the ONKALO construction.

Predictions performed here have required to use the rock properties currently available e.g. from the Site Reports. Rock response measurements have hence tested the current rock properties and how well they are understood. Many of the tests described in this report could be considered successful in the sense that the rock mass behaviour (response) can be predicted indicating that rock properties and the *in situ* stress field are also well understood. There are naturally challenges to predict rock material which is heterogeneous and anisotropic. Certain simplifications must then have been used e.g. in the numerical simulations.

This has also been a learning process to know how instrumentation is implemented and how they have worked in the ONKALO rock conditions. Although some problems have been encountered the nature and behaviour of the rock mass is now understood within these contexts, and this report indicates that it is, subject to the variability naturally introduced by the fractures, BDZs, the foliated intact rock and the stress field. Some of the instrumentations are also still left in the rock mass to serve the long term monitoring of the ONKALO rock behaviour.

The campaigns have been co-ordinated in the sense that it has all been conducted at the same overall location, by the same overall team, and using advanced computer simulations.
8 FUTURE WORK

Some type C predictions i.e. (predictions after construction or test execution), especially related to the POSE experiment are still to come, which may change some of the conclusions drawn in this report.

More rock response exercises are probably performed later in the ONKALO. One ongoing test is so-called POPLU (plug test) where a section of the demonstration tunnel at -420 m will be pressurized with water and rock response will be measured with two three-point extensometers. The analyses of results will be published in the coming Posiva reports.
REFERENCES


