Prediction of stress-driven rock mass damage in spent nuclear fuel repositories in hard crystalline rock and in deep underground mines

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Abstract
Nuclear plants have existed since the 1950s, and they provide 11% of the world’s electricity production. Worldwide, 30 countries are operating 448 nuclear reactors for electricity generation, and 57 new nuclear plants are under construction in 15 countries. Measured by deaths per terawatt hour, nuclear power is the safest method to provide energy, but it does produce a range of radioactive waste, which must be disposed of safely and responsibly. The deep geological repository is currently the only acceptable long-term solution for high-level nuclear waste.

The two most common causes of rock mass failure are structurally controlled gravity-driven failure and stress-induced failure. Usually, surface and near-surface rock excavations are subject to structurally controlled gravity-driven problems, but in deep rock spaces, the in-situ stress of the rock mass increases and the risk of stress-driven problems grows. The five most common stress-driven damage mechanisms are i) rockburst, ii) spalling, iii) convergence, iv) shearing and v) seismic. Excessive convergence is rarely a problem in hard, massive rock mass. In this thesis, the remaining four mechanisms are addressed.

The goals of the research were to discover the damage-reducing capability of thin sprayed concrete liners, to define the strength of long rock joints, and to develop a real-time risk management concept. Numerical modelling was used to design an in-situ concrete spalling experiment, the ICSE. Laboratory scale mortar rock joint replicas were used to study the scale effect, and large 2.00 m by 0.95 m (ASPERT) and 0.50 m by 0.25 m rock joints were sheared to validate the methods. A new real-time formulation of the Geotechnical Risk Management (GRTM) concept was studied using both example cases and case data. New methods were developed for the photogrammetric capture of rock joint surfaces and shear testing of large rock samples.

The numerical modelling predictions for the in-situ experiment show that the thin concrete liner produces up to 3 MPa of support pressure and using polyaxial Ottosen criterion the liner is not damaged during the heating stage. Both the replica shear tests and the large shear tests results show a weak negative scale effect. Based on the initial analyses using example data, Bayesian networks appear compatible with the Observational Method, and the approach is ready to be tested using real data.

The three main conclusions each address the stress-driven damage prediction and mitigation. The stress-driven damage can be reduced using support pressure generated by thin concrete liners. A new method was developed to capture rock joint geometry using photogrammetry and to manufacture mortar replicas for laboratory scale shear testing. The use of Bayesian networks, together with the real-time geotechnical risk management concept, was demonstrated. The results contribute towards predicting stress-driven damage in deep underground spaces.

Keywords spalling, rockburst, photogrammetry, shear test, risk management


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Tutkimuksen pääätavoitteena on selvittää ohuiden ruiskubetonikuurien jännitysvaurioita vähentävän vaikutuksen suuruus, määrittää pitkien rakojen mekaaniset ominaisuudet ja kehitää reaaliaikainen riskienhallintakonsepti. Numeerista mallintamista käytettiin betoniin hilseilyn in situ -kokeen (ICSE) suunnittelemiseksi. Laboratoriomittakaavan rakojen laatijäljenteitä käytettiin mittakaavaa vaikutuksen tutkimiseksi, ja ison mittakaavan 2,00 m kertaa 0,95 m (ASPER) sekä 0,50 m kertaa 0,25 m rakonäytteet leikkauskurmiitettiin menetelmien varventamiseksi. Uutta reaaliaikasta geotekniistä riskienhallintakonseptia (GRM) tutkittiin esimerkkitapauksia sekä tapauskohtia käyttäen. Uusia menetelmiä kehitettiin kallion rapotjoen fotogrammetriseen tallentamiseen ja suurten rakonäytteiden leikkauskoestamiseen.

Numeerisen mallinnuksen ennestet ICSElle osoittavat, että ohut betonikuori tuottaa enintään 3 MPa tukipaineen, ja käyttämällä moniaksiaalista Ottosen murtokriteeria betonikuori ei vaurioi lämitysvaiheen aikana. Sekä replikaileikkauskoesarjassa että isommissa leikkauskokeissa havaittiin negatiivinen mittakaavaa vikutus. Esimerkiksi 60 astuvien analyysien perusteella Bayes-verkon vaikuttavat yhteen sopiva seurantamenetelmän kanssa, ja lähestymistapa on valmis testattavaksi todellisella datalla.


Avainsanat
hilseily, kallioriaske, fotogrammetria, leikkauskoe, riskienhallinta

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Preface

I am excited to deliver to you this doctoral thesis, which I hope you will find an interesting read and useful for your research. My passion is discovering new things, and I enjoy teaching, so the academic life has been a lifelong dream for me. Over the last few years, I have been living my dream with research projects of my own, delivering rock mechanics lectures and instructing student theses. With this dissertation, I hope to convey what I have been up to lately.

