Analyses of tunnel stress failures at Pyhäsmi mine

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Working Reports contain information on work in progress or pending completion.

The conclusions and viewpoints presented in the report are those of author(s) and do not necessarily coincide with those of Posiva.
Analyses of Tunnel Stress Failures at Pyhäsalmi Mine

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ANALYSES OF TUNNEL STRESS FAILURES AT PYHÄSALMI MINE

ABSTRACT

The aim of this project was to develop a method to map and analyse stress failure observations, understand factors affecting stress failures, and study criteria to predict stress-induced failures around excavations. Mapping took place at Outokumpu Mining Oy Pyhäsalmi mine at a great depth between +1000 - +1400 m. The Pyhäsalmi mine was selected as a test site, because its geology and stress-strength ratio are similar to Posiva Oy’s main investigation site for an underground nuclear waste repository at Olkiluoto. Data was collected from newly excavated tunnels, 1700 m in total, by using the pre-developed mapping system.

The main rock type at the mapped area is very hard volcanite with an average uniaxial compressive strength of 220 MPa. Actually, that includes both felsic and mafic types, but because of the very dense layering and lack of detailed rock mechanical properties, they are handled as one type of rock. The quality of volcanite rock mass is good or very good, GSI – value being 77. Maximum principal stress is 65 MPa to 75 MPa, and the maximum elastic secondary stress around the tunnels is about 150 MPa and around the shafts 135 MPa.

The majority of stress failure observations were classes of none (J0), 34 %, noise only (J1), 36 %, or, light spalling (J3), 26 %. Based on the analyses, it was found that the most important parameters inducing stress failures were the state of stress around the tunnel and the orientation of the rock anisotropy. However, the rock type and its strength, the shape of the tunnels, time after excavation, and the geological anomalies also have a correlation with the stress failures.

The observed stress failures were compared with the failure predictions based on the elastic 3D secondary stresses and three statistical yield criteria. The best estimations were found when using Read and Hoek-Brown yielding criteria. Also, the criteria of the peak strength were used, but were found to overestimate the in situ strength of the rock. However, the effect of stress anisotropy was found to be significant.

Keywords: stress failure, mapping, tunnel, shaft, in situ stress, rock strength, failure analyses
TIIVISTELMÄ


Kartoituksessa yhrättiin jännitystilavaurioja ja -kartoituksena emäksiä (JO), 34%, ei murtumia, mutta kallioon (J1), 36%, tai hilseilyä (J3), 26%. Keskeiset jännitystilavauriotekijät olivat kalliossa vallitseva in-situ jännitystila ja kiven suuntautuneisuus, mutta myös kivilajia ja sen Lujuus, louhitten tilan muoto, louhinnan ja kartoituksen välineen aika ja geologiset anomaliat korreloivat jännitystilan aiheuttamien murtumien kanssa.


Avainsanat: jännitysmurtuma, kartoitus, tunneli, kuilu, jännitystila, kiven Lujuus, murtotutkimus, 3D simulointi.
ANALYSES OF TUNNEL STRESS FAILURES AT PYHÄSALMI MINE

ABSTRACT

TIIVISTELMÄ

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LIST OF SYMBOLS

Roman Letters

\( a \) constant in Hoek-Brown failure criterion ( )

\( GSI \) Geological Strength Index

\( dd \) dip direction angle from north ( degrees )

\( dip \) dip angle from horizontal ( degrees )

\( E \) Young’s modulus

\( m_b \) rock mass parameter in Hoek-Brown failure criterion ( )

\( m_i \) intact rock parameter in Hoek-Brown failure criterion ( )

\( n \) number of values

\( s.d. \) standard deviation

\( s_i \) intact rock parameter in Hoek-Brown failure criterion ( )

\( S_{1,2,3} \) major, intermediate, and minor principal stress ( Pa )

\( S_H \) major horizontal \textit{in situ} stress ( Pa )

\( S_h \) minor horizontal \textit{in situ} stress ( Pa )

\( S_v \) vertical \textit{in situ} stress ( Pa )

\( Q \) Rock mass quality classification value in Q-system

\( Q' \) Rock mass quality classification value in Q-system without the effect of stress and ground water

Greek Letters

\( \sigma_{1,2,3} \) major, intermediate, and minor principal stress ( Pa )

\( \sigma_{cd} \) crack initiation stress ( Pa )

\( \sigma_{ci} \) crack damage stress ( Pa )

\( \sigma_H \) major horizontal \textit{in situ} stress ( Pa )

\( \sigma_h \) minor horizontal \textit{in situ} stress ( Pa )

\( \sigma_v \) vertical \textit{in situ} stress ( Pa )

\( \sigma_t \) tensile strength ( Pa )

\( \sigma_{ucr} \) uniaxial compressive strength ( Pa )

\( \nu \) Poisson’s ratio ( )
**NOTATIONS**

<table>
<thead>
<tr>
<th>CR</th>
<th>Country rock</th>
</tr>
</thead>
<tbody>
<tr>
<td>HB</td>
<td>Hoek-Brown</td>
</tr>
<tr>
<td>MC</td>
<td>Mohr Coulomb</td>
</tr>
<tr>
<td>OKP</td>
<td>Outokumpu Mining Oy Pyhäsalmi Mine</td>
</tr>
</tbody>
</table>
PREFACE

This study is part of Posiva’s “Parvi” project where one objective is to collect information on cavern excavation and response in deep rock conditions. One special interest is the stress strength conditions in which the rock will fail around the excavation. The work has been commissioned and supervised by Posiva Oy and the following organisations and people have participated in the project:

Posiva Oy
Fortum Engineering Oy
Outokumpu Mining Oy
Pyhäsaari Mine
Outokumpu Mining Oy
Mining Technology Group
Gridpoint Finland Oy
Saanio & Riekkola Oy

Aimo Hautojärvi
Pekka Anttila
Pekka Perä
Marko Matinlasi
Seppo Tuovinen
Pekka Lappalainen
Matti Hakala
Harri Kuula
Petteri Somervuori
Pauli Syrjänen
Pasi Tolppanen
Reijo Riekkola
Erik Johansson
Nina Sacklén
1 INTRODUCTION

Posiva Oy, responsible for the final disposal of spent nuclear fuel in Finland, submitted an application in May 1999 to the Government for a policy decision in principle concerning the final disposal at the Olkiluoto site. In December 2000, the Government made, on the basis of this application, a favourable policy decision, which was also ratified by Parliament on May 18th, 2001. The decision in principle does not grant a license for the construction or operation of the facility in question, but it is needed to judge whether the facility is in line with the overall good of society. Currently, Posiva Oy is designing an underground investigation facility at the Olkiluoto site to complement the site investigations. Construction will be completed in 2010. While planning this next investigation phase, Posiva Oy is collecting information on cavern excavation and rock mass response in deep rock conditions.

It is planned to excavate the repository for spent nuclear fuel in crystalline bedrock at depths between 400 m and 600 m. Under these rock conditions, the in situ stress concentrations on the excavation boundary can exceed the critical strength of the rock and, thereby, initiate rock failure. Depending on the stress-strength ratio and rock type, failure can vary from minor local yielding to rock burst.

The best ways to evaluate stress failures, at present, are the rule of thumb, or the commonly-used engineering and scientific criteria. The rock strength, the in situ state of stress, and the orientation and shape of the excavation must be known for all the criteria. The orientation and shape of the excavation is normally well known, the in situ state of stress can usually be measured, and the short-term strength of intact rock can also be determined. However, the effect of size and time on rock mass strength is more complicated to define before the excavation and monitoring.

Probably the best-known site where stress failure criteria have been developed and tested is the URL (Underground Research Laboratory of Atomic Energy of Canada Ltd in Manitoba, Canada). The rock in URL is massive, unfractured homogeneous granite, thus differing from the Olkiluoto site where the rock is foliated and jointed. Therefore, additional information on valid failure criteria in the Olkiluoto type of rock mechanical conditions is needed.

In Finland, the Outokumpu Mining Oy Pyhäsalmi Mine (OKP) is the only place where similar rock conditions to Olkiluoto are easily accessible for the study. The rock type in Pyhäsalmi is schistose, metamorphic, sparsely jointed, and heterogeneous as in Olkiluoto, and the stress-strength ratio is equal (Figure 1-1, Hakala et al., 1998; Hakala et al., 1999; Äikäs et al., 2000). The most suitable depth for the research is the level between 1000 - 1400 m inside the volcanite country rock that hosts the massive sulfide ore deposit. OKP volcanite consists of felsic and mafic volcanites including pegmatite...
ore deposit. OKP volcanite consists of felsic and mafic volcanites including pegmatite veins and lenses. In this region, there is no large-scale mining and the in situ state of stress and the strength of the intact rock is quite well known. However, more information on the rock mass quality and stress failures is needed to understand the factors affecting stress failures and select the most suitable strength criteria.

\begin{figure}
\centering
\includegraphics[width=\textwidth]{figure1.png}
\caption{Stress-strength ratio at Olkiluoto mica gneiss and Pyhäsalmi volcanite (Hakala et al., 1998; Hakala et al., 1999; Äikäs et al., 2000).}
\end{figure}

In this report, the rock mechanical mapping system, that was obtained, is introduced and the collected data summarised and analysed. Furthermore, the secondary stress state around the typical excavations is calculated, and by using different failure criteria, is compared to observations. All the presented depth levels follow the existing system of the Pyhäsalmi mine. The effect of excavation work on the failures is excluded from the analyses.
2 OBJECTIVES

The main aim of this Posiva Oy – OKP co-operation research project is to collect and analyse information on stress failures from known geological and rock mechanical conditions, in order to develop a usable mapping and stress failure prediction method. Geological, rock mechanical, and stress failure mapping are obtained in the decline, level access, and access drifts situated in volcanites in Pyhäsalmi mine (Matinlassi, 2001). The ends of the ventilation shafts were also mapped. All the characterised drifts were excavated by the drill and blast method while the shafts were raise-bored.