After graduation as a Master of Science from Helsinki University of Technology, I worked for three years as a rock mechanics consultant for Rockplan. I had the opportunity to work with demanding megaprojects, such as the Pyhäsalmi mine’s 1.4 km deep underground neutrino detector, LAGUNA; the Länsimetro west extension (21 km long and incorporating 12 stations); and the ONKALO underground rock characterization facility. The latter would later become a part of the world’s first underground repository for spent nuclear fuel. With financial support from Rockplan, I was able to start my Ph.D. as a part-time doctoral candidate at Aalto University, and I wrote my first research publications soon after.

Later, an opportunity presented itself to apply to be a full-time doctoral candidate, and I took a leap of faith. I only had partial funding, so a major part of the early research work was to secure a more continuously funded research position. During my time at Aalto, I wrote a total of 29 research proposals. In addition to successfully funding my doctoral research, I also helped Aalto University to secure more than a million euros of research funding that was externally competed for.

During my doctoral research, I participated in research activities in several major research projects, such as I2Mine (the Innovative Technologies and Concepts for the Intelligent Deep Mine of the Future), DynaMine (Dynamic Control of Underground Mining Operations), KARMO I–III (Mechanical Properties of Rock Joints), the SCC (Solar Community Concept), ORMID (Online Risk Management in Deep Mines), Fractuscan (Photogrammetric surface roughness profiler) and MIEDU (Mining Education and Virtual Underground Rock Laboratory). In five of these, I was the main author of the research proposal and held the research leadership role. In total, I published 26 research papers, instructed 16 theses and contributed towards 8 commercialisation projects.

One of the great things about embarking on the Ph.D. journey is the opportunity to travel a lot and to meet interesting people from all around the world. I attended a total of 12 scientific conferences. For my conference publications, I
received one best paper by young person award and two best poster awards. As part of my researcher mobility, I spent a more than a three-month-long researcher exchange at the KTH Royal Institute of Technology, and this cooperation continues today. International collaboration was also the route to enter the global scientific research society.

The International Society of Rock Mechanics and Rock Engineering (ISRM) played a major role in my research. In the early stages of my research, I became the Finnish representative in the YMPG (Young Members Presidential Group). This position put me in contact with my young colleagues in other countries. Together we were able to improve the position of young rock mechanics experts. In Finland, I entered the Finnish national group (NG) of the ISRM, and I was able to improve Finnish student mobility, and launched a new biennial award for rock mechanics students. Today, I am the president of the Finnish NG of the ISRM and the chairman of the Finnish Association for Structural Mechanics.

For Finnish society, I worked as the coordinator of the national course on nuclear waste management training for two consecutive years. I have participated as the Finnish representative in the European Union CEN/TC104/WG 10, which is focused on the renewal of the sprayed concrete design codes. During my research, I also participated in the Finnish Concrete Union workgroup BY NT-120 in order to update the Finnish sprayed concrete norms which were later published as the book *BY 63 Finnish Sprayed Concrete Guidelines*.

For this thesis, I have selected seven publications. I want you to be aware of the unsolved problems we are facing in deep underground rock spaces. One cause of these unsolved problems is the high stress which can cause abrupt rock mass failure. Predicting this failure is crucial in both damage-avoiding and damage-mitigating approaches. For jointed rock masses, the rock joint parameters dominate the rock mass strength. The parametrisation of fractures is challenging, and large-scale laboratory direct shear tests may be the next best thing. Finally, assuming that the mechanical parameters of rock joints can be obtained, the geotechnical risk related to stress-driven rock mass damage must be managed. The amount of data available can range from being scarce at the beginning of the project to being abundant as the project is nearly completed. In my thesis, I have chosen to address these problems and present both new methods and test results for the continuation of the research.

After having compiled the thesis, I am pleased with how it has turned out. I was initially concerned that a thesis compiled from the results of several research projects might lack coherence. However, it turned out I was solving the same stress-driven problem area in each of the research projects – it was just a matter of different perspectives. I will close with a Vorlon quote by J. Michael Straczynski: “*Understanding is a three-edged sword: your side, their side, and the truth.*”

Espoo, 9th April 2018
Lauri Kalle Tapio Uotinen
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During my doctoral studies, the rock mechanics research staff moved several times, and I had the opportunity to work in four different rock mechanics laboratories, including two open office environments, the Article Factory and CORE (Center of Rock Engineering). I had many great moments with my colleagues Zhen Song, Topias Siren, Mateusz Janiszewski, Ritesh Mishra, Tuomo Hänninen, Pekka Kantia, Risto Kiuru, Jyrki Oraskari, Eeva Huuskonen-Snicker, Tiina-Liisa Toivanen, Zhijie Wen, Zhou Pin, Mashuqor Rahman, Ari Hartikainen, Jorma Palmén, Tero Hokkanen, Marjo Sairanen, Teemu Ojala, Klaus Viljanen, Steven Collins, Paula Barbens Torres, Mikael Siirtola, Dong Shichao, Johannes Suikkkanen, Esa Koskinen, Harm Oosterbaan, Martyna Szydlowska, Magdalena Dzugala, Joni Sirkiä, Antti Matikainen, Juha Pennala, Matias Napari, Markus Napari, Eero Korpi, Raphaël Yorke, Teemu Taajaranta, Janina Korkalainen, Stepan Kodeda, Frans Ritala, Pauliina Kallio, Laura Tolvanen, Joni Sirkiä, Sivi Kivivirta, Henri Munukka, Susanna Mikkola, Enrique Caballero and Jakub Jastrzebski.
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I want to thank my colleagues in the ISRM Young Members Presidential Group. It has been a pleasure to cooperate with you and I always had fun times at conferences. I want to thank Xia-Ting Feng for starting the YMPG and David Beck for helping the group to organize itself. Especially I want to thank my Fennoscandian colleagues Øyvind Dammyr (Norway) and Henrik Ittner (Sweden) who helped me create the Fennoscandian Geotechnics Newsletter initiative. I also want to thank the ISRM Commission on Design Methodology for warmly welcoming the new young members: John Hudson, Xia-Ting Feng, Wulf Schubert, Resat Ulusay, Ove Stephansson, Leslie G. Tham, Erik Johansson, Claus Erichsen, Alexandron Sofianos, Thierry You Dr Congrad Felice, Zhou Yingxin, Antonio Samaniego, Christopher Vibert, Philippe Vaskou, M. Sharifzadeh, Eda Freitas de Quadros and Seokwon Jeon. I look forward to seeing you again at the next meeting.

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I dedicate this thesis to my two sons Leevi Uotinen and Luukas Uotinen. Stay persistent and you will reach your goals.

In memory of Mircea Cozma and Stepan Kodeda. I will always remember you.
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List of abbreviations

APSE  Äspö Pillar Stability Experiment
ASPERT  Aalto Shear Pull Experiment for Rock Tensile Fracture
AziSA  Architecture for underground measurement and control network
BN  Bayesian network
DDM  Displacement Discontinuity Method
Digmine  Dynamic Intelligent Ground Monitoring
DynaMine  Dynamic Control of Underground Mining Operations
EDZ  Excavation damaged zone
FN  Frequency to number (of events)
Fractuscan  Photogrammetric surface roughness profiler
HLNW  High-level nuclear waste
I²-Mine  The Innovative Technologies and Concepts for the Intelligent Deep Mine of the Future research project
IAEA  International Atomic Energy Agency
ICSE  In situ concrete spalling experiment
KARMO  Mechanical parameters of rock joints
KBS-3H  The horizontal variant of the KBS disposal concept for HLNW
KBS-3V  The vertical variant of the KBS disposal concept for HLNW
MS-RAP  Mine seismicity risk analysis programme (2000–2013)
MyRAP  Mine seismicity risk analysis programme (2012–)
ONKALO  Posiva’s rock characterization facility
POSE  Posiva’s ONKALO spalling experiment
POSE-EH3  POSE niche, experiment hole 3
PRIS  Power reactor information system
SCC  Solar Community Concept
SKB  Swedish Nuclear Fuel and Waste Management Company
This doctoral dissertation consists of a summary and of the following seven publications which are referred to in the text by their numerals


7. Mishra, Ritesh; Uotinen, Lauri; Rinne, Mikael. 2017. Bayesian network approach for geotechnical risk assessment in underground mines. Manuscript submitted to *Safety Science* on 6th October, 2017 and revised according to pre-examination comments. ISSN 0925-7535 (online)
Author’s contributions

Publication 1: The author coordinated the writing of the article, wrote the majority of the text, carried out the numerical finite element modelling, designed the instrumentation plan, collated the results and drew the conclusions. Siren did the fracture mechanics modelling. Other co-authors commented on the manuscript.

Publication 2: The author wrote the chapter “Concrete Material Properties, and parts of Temperature and Concrete Support Pressure”, carried out the 3D FEM comparison modelling and compared the results of 2D DDM to the 3D FEM. The majority of the text was written by Topias Siren. Other co-authors commented on the manuscript.

Publication 3: The author coordinated the writing of the article, wrote the majority of the text, carried out the finite element fully coupled thermomechanical transient numerical modelling, presented the results and drew the conclusions. Siren did the fracture mechanics modelling and wrote chapters 3.3 and 4.3.

Publication 4: The author coordinated the writing of the article, wrote the majority of the text, took the photographs for the photogrammetry, analysed them, planned the downscaling and replication experiment, and analysed the results. Johansson wrote part of the introduction, and Korpi wrote part of “Replica production”. Other co-authors commented on the manuscript.