The following stages have been planned to achieve the objectives:

1) development of the rock mechanical mapping system,
2) visualise and analyse OKP stress failures and data of the rock mechanical classification from the first mapping period, June 1999 to January 2000,
3) study factors that correlate between the failures and classified data,
4) summarise the rock mechanical testing data,
5) perform 3D-stress analyses for selected locations
6) compare the stress analyses results, using different failure criteria, to the observed behaviour.
3 SITE CHARACTERISTICS

3.1 Pyhäsalmi location and mining history

The Outokumpu Mining Oy Pyhäsalmi (OKP) mine is located in Central Finland about 450 km north of Helsinki (Figure 3-1). The Pyhäsalmi ore deposit was found in 1958. The first estimation of reserves indicating 12.2 Mt of ore with 0.81% Cu, 2.93% Zn, and 36.8% S, was made in February 1959. Construction work started in August 1959 and production commenced in March 1962, three and a half years after the discovery. The mine started as an open pit and the final open pit depth of 125 m was reached in 1967. Underground mining started the same year. The total production from the open pit was 6.8 Mt of ore and 5.6 Mt of waste rock. In 1967, after extension and automatisation of the processing plant, the annual production was increased from 600 000 to 800 000 tons. The current annual production from the underground mine is about 1.08 Mt. (Weihed & Mäki, 1997).

![Figure 3-1. Location of Pyhäsalmi mine and Olkiluoto investigation site.](image)

3.2 Ore formation

The supracrustal rocks in Pyhäjärvi are part of a Palaeoproterozoic, Svecofennian schist belt. The NW-trending schist belt is situated between the Archaean craton in the east and the Central Finland Granitoid Complex in the west.
The metavolcanic rocks in Pyhäsjarvi can be divided into two groups on the basis of field observations and chemical composition (Kousa et al., 1994). The western part of the Pyhäsjarvi area belongs to the Nivala gneiss complex. The Pyhäsalmi and Mullikkoräme ores are situated in the eastern part of the volcanic belt (the Pyhäsalmi volcanic complex). The Pyhäsalmi volcanic complex can be divided into the Ruotanen Formation and the Mullikkoräme Formation. A large gneiss area (the Kettuperä gneiss), which is intruded by syntectonic plutons is situated between the Ruotanen and Mullikkoräme formations. Both formations contain large areas of mafic volcanic rocks with pillow lavas, sodium-rich rhyolitic volcanic rocks, and volcaniclastic. The stratigraphy of the area is still unsolved. According to Lahtinen (1994), the Kettuperä gneiss and the rhyolites in the Riitavuori and Lippikylä Mb:s are chemically similar and may be genetically related. The age of the Kettuperä gneiss and the rhyolites, 1.93 and 1.92 Ga respectively, also suggests a common history (Kousa, 1990). Age determinations for the plagioclase porphyrites, which have been found as inclusions in the massive ore, indicate distinctly younger ages at 1875 Ma (Helovuori, 1979). These plagioclase porphyritic inclusions, as well as the quartz-porphyry and amphibolite inclusions, are unaltered and represent younger intrusive dykes. Similar dykes have been found in outcrops and drill cores outside the massive ore.

Field observations indicate that most of the mafic volcanic rocks seem to be younger than the main part of the felsic volcanic rocks. The volcanism started with felsic volcanism in an extensional continental margin, followed by mafic volcanism in a rifted marine environment. Large-scale hydrothermal alteration is associated with this stage causing strong sodium enrichment in the rhyolites. Ore formation near the centres of mafic volcanism seems to be related to this period. Without a longer hiatus, the volcanism continued with more calc-alkaline volcanism south and west of the Pyhäsalmi volcanic complex.

The Pyhäsalmi ore deposit is a typical volcanic hosted massive sulphide deposit. The volcanic rocks are mainly felsic pyroclastic rocks and coherent quartz-porphyries. Mafic volcanic rocks are coarse-grained tuff breccias and lavas, including pillow lavas. Mafic and felsic dykes are common. A lithological map of the Pyhäsalmi volcanic complex is shown in Figure 3-2.
Figure 3-2. Lithological map of the Pyhäsalmi volcanic complex (Weihed & Mäki, 1997).

The stratigraphy is unclear. Polyphase deformation together with amphibolite facies metamorphism makes the interpretation difficult. Both felsic and mafic volcanic rocks are strongly altered near the ore. The ore body has a complex shape due to the polyphase deformation and is surrounded by a large alteration zone. The contact between unaltered and altered volcanic rocks can be sharp or gradational. The thickness of the altered rock sequence at the surface is about 100 m in the west and 300 m in the east. At the deeper
levels, the alteration zone is only a few meters thick. Sericite-quartzites with a high pyrite content are presented adjacent to the ore. Pennitised cordierite porphyroblasts are common. Two zones of cordierite-anthophyllite rocks occur in the footwall, partly within the sericite quartzites. A zone of cordierite mica gneisses, that contain portions of a cordierite-anthophyllite rock, occurs outside the hanging wall sericite quartzites. The folded alteration zone is over 5 km long at the surface.

### 3.3 New mine

In spring 1996, an ore prospecting project was started in order to find mineral reserves below the +1050 level. Core drillings began in June 1996. As a result of the core drillings, the size and grade of mineralisation appeared to be auspicious and it was decided to excavate a research tunnel. This 500 m long tunnel was completed in October 1997. An intensive core-drilling period from the research tunnel and decline then began. Based on exploration results (22 000 m of drill core) the ore evaluation was completed in May 1998. The probable mineral reserves between level +1050 and +1350 were calculated to be:

<table>
<thead>
<tr>
<th>Mineralisation</th>
<th>Mass (t)</th>
<th>Cu (%)</th>
<th>Zn (%)</th>
<th>S (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Zn-mineralisation</td>
<td>3 155 000</td>
<td>0.9</td>
<td>5.1</td>
<td>39.9</td>
</tr>
<tr>
<td>Cu-mineralisation</td>
<td>11 400 000</td>
<td>1.4</td>
<td>1.1</td>
<td>45.0</td>
</tr>
<tr>
<td>Total</td>
<td>14 555 000</td>
<td>1.3</td>
<td>2.0</td>
<td>43.9</td>
</tr>
</tbody>
</table>

The mineralisation also contains silver 14 g/t and gold 0.4 g/t.

According to core drillings, the ore extends to level +1410 (Figures 3-3 and 3-4). The ore is formed as a top of fold with a massive pyrite ore core surrounded by copper ore and zinc ore. Contacts between the ore and surrounding rocks (volcanite) are sharp. Altered rock types in the contacts do not exist thus differing from the formation of "old ore".

The excavation of the decline from level +1100 started in April 1998. The end level, +1445, was reached in May 2000. A new hoisting shaft between the surface and level +1441 has been constructed and production began on July 1st, 2001.
3.4 Rock types

**Geology**

The massive Zn-Cu ore deposit at Pyhäsalmi is associated with Proterorozoic volcanites. Most of the volcanites are acid pyroclasts and porphyric rocks. The basic rocks in the mine area are mainly coarse tuff breccias and veins. The volcanism was bimodal in character. Both felsic and mafic volcanic rocks are strongly altered near the ore. The thickness of the altered rock sequence at the surface is about 100 m in the west and
300 m in the east. At the new mine area, the alteration zone around the massive sulphide ore is lacking. Sporadic lenses of week altered rocks can be fined as inclusions in massive sulphides.

Figure 3-4. West view of the new mine area. Copper and Zinc ores are presented in different colors.

**Acid volcanite**

The felsic volcanic rocks are sodium-rich rhyolites with high SiO₂ content. The essential minerals are quartz and alkaline feldspar. Schistosity is quite clear (Figure 3-5). Felsic rocks are altered near the ore at the upper part. Felsic rocks are sheared near the ore at the deeper parts. Grain size is under 1 mm. About 60% of the rocks are felsic volcanites in the mine scale.
**Mafic volcanite**

The mafic volcanic rocks are basaltic low K-tholeiites of primitive island arc type. The essential minerals are plagioclase and amphibole. The colour is very dark to black (Figure 3-6). Schistosity is quite clear near the ore but quite massive far away from the ore. The grain size is less than 1 mm. About 40% of the rocks are mafic volcanites in the mine scale.

**Figure 3-6.** Massive mafic volcanite (black) at the +1350 level.
**Pegmatites**

The pegmatites are a minor rock type in the Pyhäsalmi Mine (Figure 3-7). The pegmatites usually occur as a vein with varying thickness. Usually the veins are 0.1-2.0 metre wide and might be very long. The pegmatites intruded in many stages, so their composition varies slightly. The essential minerals are usually quartz and alkaline feldspar. The grain size ranges quite considerably from 5 mm to 20 mm.

![Figure 3-7. Pegmatite at the +1420 level.](image)

**Talc Schist**

The talc schists are also minor rock types in the mine. The talc schists are very soft with perfect cleavage into laminae (Figure 3-8). The talc schists are strongly deformed and altered and are originally were probably dolomitic limestones. In the upper parts, talc schists occur near the footwall contact, partly inside the ore. In the deeper parts, talc schist only occurs inside the ore.
Zn-Cu-S Ore

The Pyhäselmi ore deposit is a typical volcanic hosted massive sulphide deposit. The average grade of the ore in the New Mine is 1.1% Cu, 2.1% Zn, 38% S. The composition of the Pyhäselmi ore varies both horizontally and vertically. As a rule, chalcopyrite is concentrated in the central part of the ore and sphalerite near the contact in the New Mine. The highest barite contents are generally encountered in the sphalerite-rich areas. The Pyhäselmi ore is a massive pyrite ore with 70% sulphides (Figures 3-9 and 3-10). The sphalerite-rich ore is in some places finely banded and thin mylonitic bands are common. Round pyrite phenocrysts occur in the fine-grained sphalerite matrix of the mylonitic ore. A pyrite dissemination, which in some places has a breccia structure, exists around the massive ore.

Figure 3-8. Talc schists on the wall at the +1125 level.
Figure 3-9. Pyrite-copper ore in drill core 41 mm in diameter.

Figure 3-10. Drill core of Zinc-pyrite ore 41 mm in diameter.