Publication 5: The author designed both of the slab shear experiments and the photogrammetric arrangements, supervised the execution of the experiments and analysis of the results. The author wrote the introduction and conclusions chapters. The majority of the text was written by Magdalena Dzugala under the supervision of the author. Other co-authors commented on the manuscript.

Publication 6: The author wrote the sections concerning the preceding research on DynaMine, contributed towards the Event Tree Analysis, and wrote the discussion and conclusions chapters. The majority of the text was written by Ritesh Mishra. Janiszewski wrote the Geotechnical Hazard Potential and Geotechnical Risk Assessment descriptions, Szydlowska wrote parts of the introduction and Geotechnical Risk Management, Siren wrote a part of the introduction, GRM and discussion. Rinne commented on the manuscript.

Publication 7: The author contributed to the calculation examples and wrote the Observational Method chapter and the Conclusions chapter. In addition, the author made numerous smaller contributions. The majority of the text was written by Ritesh Mishra. Rinne commented the manuscript.
 Atomic energy has become an inevitable part of our lives. Nuclear plants have existed since the 1950s, and they provide 11% of world’s electricity production (Gibney, 2015). Thirty countries worldwide are operating 448 nuclear reactors for electricity generation, and 57 new nuclear plants are under construction in 15 countries (IAEA, 2017). Measured by deaths per terawatt hour, nuclear power is the safest method to produce energy, but it does produce a range of radioactive waste, which must be disposed of safely and responsibly.

Nuclear reactors produce high-level nuclear waste (HLNW), which is either reprocessed as nuclear fuel or deposited in repositories. Depending on the waste radioactivity, the spent fuel returns to background radioactivity level within some hundreds of thousands of years. This period may include one or more glaciations. A deep geological repository is currently the only acceptable long-term solution for HLNW. Finland started to build an HLNW repository in 2015, and the first spent nuclear fuel canister is expected to be deposited in 2023 (Figure 1). Sweden plans to start the building of an HLNW repository in 2020 and to deposit the first canister in 2030.

Natural analogies are used as the basis of the safety analyses (Alexander et al., 2015). The bedrock of Olkiluoto, Finland, is 1.8–1.9 billion years old, and it has hard, unweathered and sparsely fractured zones between fracture zones (Anttila et al., 1999). In the Swedish KBS-3 concept, the HLNW is first cooled under water for 40 years, then the waste is encapsulated in a cast iron canister, which is encapsulated in a copper capsule and is then deposited into bentonite clay, in either horizontal holes (KBS-3H) or vertical holes (KBS-3V) (KBS, 1983).

Figure 1. The planned spent nuclear fuel repository at a depth of 420 meters under the Olkiluoto island in Finland. Image: Posiva Image Gallery.
Even after 40 years of cooling, the decay heat from the canisters is substantial (Saanio et al., 2013). The in situ rock stress at the chosen depth is already close to the threshold of observing stress-induced damage. Based on thermal modelling, the decay heat will not be sufficient to cause damage to the technical area or the shafts (Publication 1). The interaction between a 40 mm concrete liner support and the surrounding rock mass is not well known, and an in situ experiment was planned. Both fracture mechanical modelling (Publication 2) and finite element modelling (Publication 3) were used to create blind predictions. The experiment was executed in spring 2017, and the experiment is now in the analysis stage. A prediction-observation study will be carried out once the results become available.

The canister hole locations will be selected in order to avoid hitting natural fractures of the rock mass. Should a canister hole and a fracture intersect, there is a risk of the canister being sheared if the rock mass moves along the fracture more than 50 mm (Börgesson, 1986). Additionally, there is always the risk of non-intersecting fracture in proximity. In this research, we propose a new method to study the mechanical properties of large rock joints using smaller laboratory scale samples (Publication 4). As validation, a 2 m by 0.95 m shear test was carried out on a large rock sample intersected by an artificially induced tensile fracture (Publication 5).

Stress-driven damage is also a major problem in deep mining operations. It may cause overbreak, spalling or strainbursting in the worst case. A geotechnical risk management concept was presented to predict stress-driven damage and to enable the real-time monitoring of risk (Publication 6). Initially, fault tree / event tree approach was used, but using Bayesian networks (BNs) proved to be a more versatile method. BNs can be used to calculate the probability of failure, and also to assist in incident investigations and as a tool in financial decision making related to economic consequences for failure events (Publication 7).

1.1 Research problem

Failures in rock spaces may occur due to a variety of reasons, but the two most common causes are structurally controlled gravity-driven problems and stress-induced problems (Hoek et al., 1995). Usually, surface and near-surface rock excavations are subject to structurally controlled gravity-driven problems, but in deep excavations the in situ stress of the rock mass increases and the risk of stress-driven problems increases. Mazaira and Konicek (2015) define five common stress-driven damage mechanisms: i) rockburst, ii) spalling, iii) convergence, iv) shearing, v) seismic. Excessive convergence is rarely a problem in a hard, massive rock mass. In this thesis, we address four of these five mechanisms. Publications 1, 2, and 3 address the spalling and rockburst types of damage, publications 4 and 5 the shearing of rock joints and publications 6 and 7 the excessive convergence type of stress-driven problems (Figure 2).
There are multiple ways of reducing the risk, such as the use of reinforcement options, tunnel layout optimisation, excavation sequence optimisation, excavation shape optimisation, blasting design or de-stress blasting. To study the damage-reducing effect of reinforcement, the author proposed an in situ experiment. In this thesis, the author designed the in situ experiment using heater elements to increase the local stress field (Publication 1). The stress-driven--damage reducing capacity of a thin concrete liner in a cylindrical hole bored into rock mass was quantified using numerical simulations (Publications 2 and 3). The in situ experiment was carried out in spring 2017 and a prediction-observation follow-up study is planned for when the results become available.

The mechanical properties of long fractures remain unknown. Large in situ experiments are expensive and spatially unrepresentative. Small laboratory experiments are affordable, and enough samples can be taken for sufficient spatial representation. However, a scale effect influences small samples and this must be taken into account during the upscaling of the results. In this thesis, we present a new method for obtaining rock joint geometry (Publication 4) and as validation, a large-scale laboratory shear test (Publication 5).

After the stress-driven load can be calculated and the countermeasures are known, the associated geotechnical risk can be calculated. Currently, the background in situ stress level is measured point-wise and at specific time intervals. New methods are being developed to enable a more comprehensive back calculation and using real-time measurements. Therefore, the risk management concept was modified to be compatible with real-time data (Publication 6). During a rock engineering project or a mining project, the amount of data varies from none, to minimal, to abundant. Most of the methods currently in use require a lot of data for reliable operation, but BNs can operate regardless of the amount of data available (Publication 7).
In conclusion, the explicit research questions addressed in this thesis follow:

1. Can thin concrete liners be used to reduce stress-driven damage?
2. How much counter-pressure can thin concrete liners produce?
3. Can rock joints be digitised with sufficient accuracy to assess the mechanical properties of rock joints?
4. How can the scale effect be taken into account when the small-scale mechanical parameters of rock joints are upscaled for larger rock joints?
5. How should the current risk management concepts be modified to accept real-time stress measurements as input?
6. How does one cope with the lack of measured data in geotechnical risk management?

1.2 Objectives

The overall goals of the research were to discover the stress-induced–damage reducing capability of thin concrete liners, to define the strength of long rock joints using small samples and to develop a risk management concept that is compatible with real-time stress measurements. To achieve these goals, the following objectives were set:

1. Design an in situ experiment to observe the damage-reduction capability of thin concrete liners in cylindrical holes bored in a rock mass.
2. Make blind predictions of the in situ experiment using both fracture mechanics and the thermomechanical finite-element method.
3. Develop a method to capture the geometry of rock joints with sufficient accuracy to determine the mechanical parameters of the joint.
4. Validate the developed roughness-measuring method using a large laboratory shear experiment.
5. Modify the geotechnical risk management concept to accommodate real-time monitoring in deep underground rock spaces.
6. Study the suitability of using BNs in the geotechnical risk-management concept.

1.3 Scope

In this thesis, we are studying the geological disposal of spent nuclear fuel in hard crystalline rock at depths of 400 metres below sea level or deeper. Only the vertical KBS-3V concept (Figure 3) was considered. The safety has been assessed considering only the structural strength of the rock tunnels, the reinforcement structures, and the natural shear strength of rock joints. Rock joint weathering, infillings or poorly mated joint surfaces were not considered. In the geotechnical risk management, only the underground deep mine stability risks were considered.
1.4 State of the art

A threefold state-of-the-art review was carried out for spalling and in situ spalling mitigation experiments, for alternative methods of rock joint geometry capturing, and for the upscaling of rock joint parameters and geotechnical risk assessment methods. The reviews were carried out in the corresponding publications, and the main results are summarised here.