Sericite-Quartzite

Near the ore the rhyolites are altered to sericite-rich schists with disseminated pyrite in the upper part of the ore. In the lower part near the ore, sericite-quartzite no longer exists. Sericite-quartzites were originally felsic volcanic rocks which can be deduced from the TiO₂/Zr ratio.
4  MAPPING

Data for the analysis is collected from decline tunnels, level accesses, and access drifts. The mapping system used in this project is mainly based on the Q-classification system (Grimstad & Barton, 1993) and the Finnish RG-classification systems (Korhonen et al., 1974) and briefly presented here. A more detailed description of the mapping system is presented in Matinlassi (2001). In the following paragraphs, the main categories of collected data such as rock type, rock tunneling quality index Q (visual estimate), fracture zones and major joints, stress failures, and scanline mapping, as well as the contents of the individual databases are presented. Down the tunnels, all the data is noted and stored as data sheets in the Excel™ program. The starting and finishing points are based on the metric point system in the mine.

Rock type database

<table>
<thead>
<tr>
<th>Field</th>
<th>description</th>
<th>value</th>
</tr>
</thead>
<tbody>
<tr>
<td>• Level</td>
<td></td>
<td></td>
</tr>
<tr>
<td>• Metric point</td>
<td></td>
<td></td>
</tr>
<tr>
<td>• Starting point of rock type</td>
<td></td>
<td></td>
</tr>
<tr>
<td>• Ending points of rock type</td>
<td></td>
<td></td>
</tr>
<tr>
<td>• Rock type (visual estimation)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>• Rock type fabric</td>
<td>massive</td>
<td>M0 = non foliated</td>
</tr>
<tr>
<td></td>
<td></td>
<td>M1 = slightly foliated</td>
</tr>
<tr>
<td></td>
<td>foliated</td>
<td>L2 = moderately foliated</td>
</tr>
<tr>
<td></td>
<td></td>
<td>L3 = strongly foliated</td>
</tr>
<tr>
<td></td>
<td>composite</td>
<td>S0 = non foliated</td>
</tr>
<tr>
<td></td>
<td></td>
<td>S1 = slightly foliated</td>
</tr>
<tr>
<td></td>
<td></td>
<td>S2 = moderately foliated</td>
</tr>
<tr>
<td></td>
<td></td>
<td>S3 = strongly foliated</td>
</tr>
</tbody>
</table>

• Dip angle in degrees

• Dip direction in degrees

• Grain size

| fine-grained | Vr (grain size) < 1 mm |
| medium-grained | Vr 1-5 mm |
| coarse-grained | Vr 5-10 mm |
| very coarse-grained | Vr > 50 mm |

• Degree of weathering

| unweathered | Rp0 |
| slightly weathered | Rp1 |
| highly weathered | Rp2 |
| fully weathered | Rp3 |

• Remarks
**Visual estimation of the Q-index**

<table>
<thead>
<tr>
<th>Field</th>
<th>value</th>
</tr>
</thead>
<tbody>
<tr>
<td>• Level</td>
<td></td>
</tr>
<tr>
<td>• Metric point</td>
<td></td>
</tr>
<tr>
<td>• Starting point of rock type</td>
<td></td>
</tr>
<tr>
<td>• Ending point of rock type</td>
<td></td>
</tr>
<tr>
<td>• RQD (%) from the walls</td>
<td>(according to Hutchinson and Diederichs 1996)</td>
</tr>
<tr>
<td>• Jn, joint set number</td>
<td>(according to Barton et al. 1974)</td>
</tr>
<tr>
<td>• Jr, joint roughness number</td>
<td>(according to Barton et al. 1974)</td>
</tr>
<tr>
<td>• Ja, joint alteration number</td>
<td>(according to Barton et al. 1974)</td>
</tr>
<tr>
<td>• Jw, joint water reduction number</td>
<td>(according to Barton et al. 1974)</td>
</tr>
<tr>
<td>• SRF, stress reduction factor</td>
<td>(according to Barton et al. 1974)</td>
</tr>
<tr>
<td>• Rock mass description</td>
<td>calculated in spreadsheet based on input values</td>
</tr>
<tr>
<td>• Remarks</td>
<td></td>
</tr>
</tbody>
</table>

**Fracture zones and major joints**

All fracture zones and joints over 20 m in length are mapped and photographed. Only the joints, which continue outside the tunnel, are included. The fracture zones and major data sheet include 9 records:

<table>
<thead>
<tr>
<th>Field</th>
<th>description</th>
<th>value</th>
</tr>
</thead>
<tbody>
<tr>
<td>• Level</td>
<td></td>
<td></td>
</tr>
<tr>
<td>• Metric point</td>
<td></td>
<td></td>
</tr>
<tr>
<td>• Location</td>
<td></td>
<td></td>
</tr>
<tr>
<td>• Dip angle in degrees</td>
<td></td>
<td></td>
</tr>
<tr>
<td>• Dip direction in degrees</td>
<td></td>
<td></td>
</tr>
<tr>
<td>• Type of failure zone</td>
<td>one or few planar joints</td>
<td>Rll</td>
</tr>
<tr>
<td></td>
<td>high fissure frequency, no fissure filling</td>
<td>Rll</td>
</tr>
<tr>
<td></td>
<td>densely fractured, fractures poorly filled</td>
<td>RIII</td>
</tr>
<tr>
<td></td>
<td>high or dense facturing, crushes</td>
<td></td>
</tr>
<tr>
<td></td>
<td>filled with clay</td>
<td>RIV</td>
</tr>
<tr>
<td></td>
<td>abundant gauge</td>
<td>RiV</td>
</tr>
<tr>
<td>• Jr, joint roughness number</td>
<td></td>
<td></td>
</tr>
<tr>
<td>• Ja, joint alteration number</td>
<td></td>
<td></td>
</tr>
<tr>
<td>• Thickness of the fracture zone</td>
<td></td>
<td></td>
</tr>
<tr>
<td>• Joint filling</td>
<td></td>
<td></td>
</tr>
<tr>
<td>• Remarks</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
Stress failure mapping

Stress failures are mapped only if they can be clearly noted to be induced by stress concentrations. In some cases, it is not so straightforward. Failure locations in a tunnel profile are also recorded. The stress failure mapping data sheet includes the following 10 records:

<table>
<thead>
<tr>
<th>Field</th>
<th>description</th>
<th>value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Level</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Metric point</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Starting point</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Ending point</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Type of stress failure</td>
<td>none</td>
<td>J0</td>
</tr>
<tr>
<td></td>
<td>some noise, not spalling</td>
<td>J1</td>
</tr>
<tr>
<td></td>
<td>visual deformations or cracks in shotcrete</td>
<td>J2</td>
</tr>
<tr>
<td></td>
<td>light spalling</td>
<td>J3</td>
</tr>
<tr>
<td></td>
<td>spalling and rock burst</td>
<td>J4</td>
</tr>
<tr>
<td></td>
<td>heavy rock burst</td>
<td>J5</td>
</tr>
<tr>
<td>Width of failure</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Depth of failure</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Location</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Breaking type</td>
<td>failure by breakage</td>
<td>M</td>
</tr>
<tr>
<td></td>
<td>failure along existing joint</td>
<td>R</td>
</tr>
<tr>
<td></td>
<td>mixture of breakage and failure along existing joint</td>
<td>M/R</td>
</tr>
<tr>
<td>Excavation date</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Shotcrete date</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Remarks</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
Scanline mapping

Scanline mapping is performed in locations where more detailed data is required. Photographs are also taken of the scanline mapped area. The scanline mapping data sheet includes headline data and 13 records. In the headline, the following subjects are defined:

<table>
<thead>
<tr>
<th>Field</th>
<th>description</th>
</tr>
</thead>
<tbody>
<tr>
<td>Level</td>
<td></td>
</tr>
<tr>
<td>Metric point</td>
<td></td>
</tr>
<tr>
<td>Code of the drift</td>
<td></td>
</tr>
<tr>
<td>Code of the scanline</td>
<td></td>
</tr>
<tr>
<td>Rock type</td>
<td></td>
</tr>
<tr>
<td>Starting point</td>
<td></td>
</tr>
<tr>
<td>Dip and dip direction of scanline (degrees)</td>
<td></td>
</tr>
<tr>
<td>Length of the scanline in metres</td>
<td></td>
</tr>
<tr>
<td>Jn, joint set number</td>
<td></td>
</tr>
<tr>
<td>RQD-value</td>
<td></td>
</tr>
<tr>
<td>Jw, joint water reduction number</td>
<td></td>
</tr>
<tr>
<td>SRF, stress reduction factor</td>
<td></td>
</tr>
<tr>
<td>Date and name of the geologist</td>
<td></td>
</tr>
<tr>
<td>Rock mass description</td>
<td>calculated in spreadsheet based on input values</td>
</tr>
<tr>
<td>Average Jr, joint roughness number</td>
<td></td>
</tr>
<tr>
<td>Average Ja, joint alteration number</td>
<td></td>
</tr>
</tbody>
</table>

13 records in the data sheet:

| ID number of joint           |                                                  |
| Dip angle in degrees         |                                                  |
| Dip direction in degrees     |                                                  |
| Joint spacing                |                                                  |
| Joint length                 |                                                  |
| Joint continuity             |                                                  |
| Joint undulating             |                                                  |
| Jr, joint roughness number   |                                                  |
| Ja, joint alteration number  |                                                  |
| Joint opening                |                                                  |
| Joint filling                |                                                  |
| Type of failure zone         |                                                  |
| Remarks                      |                                                  |
5 VISUALISATION OF MAPPING DATA

In the following pages, a visualised overview of mapped data is shown while the more detailed analyses are presented in Chapter 6. Visualisation was carried out using the software, Surpac2000 Version 3.2, which is a geological modelling, mine design, and production system. The software has applications at all stages of the mine life-cycle from resource estimation, planning and operation to remediation of the site. Originally Surpac was developed specifically for mining but it has been found to be useful for other geotechnical design areas, for example, tunnelling. (Somervuori & Middleton, 2001; Surpac Software International, 2001).

During the first mapping period, June 1999 - January 2000, a total of 2.6 km of tunnels were excavated, of which 1.7 km were mapped (Figure 5-1). A typical tunnel cross-section is 5*5 m² with a slightly arched roof. The majority of the mapped tunnel sections are between the +1200 and +1400 levels (Figure 5-2). The tunnels are mainly straight or curved although there are also crossing areas and wider profiles (Figure 5-3). The crossing area is assumed to extend 10 m away from the physical crossing.

Figure 5-1. Tunnels below the +1050 level coloured by the depth.
Figure 5-2. Excavated and mapped tunnel sections.

Figure 5-3. Mapped tunnel sections, crossing areas and wider profiles highlighted.
In the mapped tunnels the rock type is mainly volcanite but some sections of ore, pegmatite, and sericite-quartzite also exist (Figures 5-4 and 5-5). Clear mafic or felsic volcanite sections are rare while felsic volcanite with mafic volcanite stripes dominates.

**Figure 5-4.** Main rock types in the mapping area. The light grey areas are not mapped.

The volcanites are slightly or moderately foliated but the ore is massive (Figure 5-6). The dip direction of the foliation is perpendicular to the volcanite-ore contact and the dip angle is steepest close to the ore contact (Figures 5-7 to 5-9).
Figure 5-5. Rock types, all volcanites are included in one category.

Figure 5-6. Rock fabric.
Figure 5-7. 3D-view of foliation orientation.

Figure 5-8. Top view of foliation orientation.
The volcanites has fine grain size but the ore is medium or coarse grained (Figure 5-10). All rock types are unweathered. The mapped rock mass is sparsely jointed; the average RQD-value being 87 (Figure 5-11). Volcanites normally have one joint set along the local foliation. Random jointing is typical in ore (Figure 5-12).
Figure 5-10. Grain size of mapped rocks.

Figure 5-11. RQD-value of mapped tunnel sections.
Figure 5-12. Number of joint sets; In number and number of joint set.

The majority of mapped fractures belong to class RII, being a planar fracture or set of planar fractures (see table on page 25, Figure 5-13). Some RII and RIII class fractures were also encountered but none with crushed filling. The fracture orientation mainly corresponds with the foliation (Figure 5-7 – 5-9). In addition, some sub-horizontal fractures were found (Figures 5-14 to 5-16).
Figure 5-13. Fracture types based on the Ri fracture classification system (see Table on page 25).

Figure 5-14. 3D-view of fracture orientations.
Figure 5-15. Top view of fracture orientations. Exceptional subhorizontal orientations near the ore body (blue arrow).

Figure 5-16. Stereonet projections of fracture orientation.
Based on the rock mechanical mapping, the rock quality is \textit{good} or \textit{very good}, that is, \( Q' \)-value varies from 10 to 100 (Figure 5-17). GSI values (Eq. 5-1, Hoek \textit{et al.} 1995) vary between 70 and 90, respectively (Figure 5-18). Only a few sections have a GSI value in the range of 60 to 70

\[
GSI = 44 + 9\ln(Q')
\]  

\textbf{(5-1)}

\[40-100, \text{VERY GOOD}\]
\[10 - 40, \text{GOOD}\]
\[4 - 10, \text{FAIR}\]
\[1 - 4, \text{POOR}\]
\[0.1 - 1, \text{VERY POOR}\]

\textit{Figure 5-17.} \textit{Q'} rock mass quality value.
Figure 5-18. GSI rock mass quality value.

The majority of observed stress failures belong to classes J1 = noise or J3 = light spalling (Figure 5-19). In a few locations, moderate spalling or rock burst, class J3, were also found. The observed stress failures at tunnel cross sections or at wider profiles do not differ considerably from the overall behaviour (Figure 5-20).
Figure 5-19. Stress failure observations at mapped tunnel sections.

Figure 5-20. Stress failure observations at tunnel cross sections and wider profiles.
6 ANALYSES OF MAPPING DATA

6.1 General

Collected data from mapping is analysed to find the factors, which are mainly correlating with stress failure observations. The study is focused on the in situ state of stress, direction of excavation, size and shape of excavation, geological structures, rock type, rock fabric, orientation of rock foliation, and rock mass quality. Some of these factors are connected to others and cannot be studied independently.

Mapping data

A total of 1.7 km out of the approximately 2.6 km of excavated tunnels was mapped for this study (Figure 6-1). The corresponding values in the volcanic country rock are 1.3 km / 2.2 km and in the ore 0.25 km / 0.4 km. The majority of data is collected below the +1200 level. Mapping always follows the access tunnel directions, so, in the directions 30°, 60°, 90° and 150°, the length of mapped country rock tunnels is less than 50 m (Figure 6-2).

Figure 6-1. Total length (m) of tunnel excavation and mapping.
**Figure 6-2.** Length of tunnel excavation and mapping sections in different drift directions; CR is country rock.

### 6.2 Stress failures in tunnels

**Rock type versus stress failure observations**

The majority (72%) of the tunnels were excavated in felsic and/or mafic volcanites and 50% of which is striping felsic and mafic volcanites (Volc \_F\_Ms in Figure 6-3).

Three stress failure classes dominate; J0 (no failure, 34%), J1 (noise only, 36%) or J3 (light spalling, 26%). Only 4% of all the observations belonged to classes J2 (visual deformations or cracks in shotcrete) and J4 (spalling and rock burst). This result might be possible, but the definition between classes J2 to J4 is not clear enough to draw further conclusions. Therefore, classes J2 to J4 will be hereafter treated as one stress failure class J2…J4.
Total drift length / stress induced failure (m)

Rock type / Contact type

Volc_F
Volc_M
Volc_FM
Volc_F_Ms
Volc_SerQua
Volc_Ore
Volc_Pg
Pg
Pg_Ore
Ore

Figure 6-3. Stress failure classification within mapped rock types. The amount of each rock type is presented in parentheses. Failure classes J0...J4 are explained in Chapter 4.

When comparing the rock types and occurrence of all types of stress failures, the pegmatite and pegmatite-volcanite contacts are the worst (Figure 6-4). Occurrences in one main class are calculated by using

\[ \frac{l_{\text{sub-class,i}}}{\sum_{i=1,n} l_{\text{sub-class,i}}} \times 100\% \], where \( l = \text{length} \), \( n = \text{number of classes} \).

If class J1 (noise) is omitted then the mafic volcanite and serizitequartzizite-volcanite contacts are the worst, and the situation is best in the ore. Generally, in pegmatite 20% to 25% of the excavated tunnels do not show signs of stress failure, while in volcanites this portion is 30% to 35%, and in the ore it is over 40%.

Location, type and extent of stress failure

The location at the tunnel surface and breaking type could be defined for only 17% of all J2 - J4 type failure observations. Furthermore, in only a few cases was it possible to define the volume of a failure. Almost all of these were found at the tunnel roof. (In half
of these cases, the excavation direction was either 120° or 270° to the north, and the third major direction was 330°. Based on these facts, it was decided to ignore the location and breaking type for the following analyses and use the length of mapping section instead to represent the extent of the stress failure.

![Figure 6-4](image.png)

**Figure 6-4.** Occurrence of stress failure within the mapped rock types. The failure classes J0...J4 are explained in Chapter 4. The amount of each rock type is presented in parentheses.

**NOTE!** All the following analyses concentrate on the volcanites, which are the dominating rock types. Although, there are some differences in stress failure observations between the different types of volcanites (Figure 6-4) they are hereafter treated as one rock type.

**Delay between excavation and mapping**

A clear correlation between a delay in the mapping time after excavation to the stress failures observed is seen; more and stronger failures are observed immediately after excavation. In the volcano sections, half of the mapping was done within a day of excavation and only 15% after three days (Figure 6-5). In observations obtained within a day of excavation, 80% of the sections show some sign of stress failure. After three
days, the corresponding amount is less than half. There is an equal decrease in visual and sonic stress failure observations.

![Bar chart showing the effect of the time period between excavation and mapping on the observed stress failures in volcanite. The failure classes J0...J4 are explained in Chapter 4.](image)

**Figure 6-5.** The effect of the time period between excavation and mapping on the observed stress failures in volcanite. The failure classes J0...J4 are explained in Chapter 4.

**Depth level**

The effect of the depth level, which should correlate with the stress magnitude, is quite obvious when looking at the observations in volcanite within two days of excavation (Figure 6-6). The occurrence of visible stress failures clearly increases with depth, but the total number of stress failures does not increase monotonically. One explanation for the result can be the unequal representation of different tunnel orientations at different depth levels. Generally, the occurrence of stress failures seemed to be 20% higher below the +1300 levels than above the +1200 level. The effect of the depth is practically the same if data within one day of excavation is considered.

**Tunnel shape**

About 10% of mapped volcanite tunnels are in crossing areas or the profile is wider than the typical tunnel size. These areas have 10% to 15% higher stress failure potential than straight or slightly curved tunnels (Figure 6-7). This can be understood by higher stress concentrations and lower confinements.
Occurrence of stress induced failure

Figure 6-6. The effect of the depth level on observed stress failures in volcanite; observations within two days of excavation. The failure classes J0...J4 are explained in Chapter 4.

Occurrence of stress induced failure

Figure 6-7. The effect of the tunnel shape on observed stress failures in volcanite; observations within three days of excavation. The failure classes J0...J4 are explained in Chapter 4.
**Direction of excavation and orientation of foliation**

In Pyhäsalmi vulcanite, the direction of excavation compared to the major principal stress and orientation of the foliation can not be studied independently. The fact that the dip direction of the foliation is towards the ore body and increases when approaching the ore body (Figures 5-7 to 5-9) makes the situation more complex. The average trend of the major principal stress is $115^\circ / 295^\circ$, thus, stress condition is most favourable when the tunnel is driven in these directions. The highest secondary stress at the tunnel roof is induced when the tunnel is driven in a perpendicular direction to the major principal stress; that is, in the directions $25^\circ$ or $205^\circ$. The average dip direction of the foliation is $185^\circ$ and 50% of the failure observations are between $170^\circ$ to $205^\circ$ and 90% between $120^\circ$ to $225^\circ$.