Based on numerical modelling by Hakala et al. (2008), a support pressure in the megapascal range would be needed to suppress spalling damage in cylindrical deposition holes. The Swedish Nuclear Fuel and Waste Management Company (SKB) carried out an in situ spalling test and showed that the range of support pressure for expanded clay pellet filling is $10^{-20}$ kPa (Glamheden et al., 2010). In the Äspö Pillar Stability Experiment (PSE), the first experiment hole was left unconfined, and the second hole was confined with a rubber bladder. In APSE, the spalling could be totally controlled with approximately 150 kPa of confining pressure (Andersson, 2007). A similar study was conducted in the ONKALO rock characterisation facility in the second phase of Posiva’s ONKALO Spalling Experiment (POSE), where the confinement was created using sand filling. The second phase of POSE concluded that the confined hole had more damage compared to the unconfined hole (Johansson et al., 2014). Some possible explanations for this observation are the observed heterogeneity of the rock mass and the sensitivity of the experiment to the direction of the local stress field. The third phase of POSE was conducted in a single hole, which was less sensitive to the in situ stress field direction (Valli et al., 2014).

For the scale effect of rock joint strength, the consensus is that the shear strength decreases when the sample size increases, in other words, there is a negative scale effect (Bandis et al., 1981). Since then, several other studies have also reported a negative scale effect (Murahlha and Pinto de Cunha, 1990; Yoshinaka et al., 1991). Castelli et al. (2001) reported a negative scale effect for low normal stresses and no scale effect at higher stresses. However, some studies have presented contradictory results, for example, Hencher et al. (1993)
and Kutter and Otto (1990). The extent and nature of the scale effect is still a question of debate (Tatone and Grasselli, 2013). Johansson and Stille (2014) have presented a possible explanation for the scale effect using a conceptual model for fresh and unweathered rock joints. Later slab shear tests for 60 mm x 60 mm and 200 mm x 200 mm fresh and unweathered rock joints showed no scale effect (Johansson, 2016).

A comprehensive review of the scale dependency of roughness and shear strength can be found in Tatone and Grasselli (2013). Only a few studies have been carried out on large over 1 m² rock joints (Feng et al., 2003; Fardin et al., 2004; Haneberg, 2007). The largest shear tests have been carried out on 0.5 m² samples by Pratt et al. (1974) and by Muralha and Pinto da Cunha (1990). The largest known shear test for fresh joints was 1 m², conducted by Bakhtar and Barton (1984). Numerical scale effect studies have since been carried out by Bahaaddini et al. (2014) and Wang et al. (2016).

Potvin (2009) described strategies and tactics to control the seismic georisks in mines. Szydlowska (2016) did a systematic review of georisk in underground hard rock mines and concluded that most georisk management approaches focus on seismicity analyses. Szydlowska identifies three major georisk management approaches: the Mine Seismicity Risk Analysis Programme (MS-RAP), Dynamic Intelligent Ground Monitoring (known as Digmine) and AziSA. Hills and Penney (2008) contain an example of the MS-RAP application, used in the calculation of the excavation vulnerability potential of all positions in the existing and designed mining horizons. The work on seismic risk management was continued, and Hudyma and Potvin (2010) introduced an engineering approach to seismic risk management. Kaiser and Cai (2012) presented the principles for rock support in rockburst-prone ground using BurstSupport software. Tonnelier and Buffier (2015) introduced the Digmine real-time stress monitoring concept in I-Mine. The underlying method is described by Feng et al. (2015). The method has been tested in the Garpenberg mine, where instrumentation with CSIRO cells was performed, and based on the obtained data, the analysis was successfully conducted (Souley et al., 2016). AziSA is a set of standards and procedures for georisk management (Stewart et al., 2008). The method is also compatible with real-time data (Vogt et al., 2009).
2 Research methods

For this research, a mixture of different research approaches was needed: first, scoping numerical modelling using the finite element method and fracture mechanics method, then hands-on pilot using photogrammetry, 3D printing, mortar casting and small-scale portable shear box testing. Several scales, ranging from 17 cm to 170 cm, were downscaled to 17 cm and shear tested. Tilt table tests up to a size of 50 cm were carried out. Two larger rock block tests were carried out at sizes of 50 cm by 25 cm and 200 cm by 95 cm, of which the latter is also the world’s largest shear-pull test published in a scientific journal. Additionally, an in situ experiment to be carried out in the ONKALO underground rock characterisation facility was designed and numerically modelled using a transient thermomechanical model with the Ottosen polyaxial strength criterion for the thin concrete liner. Finally, for further development of the geotechnical risk management concept, theoretical model development, along with synthetic benchmarking and comparisons to literature data, was used.

Several new research methods were developed during the research. First, a new type of in situ concrete spalling experiment was proposed (Publication 1). The support pressure was predicted using two different numerical modelling codes and then cross-comparing the 2D and 3D results (Publication 2). For blind prediction, it was necessary to predict the Ottosen polyaxial strength criterion parameters from existing material testing data (Publication 3). For the photogrammetric replication of rock joints (Publication 4), several new methods were developed in photogrammetric workflow, point cloud processing, subsampling and rescaling, digital mould modelling, and mortar casting technology. Publication 5 describes a large rock block pull shear test for a 200 cm by 95 cm artificially induced tensile fracture. It also describes a new type of revolving table type of photogrammetry setup and workflow. Publication 6 first introduces the geotechnical risk concept that is compatible with real-time data and shows practical use cases using fault trees and event trees, and frequency to number of events (FN) diagrams. Finally, the BN approach is used in conjunction with the geotechnical risk assessment (Publication 7).

In conclusion, all of the publications, advance the state of the art. Publications 1, 2 and 3 propose a new type of in situ experiment and contain blind predictions for a prediction-observation type of research. Publications 4 and 5 propose a new type of replication method and show experimental laboratory testing results. Publications 6 and 7 suggest a real-time modification and attempt to implement the BN approach to the geotechnical risk assessment concept.
3 Numerical predictions for ICSE

3.1 Experiment design

The experiment intended to empirically observe the failure strength of shotcrete on a pre-stressed rock surface, when the stress state of both rock and shotcrete are increased by heating (Publication 1). The in situ validation experiment is a part of prediction-observation research concerning the stress-driven damage and damage mitigation using thin concrete liners. The in situ experiment was chosen to include the effects of the in situ stress of the rock mass and to execute the experiment in full scale and under realistic geological conditions. The experiment was carried out in the ONKALO facility in Finland during spring 2017. The experiment was carried out in the niche of Posiva’s Olkiluoto Spalling Experiment (POSE) at a depth of -345 m (Figure 4).

In the preceding third stage experiment of POSE, the third experiment hole was heated from inside to simulate the heat from a single spent nuclear fuel canister. In the subsequent in situ concrete spalling experiment (ICSE), the eight heaters were reused and placed in the surrounding rock mass to simulate the decay heat from the spent nuclear fuel canister panels arriving at the technical area circular shafts (Figure 4).

![Figure 4](image.png)

*Figure 4.* The ONKALO underground rock characterisation facility and the location of the POSE niche, the shafts and the technical area. Modified after the Posiva Image Gallery.
Sprayed concrete is a possible reinforcement method for the technical area (Napari, 2013). The compressive spalling strength of web-thinned concrete cylinders was studied in an experimental pilot test by Taajaranta (2014) using the shape suggested by Jacobsson et al. (2010). Based on thermal modelling, the temperature increase near the shafts is only 4 °C after 120 years and poses no risk to the structures (Publication 1). However, to establish a factor of safety, the load-bearing capacity of the structure must be known.

The ICSE was planned so that the POSE niche’s experiment hole 3 (POSE-EH3) and the eight heaters could be reused. Optimal distance for the heating holes was found iteratively, and the hole pattern was rotated to provide maximum clearance from the already-existing temperature and acoustic emission monitoring holes. The heating power pattern was designed to reach the expected damage strength within nine weeks. If damage was observed, the experiment would enter seven weeks of cooling. Otherwise, the heaters would be set to maximum power until damage was observed. To study the damage reducing capacity of the concrete liner, the top part of the hole was left unlined. Numerical modelling was used to find the optimal height for the lined part. Finally, sensor locations were chosen based on preliminary numerical modelling and predictions for each sensor were calculated and stored for later comparison when the experiment results are published.

3.2 Preliminary analyses

Preliminary numerical modelling analyses were used to speed up the time-consuming fracture mechanics modelling and the memory-intensive, full-scale modelling (Publication 1). Using symmetry, the problem could be reduced to a 1/16th slice (Figure 5). The slice gives accurate predictions of the heating stage stresses for the concrete liner, but it cannot include the magnitude or direction of the in situ stress of the surrounding rock mass.

Figure 5. Location of the 1/16th slice (a), the entire model geometry (b) and a mesh detail showing the insulation, concrete foundation slab, liner and half a heater hole (c) (Publication 1).
As the preliminary model was both fast to compute and memory effective, it could model the small pilot hole (bored before the larger experiment hole), the concrete foundation slab and the air convection inside the tunnel above the experiment. To define the optimal concrete liner element size, a parametric 2D sweep was used to iteratively find a size with a less than 5% error in tangential stress and temperature. For the surrounding rock mass, gradually bigger elements were used except for near the half-heater hole.

As results from the preliminary analyses, it was concluded that the pilot hole, foundation slab and convection could all be omitted with negligible error. For elastic modelling, it was sufficient to model the concrete liner with two layers of quadratic elements for mechanics and use linear elements for temperature. After nine weeks of heating, the following results are obtained: the tangential stress in the concrete will reach 53 MPa, using the Ottosen criterion, no damage in the concrete is expected, the tangential stress in rock mass is 130 MPa and the temperature will peak at 125 °C in the hole wall after nine weeks of heating.