The stress failure observations from a day after excavation and within three days of excavation correlate with the orientation of major principal stress and dip direction of foliation. The probability of stress failure is highest when the tunnel is excavated perpendicular to the major principal stress or with the strike of foliation (Figures 6-8 and 6-9). In the latter case, the shear failure potential of the foliation planes is highest when the tunnel is driven in the strike direction of the foliation. On the other hand, the lack of data in many directions, and the change of the foliation plane dip direction, make this conclusion even more uncertain.

**Figure 6-8. The effect of the excavation direction on the observed stress failures in volcanite when the observation time was less than one day after excavation.**
The result is almost the same even if the dip direction of the foliation is limited between 165° to 215° (see Figure 5-8) and all the stress failure observations related to geological structures such as lenses, structures, and veins are rejected (Figure 6-10). On the other hand, the probability of visual stress failure is decreased by some 20%.

Figure 6-9. The effect of the excavation direction on observed stress failures in volcanite when the observation time was less than three days after excavation.

**Dip of foliation**

The effect of the dip angle of the foliation was studied in three tunnel excavation directions 120°, 180°, and 240° having the highest amount of data. The observations were limited to the first two days after excavation. In these cases, the occurrence of stress failure is approximately 15% higher with lower dip values, but the type of damage does not correlate with the dip (Figure 6-11).
Occurrence of stress failures with tunnel excavation direction
- volcanite, observation period is three days after excavation,
- dip direction of foliation limited between 165° and 205°, data with geological structures rejected

Trend of the major principal stress 280° - 310°

Figure 6-10. The effect of excavation direction on observed stress failures in volcanite when the observation time was less than three days after excavation. The dip direction of the foliation is limited to between 165°-205° and all data associated with local geological structures is rejected.

Fabric of rock

The effect of fabric, that is, particle arrangement, was studied based on the same data set, which was used previously for the dip of the foliation study. The volcanite is massive (M), and slightly (S1) or moderately schistose (S2). The fabric does not correlate with the stress failure data so that it can be seen over the other factors (Figure 6-12). Neither is any correlation seen if all volcanite data is studied.

Rock mass quality

The effect of the rock mass quality was studied based on the same data set, which was used in the two previous studies. The Geological Strength Index GSI of the volcanites varied between 58 and 82 and the average was 69 (Figure 6-13). In two tunnel directions, severe damage was observed in the better rock quality, and in one orientation no correlation could be seen. If all data from volcanite tunnels is studied then more severe stress failures are observed if the GSI value is above 65.
Occurrence of stress induced failure

Figure 6-11. The effect of the dip angle of the foliation on observed stress failures in volcanite. The observation time was less than two days after excavation.

Occurrence of stress induced failure

Figure 6-12. The effect of rock fabric on observed stress failures in volcanite. The observation time was less than two days after excavation. M = massive, S1 = slightly, and S2 = moderately schistose.
Figure 6-13. The effect of rock mass quality on observed stress failures in volcanite. The observation time was less than two days after excavation.

Other factors

The effect of grain size or degree of weathering could not be studied because all the volcanites are fine grained and unweathered.

6.3 Stress failures in shafts

During the first mapping period, there was no opportunity to proceed with the rock mechanical mapping in the shafts. Some visual observations were made from the tunnels above and below the shafts. In all cases, the stress failures were clear and were seen in the directions 10°-30° and 190°-210°. Inside a shaft of 3.1 m in diameter (Figure 14), the width of the failure area was about one metre and the depth was a few dozen centimetres.

The stress failures in shafts are especially interesting because they can be considered systematic and continuous and the effect of the excavation method is found to be minor. These observations confirm the in situ stress measurement results, where the major in situ stress is in the direction of 110° / 290°.
Figure 6-14. Stress failures in a 3.1 m wide shaft (ore pass) at +1375 level; the failure orientations are 20° (left) and 200° (right).
7 ROCK MECHANICAL DATA

7.1 Intact rock

For the new mine project, several uniaxial compressive and indirect tensile tests were conducted for the main rock types (Hakala et al., 1998 and 1999). Rock mechanical parameters like uniaxial compressive strength ($\sigma_c$), crack damage stress ($\sigma_d$), crack initiation stress ($\sigma_i$), tensile strength ($\sigma_t$), and the elastic parameters, Young's modulus (E) and Poisson's ratio ($\nu$), have been defined (Table 7-1). The data of intact rock is only missing for serizite schist and serizite quartzite, but, from the previous studies of old mine area, it is known that the average uniaxial compressive strength of these rock types varies between 40 MPa - 80 MPa.

Table 7-1. Strength and deformation parameter values for OKP intact rock types.

<table>
<thead>
<tr>
<th>Rock type</th>
<th>Young's modulus ( GPa )</th>
<th>Poisson's ratio</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>ave</td>
<td>stdev</td>
</tr>
<tr>
<td>Volcanite, Felsic</td>
<td>68</td>
<td>24%</td>
</tr>
<tr>
<td>Volcanite, Mafic</td>
<td>76</td>
<td>21%</td>
</tr>
<tr>
<td>Pegmatite</td>
<td>63</td>
<td>13%</td>
</tr>
<tr>
<td>Ore, Zinc</td>
<td>98</td>
<td>37%</td>
</tr>
<tr>
<td>Ore, Copper</td>
<td>132</td>
<td>33%</td>
</tr>
<tr>
<td>Pyrite</td>
<td>120</td>
<td>21%</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Rock type</th>
<th>Uniaxial compressive strength ( MPa )</th>
<th>Tensile strength ( MPa )</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>ave</td>
<td>stdev</td>
</tr>
<tr>
<td>Volcanite, Felsic</td>
<td>241</td>
<td>18%</td>
</tr>
<tr>
<td>Volcanite, Mafic</td>
<td>206</td>
<td>45%</td>
</tr>
<tr>
<td>Pegmatite</td>
<td>119</td>
<td>10%</td>
</tr>
<tr>
<td>Ore, Zinc</td>
<td>92</td>
<td>39%</td>
</tr>
<tr>
<td>Ore, Copper</td>
<td>123</td>
<td>38%</td>
</tr>
<tr>
<td>Pyrite</td>
<td>93</td>
<td>24%</td>
</tr>
</tbody>
</table>

The following is mainly a study of the volcanites. These are the major rock types in the mapping area. No difference between felsic and mafic volcanite can be found from the test results (Figures 7-1 to 7-3). The angle of foliation is not defined for all specimens. However, based on the data available, some correlation with foliation and strength of mafic volcanite has been found, but the deviation in the results is very high and so no definite conclusions can be drawn (Figure 7-4). Because the felsic and mafic volcanites are typically frequently layered and the deviation of the results overlap strongly, it was decided to treat them as one volcanite rock type.
Figure 7-1. Uniaxial compressive strength of felsic and mafic volcanites.

Figure 7-2. Indirect tensile strength of felsic and mafic volcanites.
Figure 7-3. Young's modulus of felsic and mafic volcanites.

Figure 7-4. Uniaxial compressive strength of felsic and mafic volcanites versus the foliation angle.
Table 7-2 and Figures 7-5 to 7-10 summarise the statistical values of the rock mechanical parameters of volcanite. In all the figures, the cumulative probability together with the normal distribution at 10%, median, and 90% values, as well as quartals are presented.

### Table 7-2. Statistical parameters of normal distribution and quartiles for volcanic rock.

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Normal distributions</th>
<th>Quartiles</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>10%</td>
<td>Median</td>
</tr>
<tr>
<td>Youngs modulus, E</td>
<td>61</td>
<td>75</td>
</tr>
<tr>
<td>Poisson ratio, v</td>
<td>0.21</td>
<td>0.26</td>
</tr>
<tr>
<td>Tensile strength, σ_t</td>
<td>9.3</td>
<td>16.5</td>
</tr>
<tr>
<td>Crack initiation stress, σ_ci</td>
<td>63</td>
<td>101</td>
</tr>
<tr>
<td>Crack damage stress, σ_cd</td>
<td>106</td>
<td>184</td>
</tr>
<tr>
<td>Peak strength, σ_p</td>
<td>129</td>
<td>222</td>
</tr>
</tbody>
</table>

**Figure 7-5.** Deviation of Young's modulus of OKP volcanite.
Figure 7-6. Deviation of Poisson's ratio of OKP volcanite.

Figure 7-7. Deviation of tensile strength $\sigma_t$ of OKP volcanite.
Figure 7-8. Deviation of the crack initiation stress $\sigma_{ci}$ of OKP volcanite.

Figure 7-9. Deviation of the crack damage stress $\sigma_{cd}$ of OKP volcanite.
Figure 7-10. Deviation of the peak strength $\sigma_p$ of OKP volcanite.

For the purposes of the stress failure study, the probability envelopes for the strength and critical stress states are defined (Figures 7-11 and 7-12). Tensile and uniaxial compressive strength data (see Figs. 7-7 and 7-10) is used in pairs; high tensile strength correlates with the high uniaxial strength of the material. The cumulative probability strength envelope presentation means that with a probability of $n\%$, the strength envelope is below the C.p. $n\%$ envelope. Thus, C.p.50% is an average strength envelope and with 95% probability the strength envelope is between the envelopes C.p. 2.5% and C.p. 97.5%. The stress envelope for crack damage ($\sigma_{cd}$) follows the Hoek-Brown criteria with constant $m_i$ -value equal to $m_i$ for peak strength (Hoek & Brown, 1980). The crack initiation ($\sigma_{ci}$) envelope is based on the formula $\sigma_1 - \sigma_3 = \sigma_{ci}$ (Read et al., 1998). For peak strength, $\sigma_{cd}$ and $\sigma_{ci}$ the $\sigma_1$ values at $\sigma_3 = 0$ are according to Figures 7-8 ... 7-10.
Figure 7-11. Probability of strength envelope of OKP volcanite.

Figure 7-12. Probability of critical stress states envelopes of OKP volcanite.
7.2 Rock Mass

The rock mass parameters are defined from intact rock parameters together with rock mass classifications values, that is, joint characteristics (Hoek et al., 1995). When the GSI-value is above 25, the rock mass strength envelope can be defined using the following equations:

\[
\sigma_1 = \sigma_3 + \sigma_c \left( m_b \frac{\sigma_3}{\sigma_c} + s \right)^a
\]  

\[ (7-1) \]

\[
m_b = m_t \exp \left( \frac{GSI - 100}{28} \right) \]  

\[ (7-2) \]

\[
s = \exp \left( \frac{GSI - 100}{9} \right) \]  

\[ (7-3) \]

where

- \( a = 0.5 \), constant for Hoek-Brown failure criteria
- \( \sigma_c \) = uniaxial compressive strength of intact rock
- \( m_t \) = is a Hoek-Brown intact rock strength parameter, which is defined as best fit for the intact rock tensile and compressive strength envelope. The form of envelope is equal to Equation 7-1 assuming that \( s=1 \) and \( a=0.5 \).

Taking into account the deviation of the GSI value and intact rock strength values (Figures 7-7, 7-10 and 7-13), the rock mass strength envelope is defined using the Monte-Carlo simulation (Figure 7-14).
Figure 7-13. Deviation of GSI-value of OKP volcanite.

Figure 7-14. Probability envelopes for Hoek-Brown rock mass strength of OKP volcanite analysed using the Monte-Carlo simulation; C.p = cumulative probability.
7.3 In situ state of stress

In the new mine area the \textit{in situ} state of stress was measured in two. Firstly, the measurements were obtained in three locations at the +1125 level and, secondly, at the +1325, +1350, and +1375 levels (Ledger, 1999; Mononen, 2000). At least two measurements were obtained at each location. At the +1125 level, five of the seven measurements were successful, but, at deeper levels all measurements were quite uncertain because of core damage through disking.

At the +1125 level, two measuring locations were in the, on different sides of the ore body, and the third location was in the ore. The difference between the five successful measurements was very minimal. The major and intermediate principal stresses were almost horizontal and the minor was vertical. The average major principal stress was 65 MPa dipping 5 degrees to direction 310. The average horizontal stress ratio $\sigma_H/\sigma_h$ was 1.6 and stress ratio $\sigma_H/\sigma_v$ was 2.0 (Figure 7-15).

![Figure 7-15. The stress ratio versus depth of the latest in situ stress measurements at the Pyhäsalmi mine and Posiva investigation sites together with results around the world (Modified from Hoek & Brown, 1980).]
In the +1350 level, all the measurements were done in the volcanites. The only successful measurement as well as the failed ones indicate major horizontal stress of 75 MPa in a direction of 290° (Figure 7-16). The horizontal stress ratio $\sigma_H/\sigma_h$ was 1.7 and the vertical $\sigma_H/\sigma_v$ was 1.8.

*Figure 7-16. Orientation of the major and minor principal stresses.*

The stress failure observations in shafts confirm the bearing of the major principal stress to be between 280 to 310 MPa, but the magnitudes of all principal stress components at the +1350 level are not certain.
8 STRESS STRENGTH ANALYSES

8.1 Cases and failure criteria

The numerical stress strength simulations were done using a three dimensional boundary element code Examine$^3$D (RocScience, 1999). The volcanite rock mass surrounding the excavations was assumed to be homogenous linear elastic. This assumption was made because no reliable test data on the effect of the anisotropy was available. Two in situ stress regimes were studied; magnitudes of 65 MPa or 75 MPa for major in situ principal stress were used, which corresponds to the depth levels of +1125 and +1350. The horizontal stress ratios ($\sigma_H/\sigma_h$) have a value of 1.6 or 1.7 and the horizontal - vertical stress ratio ($\sigma_H/\sigma_v$) was 2.0 or 1.8.

The simulated horizontal tunnels were the typical 5 m * 5 m face area with a vertical wall of 4 m height (Figure 8-1). The vertical shaft was 3.1 m in diameter (Figure 8-2). In both cases, the length of excavations was over ten times the maximum radius to secure a two dimensional stress state at the middle of the excavation. The excavation faces were chamfered to avoid stress discontinuity. The development of secondary stresses with advancing coring was monitored by setting lines of “field points” parallel to the excavation, extending five radii from the face in both directions. In the tunnel model, the stresses were monitored at the tunnel roof, in the upper corner, and in the middle of the wall. In each location, there were three field point lines at the distances of 1 cm, 5 cm, and 15 cm from the excavation surface. In the shaft model, the field point lines were on the sides of the major and minor horizontal stress and the third was between the first and second.

Five excavation directions were simulated for the tunnel models. The angle between the major horizontal stress and tunnel axis was assumed to be 90, 60, 45, 30, or 0 degrees. In the case of the vertical shaft model, similar studies were not needed because of the circular shape. The elastic principal stress paths of these eight tunnel simulations and two shaft simulations were compared to the following failure criteria:

1) Peak strength of intact rock.

2) Failure criteria presented by Read et al. (1998), which is based on crack damage and crack initiation strength surfaces as well as confining stress. In this criterion, the crack damage strength is assumed to be the true strength of the rock. The rock is assumed to be damaged, if the stress exceeds the crack initiation surface and confinement is less than 5% of maximum stress. This damage can reduce the crack damage strength. The rate of strength reduction is not given, but can be defined from in situ observations.

3) Hoek-Brown rock mass strength criteria (Hoek & Brown, 1997 and 1980).
All the failure criteria were presented in a form of probability based on laboratory test values and mapped rock mass quality, which are presented in previous chapters (Figures 7-11, 7-12 and 7-14).

Figure 8-1. Simulation model for the tunnel (a) and the lines of field points close to the tunnel face.
Figure 8-2. Simulation model for the shaft (a) and the lines of field points close to the tunnel face.

8.2 Tunnel simulation

Secondary stress state at tunnel surface

The secondary state of stress refers to the situation when excavation has passed the observation point and the advancing excavation no longer affects that point. The highest compressive stresses take place at the tunnel roof and upper corner when the major horizontal stress is perpendicular to the tunnel axis (Figures 8-3 to 8-5). The maximum stress concentration values are 135 MPa to 155 MPa being 2.0 – 2.1 times the maximum in situ stress. The minor principal stress is zero at the roof, but in the upper edge it is 2 – 5 MPa in tension. At the tunnel wall, a tension stress < 2 MPa can take place regardless of the trend of the major horizontal stress.
**Transient stress state at fictitious tunnel surface**

Transient stress state refers to a situation when the excavation face passes the observation point. When the tunnel is excavated by the drill and blast method, as in Pyhäsalmi, all points do not follow the complete transient stress path presented in Figures 8-3 to 8-5, instead they are shifted rapidly from one transient state to another. But, within one cut length, the union of stress paths of points in a line parallel to the tunnel axis forms a complete stress path.

The maximum transient deviatory stresses take place at the tunnel wall when the major horizontal stress is at 45° to the tunnel axis (Figure 8-5). The maximum principal stress is 310 MPa to 360 MPa, being 4.7 times the major horizontal stress, and the minor principal stress is about 30 MPa. There are remarkable transient tensile stresses.

![Diagram showing stress envelopes and lines with different cumulative probabilities](image)

**Figure 8-3.** Principal stress path at the tunnel roof surface using failure criteria presented by Read et al. (1998), Level +1350, $\sigma_H = 75$ MPa.
Figure 8-4. Principal stress path at the tunnel edge surface based on the failure criteria presented by Read et al. (1998), Level +1350, $\sigma_H = 75$ MPa.

Figure 8-5. Principal stress path at the tunnel wall surface with failure criteria presented by Read et al. (1998), Level +1350, $\sigma_H = 75$ MPa.
**Stress state at 15 cm depth from tunnel surface**

Apart from the maximum secondary stresses at the tunnel roof, the highest stresses at 15 cm distance from tunnel surface are remarkably lower than at the surface (Figures 8-6 to 8-8). The transient stresses, in particular, decrease 40% to 50% from the surface values, which means that the highest transient stresses take place close to the tunnel face and to a very limited extent.

![Diagram showing stress states](image)

**Figure 8-6.** Principal stress path at 15 cm distance from the tunnel roof surface by using the failure criteria presented by Read et al. (1998), Level +1350, $\sigma_H = 75$ MPa.
Figure 8-7. Calculated principal stress path at 15 cm distance from the tunnel edge surface based on the failure criteria by Read et al. (1998), Level +1350, $\sigma_H = 75$ MPa.

Figure 8-8. Principal stress path at 15 cm distance from the tunnel wall surface using the failure criteria by Read et al. (1998), Level +1350, $\sigma_H = 75$ MPa.
**Failure criteria comparison**

The stress paths, obtained by using the theories by Read *et al.* (1998), Hoek-Brown and peak strength criterion, are compared with the observed stress failures (Figures 6-5, 6-6, 6-7, 8-3, 8-9 and 8-10). It can be seen that the criterion of the peak strength overestimates the *in situ* strength, but the other two criteria present reasonable results. Therefore, the peak strength is rejected from further studies.

Based on the comparison of the mapped stress failure data with the stress failure criterion on elastic stresses, the following assumptions were made:

a) probability of visual stress failure ($J_2...J_4$) was defined, based on final secondary stress state,

b) probability of noise stress failure ($J_1$) was defined, based on transient stress state,

c) probability for noise stress failure is taken into account only if it is higher than the probability for visual stress failure,

d) yield strength reduction was ignored for the Read *et al.* (1998) criteria, because the transient phase is valid only for limited discontinuous areas; that is, the current and previous areas around the tunnel face having spacing equal to the cut length, and,

e) finally, the comparison is based on the union of the roof, corner, and wall stress failure probabilities where higher stress failure class dominates (Figure 8-11).

The comparison of the resulting failure probability presented in Figures 8-11 to 8-15 with mapping results (Figures 6-5 to 6-7) shows that the Read *et al.* (1998) failure criterion overestimates the strength, whereas Hoek-Brown underestimates it. At 15 cm distance from the tunnel surface, the Hoek-Brown results are more reasonable, but the high stresses at the upper edge dominate the stress failure behaviour in the excavation orientations $30^\circ / 210^\circ$ (Figure 8-11b). However, the stress redistribution after stress failure is not taken into account, which would increase the stress failure potential at 15 cm depth.

Based on elastic stresses and homogenous material, the *in situ* failure criterion for the tunnel seems to be between the strength envelopes presented by Read *et al.* (1998) and Hoek-Brown (Hoek & Brown, 1997). It must be noted that the degree of anisotropy of the Pyhäälmi volcanite is unknown nor was it taken into account in the simulations.
Figure 8-9. Principal stress path at the tunnel roof surface with peak strength envelopes. Level +1350, $\sigma_H = 75$ MPa.

Figure 8-10. Principal stress path at the tunnel roof surface by using the Hoek-Brown failure criteria (Hoek & Brown, 1980). Level +1350, $\sigma_H = 75$ MPa.
a) Probability of stress failure at tunnel roof with tunnel excavation direction - 1350 level, sH=75 MPa, 1 cm, Read et al.

Trend of the major principal stress

Yielded, J2 ... J4
Damaged, J1
Elastic, JO

b) Probability of stress failure in upper corner of tunnel with tunnel excavation direction - 1350 level, sH=75 MPa, 1 cm, Read et al.

Trend of the major principal stress

Yielded, J2 ... J4
Damaged, J1
Elastic, JO

Figure 8-11. Probability of stress failure at the tunnel a) roof, b) upper edge, and ...(continues on the next page)
Figure 8-11 continues  
c) wall and d) union for the tunnel periphery based on failure criteria presented by Read et al. (1998), Level +1350, $\sigma_H = 75$ MPa.
Figure 8-12. Probability of stress failure at the tunnel periphery based on failure criteria presented by Read et al. (1998) (a) and Hoek-Brown (Hoek & Brown, 1980) failure criteria (b). Level +1350, $\sigma_H = 75$ MPa.
Probability of stress failure at tunnel periphery with tunnel excavation direction  
- 1350 level, $s_H = 75$ MPa, 15 cm, Read et al.

Figure 8-13. Probability of stress failure at 15 cm depth from tunnel periphery based on failure criteria presented by Read et al. (1998) (a) and Hoek-Brown (Hoek & Brown, 1980) failure criteria (b). Level +1350, $s_H = 75$ MPa.
Figure 8-14. Probability of stress failure at the tunnel periphery based on the failure criteria presented by Read et al. (1998) (a) and Hoek-Brown (Hoek & Brown, 1980) failure criteria (b). Level +1125, $\sigma_H = 65$ MPa.
Figure 8-15. Probability of stress failure at 15 cm depth from tunnel periphery based on the failure criteria presented by Read et al. (1998) (a) and Hoek-Brown (Hoek & Brown, 1980) failure criteria (b). Level +1125, $\sigma_H = 65$ MPa.
Effect of strength anisotropy

A simple analysis of strength anisotropy was made based on the elastic simulation results. The shear capacity on the average foliation plane (dip 60°, dd 185°) was calculated for exaction directions of 30°, 300°, 330°, and, 360° (Figure 8-16). The results showed that the shear failure potential on the foliation plane is highest when the tunnel is excavated along the trend of foliation and lowest when the excavation direction is 60° from the trend of foliation. Thus, it can be assumed that the effect of anisotropy on the probability of stress failure is similar, bringing the calculated and observed probabilities closer to each other (Figure 8-17).
Figure 8-16. Shear failure potential on the foliation plane (dip 60, dd 185) with different excavation directions, red = highest potential and blue = lowest potential. Trend of major horizontal stress is 120° / 300° (see also Figure 8-17).
Figure 8-17. The conceptual effect of stress/strength anisotropy on different stress failure probabilities of Pyhäsalmi volcanite. The yellow arrows and lines show that the anisotropy increases the yield probability.

8.3 Shafts

The definitions used for secondary and transient state of stress are given in the previous Chapter 8.2. Unlike the tunnel excavation, every point on the shaft wall goes through the complete transient stress path because of raise boring which is a continuous excavation method.

Secondary stress state at shaft wall

The highest compressive stresses take place on the sides perpendicular to the trend of the major horizontal stress (Figures 8-18 and 8-19). The maximum values are 160 MPa to 190 MPa being 2.5 times the maximum in situ stress. The minor principal stress is zero with maximum compression, but, on the side of the major horizontal stress trend, a relatively high tension of 8 MPa to 10 MPa exists.
**Transient stress state at fictitious tunnel surface**

The maximum compressive stress increases almost monotonically at the side perpendicular to the maximum horizontal stress, but on the side parallel to the maximum horizontal stress the maximum transient stress is 115 MPa to 135 MPa being 1.8 times the maximum horizontal stress (Figures 8-18 and 8-19). The stress failure potential is high at the maximum transient stress because the minor principal stress is close to zero.

**Stress state at 15 cm depth from shaft wall**

At a depth of 15 cm from shaft wall, the maximum compressive secondary stresses are 20% and the transient 45% lower (Figures 8-20 and 8-21). The minor principal stress is 5 MPa to 15 MPa at maximum compression and no tensile stresses exist.
Figure 8-18. Principal stress path at the shaft wall surface with failure criteria presented by Read et al. (1998), Level +1350, $\sigma_H = 75$ MPa.

Figure 8-19. Principal stress path at the shaft wall surface with failure criteria presented by Read et al. (1998), Level +1125, $\sigma_H = 65$ MPa.
Figure 8-20. Principal stress path at 15 cm from the shaft wall surface with failure criteria presented by Read et al. (1998), Level +1350, $\sigma_H = 75$ MPa.

Figure 8-21. Principal stress path at 15 cm from the shaft wall surface with failure criteria presented by Read et al. (1998), Level +1125, $\sigma_H = 65$ MPa.
**Failure criterion**

A comparison of the stress paths with the Read *et al.* (1998), Hoek-Brown (Hoek & Brown, 1997 and 1980) and peak strength criteria to the systematic visual stress failure observations shows that the peak strength overestimates the *in situ* strength. The other two criteria present reasonable results (Chapter 6.2). Therefore, the peak strength analysis is rejected from further studies.

The interpretation of elastic stresses and failure criteria was based on the same assumptions as in the tunnel case (Chapter 8.2). The comparison of results of failure probability, presented in Figures 8-22 and 8-23, with visual observations (Chapter 6.2) leads to almost the same conclusions as in the tunnel case. The failure criterion presented by Read *et al.* (1998) overestimates the strength, but the Hoek-Brown method presents quite good estimates. At a depth of 15 cm from the tunnel surface, the Hoek-Brown results are more reasonable (Figures 8-24 and 8-25). On the other hand, the stress redistribution after stress failure is not taken into account, which would increase the stress failure potential at the 15 cm depth.

Based on the elastic stresses and homogenous material, the *in situ* failure criterion for the tunnel seems to be close to the Hoek-Brown (Hoek & Brown, 1997) strength envelope, but the criterion presented by Read *et al.* (1998) also give reasonable estimates. It must be remembered that the degree of anisotropy of Pyhäsalmi volcanite is unknown nor was it taken account in simulations.
Probability of stress failure at shaft periphery with angular orientation - 1 cm from surface, +1350 level, $\sigma_H = 75$ MPa, Read et al.

**Figure 8-22.** Probability of stress failure at the shaft wall based on failure criteria presented by Read et al. (1998) (a) and Hoek-Brown (Hoek & Brown, 1980) failure criteria (b). Level +1350, $\sigma_H = 75$ MPa.
Figure 8-23. Probability of stress failure at the shaft wall based on failure criteria presented by Read et al. (1998) (a) and Hoek-Brown (Hoek & Brown, 1980) failure criteria (b). Level +1125, $\sigma_{H} = 65$ MPa.
Figure 8-24. Probability of stress failure at 15 cm from the shaft wall based on failure criteria presented by Read et al. (1998) (a) and Hoek-Brown (Hoek & Brown, 1980) failure criteria (b). Level +1350, $\sigma_H = 75$ MPa.
Figure 8-25. Probability of stress failure at 15 cm from the shaft wall based on failure criteria presented by Read et al. (1998) (a) and Hoek-Brown (Hoek & Brown, 1980) failure criteria (b). Level +1125, $\sigma_H = 65$ MPa.
Effect of anisotropy

Based on the elastic simulation results, a simple analysis of the strength anisotropy was made by calculating shear capacity on the average foliation planes dipping 60°±20° with a dip direction of 185°±20° (Figure 8-26 and 8-27). The results showed increasing stress failure potential with as the dip became steeper, which can be understood by decreasing normal stress when the normal of the plane of anisotropy converges with the normal of the shaft surface. Another finding is that the highest shear failure potential on the anisotropy plane is always on the side of the dip direction or opposite to it. In the case of Pyhäsalmi, this means that on the north or west side of the ore body the stress failure locations on the shaft walls are rotated counter-clockwise, that is, the orientation of the major in situ horizontal stress is not perpendicular to the failure apex orientation but to some extent is clockwise from it. This can explain the difference between the stress failure observations in orientations 10°-30°/190°-210° and the measured major in situ horizontal stress orientation of 130°/310°.
Figure 8-26. Shear failure potential on the foliation plane with three different dip values.
Figure 8-27. Shear failure potential on the foliation plane with three different dip direction values.
9 DISCUSSION AND CONCLUSIONS

Aim of this study

The objective of this work was to collect stress failure information from known geological and rock mechanical conditions and analyse this data in order to establish a mapping system, understand factors affecting stress failures, and develop stress failure prediction methods. If the method is found to be usable, it could be utilised for further investigations at the Olkiluoto site, which is chosen as a potential site for the Posiva Oy nuclear waste repository. The Pyhäsalmi volcanites, where most of the mappings were obtained, is found to be suitable for this purpose being schistose, metamorphic, sparsely jointed, and heterogeneous like the Olkiluoto mica gneiss. Also the stress - strength ratio of the rockmass is equal to that of Olkiluoto.

Data collection and management

The stress failure mapping system, that was developed, was found to be usable, but the complex geology and massive tunnel driving limited the mapping of all the details. In particular, the location and volume of visual stress failures were difficult to measure from the tunnel surface, which is excavated by the drill and blast method. The detailed measuring of noise type of stress failures was especially problematic. Furthermore, the stress failure classification has too many classes or they are hard to define. However, despite the tight excavation and support timetable, a total of 1700 m of tunnel out of the total excavation length of 2600 m was mapped. The effect of the excavation on the failures is excluded from the analyses.

During the analyses of the stress failure data, it was found that the tunnel driving direction and any changes in tunnel profile should be recorded during the mapping time, because the order of excavation steps in the cross-section areas is difficult to define afterwards.

A continuous centerline for each tunnel should be defined. This would help to combine the data from different logging periods in the final analyses. This is especially important in the case of extensive and complex tunnel networks, as in the Pyhäsalmi mine.

A geological survey program, “Surpac2000”, with connection of the database was used for management, synchronising and, visualisation of the data. The mapped tunnels were treated as boreholes and the data was recorded either as “from-to” or “point” -values.
**Geology**

The majority, 72%, of the mapping data is from tunnels in volcanites below the +1200 level. The Pyhäsalmi volcanite consists of felsic and mafic volcanite, but the implementations are various, pegmatite veins and lenses being common. Most typical is slightly or moderately foliated volcanite where mafic and felsic volcanites are densely alternating. The average grain size is below one millimeter and no alteration exists. The volcanites are sparsely jointed, the average RQD value is 87, and only one joint set along the foliation exists ($J_n=2$).

**Analyses of stress failure data**

Although, the mapping data includes a total of 1700 m of tunnel that turned out to be too little for statistical analyses. This is due the anisotropic and heterogenic geological nature of volcanites in relation to various tunnel excavation directions and orientation of the *in situ* state of stress.

The analyses were focused on finding factors correlating with stress failure observations. Among the studied factors were the rock type, *in situ* state of stress, direction of excavation, size and shape of excavation, orientation of foliation, geological structures, rock fabric, and rock mass quality. Because many factors are dependent on others, only those having an effect on the others were found.

In most cases, the location or length of the stress failure on the tunnel cross-section could not be defined. Therefore, the length of stress failure was assumed to be equal to the mapping length and the location was ignored. The majority of stress failure observations were in the classes of none (J0), noise only (J1) or light spalling (J3).

The following general trends were observed:

<table>
<thead>
<tr>
<th>Rock type</th>
<th>Occurrence</th>
<th>Stress failure type</th>
</tr>
</thead>
<tbody>
<tr>
<td>volcanite</td>
<td>33%</td>
<td>none, J0</td>
</tr>
<tr>
<td></td>
<td>67%</td>
<td>noise or minor spalling, J1...J4</td>
</tr>
<tr>
<td></td>
<td>33%</td>
<td>minor spalling J3...J4</td>
</tr>
<tr>
<td>pegmatite</td>
<td>25%</td>
<td>none, J0</td>
</tr>
<tr>
<td></td>
<td>75%</td>
<td>noise or minor spalling, J1...J4</td>
</tr>
<tr>
<td></td>
<td>25%</td>
<td>minor spalling J3...J4</td>
</tr>
<tr>
<td>ore</td>
<td>45%</td>
<td>none, J0</td>
</tr>
<tr>
<td></td>
<td>55%</td>
<td>noise or minor spalling, J1...J4</td>
</tr>
<tr>
<td></td>
<td>5%</td>
<td>minor spalling J3...J4</td>
</tr>
<tr>
<td>volcanite contact</td>
<td>40%</td>
<td>minor spalling J3...J4</td>
</tr>
<tr>
<td>Time between mapping and excavation</td>
<td>Occurrence</td>
<td>Stress failure type in volcanite</td>
</tr>
<tr>
<td>----------------------------------</td>
<td>------------</td>
<td>----------------------------------</td>
</tr>
<tr>
<td>&lt; 1 day</td>
<td>20%</td>
<td>none, J0</td>
</tr>
<tr>
<td></td>
<td>80%</td>
<td>noise or minor spalling, J1...J4</td>
</tr>
<tr>
<td></td>
<td>40%</td>
<td>minor spalling J3...J4</td>
</tr>
<tr>
<td>2 to 3 days</td>
<td>40%</td>
<td>none, J0</td>
</tr>
<tr>
<td></td>
<td>60%</td>
<td>noise or minor spalling, J1...J4</td>
</tr>
<tr>
<td></td>
<td>30%</td>
<td>minor spalling J3...J4</td>
</tr>
<tr>
<td>&gt; 3 days</td>
<td>55%</td>
<td>none, J0</td>
</tr>
<tr>
<td></td>
<td>45%</td>
<td>noise or minor spalling, J1...J4</td>
</tr>
<tr>
<td></td>
<td>20%</td>
<td>minor spalling J3...J4</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Depth level</th>
<th>Occurrence</th>
<th>Stress failure type in volcanite</th>
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</thead>
<tbody>
<tr>
<td>&lt; 1200</td>
<td>50%</td>
<td>none, J0</td>
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<td></td>
<td>50%</td>
<td>noise or minor spalling, J1...J4</td>
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<tr>
<td></td>
<td>20%</td>
<td>minor spalling J3...J4</td>
</tr>
<tr>
<td>&gt; 1300</td>
<td>30%</td>
<td>none, J0</td>
</tr>
<tr>
<td></td>
<td>70%</td>
<td>noise or minor spalling, J1...J4</td>
</tr>
<tr>
<td></td>
<td>55%</td>
<td>minor spalling J3...J4</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Tunnel shape</th>
<th>Occurrence</th>
<th>Stress failure type in volcanite</th>
</tr>
</thead>
<tbody>
<tr>
<td>Straight or curved tunnel</td>
<td>30%</td>
<td>none, J0</td>
</tr>
<tr>
<td></td>
<td>70%</td>
<td>noise or minor spalling, J1...J4</td>
</tr>
<tr>
<td></td>
<td>35%</td>
<td>minor spalling J3...J4</td>
</tr>
<tr>
<td>Crossing or wider profile</td>
<td>10%</td>
<td>none, J0</td>
</tr>
<tr>
<td></td>
<td>90%</td>
<td>noise or minor spalling, J1...J4</td>
</tr>
<tr>
<td></td>
<td>50%</td>
<td>minor spalling J3...J4</td>
</tr>
</tbody>
</table>

The effects of the orientation of major horizontal stress and orientation of foliation compared to the tunnel excavation direction could not be studied independently. In Pyhäsalmi, the trend of the major horizontal stress is 120°/310°. The average dip of the foliation plane is 64° and its direction 185°. Based on that, the following conclusions could be drawn:
Any geological structure such as a vein or fracture, which differs from the normal foliation of volcanite, increases the stress failure probability by 20%. Among the other factors, the dip of the foliation and rock mass quality did not have an unambiguous correlation with the stress failures. The fabric does not correlate with the failures. Grain size and weathering could not be studied because no variation on these parameters was found.

During the first mapping period, there was no rock mechanical mapping in the raise-bored shafts, but some visual observations were made from above and below the tunnels. In all cases, the stress failures were systematic and the predicted failure directions were 10°-30° and 190°-210°. In the 3.1 m diameter shaft, the width of the failure area was about one metre and the depth was several dozen centimetres.

The magnitude and orientation of the major in situ principal stress and the anisotropy proved to be the most important areal factors affecting the stress failure probability. The most unfavourable excavation directions are perpendicular to the major principal stress and along the trend of anisotropy.

**Strength and deformation parameter values of volcanites**

In the tunnel scale, the layering of mafic and felsic volcanites is dense. The laboratory test data for both types of volcanites was combined, and the average deformation parameters and statistical strength envelopes were defined and used in the analyses. No reliable test results of anisotropy of the mafic/felsic volcanite composite were available for this study.

**Failure criteria**

The possibility of estimating stress failures based on the elastic stresses was studied for both the tunnel and shaft using three-dimensional boundary element simulations. The calculated elastic stresses were compared with three different statistical failure
envelopes, which were 1) the crack damage strength envelope by Read et al. (1998), 2) the Hoek-Brown rock mass strength envelope (Hoek & Brown, 1997 and 1980), and, 3) the Hoek-Brown intact rock peak strength envelope. The results showed that the peak strength envelope overestimates the in situ strength, but the other two methods result in reasonable failure probabilities. The stress failure observation, where the excavation along the trend of anisotropy is unfavourable, was supported by a fast and simple study of the effect of strength anisotropy.

The more detailed qualification of failure criteria was not possible, because there were no measured values of the extent of the failure available. Moreover, the degree of deformation and strength anisotropy of Pyhäsalmi volcanite was unknown. Based on the tunnel observations, the anisotropy is formed by dense layering of mafic and felsic volcanites, not from internal anisotropy of one of these rock types.

The simple shaft stability study with strength anisotropy showed that the trend of the major horizontal stress could be clockwise from the orientation assumed by the orientation of the stress failure observations. This result is supported by the stress measurement results.
10 RECOMMENDATIONS

The lack of data was often found to be problematic in the statistical analyses. The mapping has continued and the additional data should improve the reliability of the analyses presented and the results achieved.

A more detailed qualification of the failure criteria involves more accurate data on the quality and extent of the stress failure. Quantifying the extent of the stress from the surfaces of the tunnels, which were excavated by the drill and blast method, was found to be difficult. The second mapping phase will also include data from raise-bored shafts where the extent of the stress failure can easily be seen and measured from pre-known diameters. Furthermore, the geological and orientational conditions are more stable around the shafts.

The stress failure noise is originated from micro seismic events and can be localised and quantified using a 3-D micro-seismic monitoring system. On the other hand, the expected accuracy of the micro-seismic monitoring involves a limited monitoring area on a scale of a few dozen metres.

The anisotropy of the rock strength seems to be an important parameter. Further study should involve laboratory analysis of the strength and deformation anisotropy induced by the dense layering of mafic and felsic volcanites.

Minor improvements should be made before proceeding further with the stress failure mapping. The most important thing for the analyses is the need for flags for profile changes, cross section areas, tunnel face, geological structure, and direction of excavation. All of these can be defined from the current mapping data and survey data, but it is very laborious.
REFERENCES


Hutchinson, D. & Diederichs, M., 1996. Cable bolting in underground mines. Richmond, Canada, BiTech Publisher.


Mononen, S., 2000. CSIRO HI rock stress measurements at Pyhäsalmi mine
Unpublished work report, Laboratory of Rock Engineering, Helsinki University of Technology (in Finnish).