3.3 Fracture mechanics

Fracture mechanics analysis was carried out by Topias Siren at depths of -3 m and -5 m in order to study the fracture growth inhibiting effect of thin concrete liners. The author defined the mechanical parameters and the material model for the concrete. Additionally, the -5 m model was run with two lithological units (Publication 2). Each of the three models was run with two alternative in situ stress interpretations, for a total of six models. The stress data comes from measurements from stress measurements inside the experiment hole and from old measurements near the POSE niche entrance. The 2D equivalent stress field was obtained from 3DEC modelling of the entire area (Hakala and Valli, 2013).

The fracture mechanics modelling predicts butterfly-shaped damage after three weeks of heating in the unsupported top part. For the supported part, the concrete layer is predicted to suffer a shear failure after three weeks and be severely damaged at nine weeks. In both stress interpretations, the presence of a thin concrete liner significantly suppresses the fracture formation in the rock mass (Figure 6). The predicted support pressure is 3.0 MPa at nine weeks. The temperature will reach 134 °C in the hole wall after nine weeks of heating.

![Figure 6. Fracture propagation at -3 m depth after nine weeks of heating without concrete (a, c) and with concrete (b, d) and VT1 (a, b) and ONK-EH3 (c, d) in situ stress interpretations. (Modified after Publication 2.)](image)
3.4 Thermomechanics

Finally, full-scale modelling was carried out in a 100 m x 100 m x 100 rock cube which contained the POSE niche and the experiment area. Based on preliminary modelling, the pilot hole, foundation slab and convection were omitted. The Ottosen damage model was used for the concrete liner to account for any intermediate stress effects and the Mohr-Coulomb model was used for the rock mass surrounding the experiment hole. Fully coupled thermomechanical modelling was used to model a nine-week heating period and then either seven weeks of additional heating or seven weeks of a cooling period. Both in situ interpretations were considered, totalling four numerical models.

The results predict 59 MPa of tangential stress in the concrete after nine weeks and 117 MPa in the rock mass (Figures 7a and 7b). The plastic modelling used a simplified version of the full model with emphasis on the concrete shell. The plastic model predicts damage on Week 10 on the North and South walls (Figure 7c). The concrete liner produces 3.2 MPa of support pressure and reduces the tangential stresses in the rock mass by -30 %. The results are in line with the preliminary predictions and the fracture mechanics predictions.

3.5 Numerical prediction results

The numerical modelling produced several measurable blind predictions. The support pressure generated by the thin concrete liner will reach 3.0 MPa (Figure 8) and significantly suppress the stress-driven damage. This effect can best be observed near the -3 m level, where the stresses are the highest and where the concrete-lined section begins. Most importantly, the fracture mechanics modelling predicts the concrete liner will be damaged on week 3, the classical criteria predict damage on week 5 and the polyaxial criterion predicts damage on week 10 (one week into the extended heating option). The temperature distribution is even on the inner surface in horizontal cross sections with a less than 2 °C deviation along the circumference, and the temperature is predicted to reach over 120 °C during the nine weeks of heating (Figure 9).

**Figure 7.** The largest numerically modelled principal stress after nine weeks for EDZ and VT1 (a) and ONK-EH3 (b) in situ stress interpretations and plastic strain in concrete on week 10 (c) for both interpretations. (Modified after Publication 3.)
Figure 8. The modelled support pressure (MPa) generated by the thin concrete liner after three weeks (a), six weeks (b) and nine weeks (c) (Publication 2).

Figure 9. Modelled temperature evolution in the inner surface of the hole during the first 9 weeks of heating (Publication 3).
4 Rock joint replication method

4.1 Strength of rock joints and scale effect

The strength of jointed massive rock is controlled by the shear strength of the rock joints. The shear strength is found to be influenced by several parameters, including normal stress, the basic friction angle of the intact rock, the joint surface roughness, the joint wall compressive strength, the matedness of the joint surfaces, weathering and infilling affecting the cohesion. The effect of these parameters on the joint shear strength can be evaluated in laboratory scale.

However, the shear strength is known to be scale dependent. How the mechanical properties change as a function of scale is not fully understood. In most cases, the shear strength decreases when the sample size increases (Figure 10), for example, there is a negative scale effect (Bandis et al., 1981).

In Publication 4, we present a replication method which captures and digitises rock joint geometry, rescales or subsamples the geometry to the desired size, creates casting moulds using 3D printing, and finally mortar shear replicas can be cast and tested in a portable shear box apparatus. The method allows the study of the scale effect using non-damaging research methods, and low cost, commonly available research equipment. The method is based on a feasibility study by Yorke (2014) and a technical pilot by Korpi (2016).

Figure 10. The two shear strength components of basic friction and joint roughness, and their length scale effect from Bandis et al. (1981).
4.2 Geometry digitisation using photogrammetry

A single 175 cm x 95 cm x 6 cm mechanically split Kuru grey granite rock slab was used as the rock joint sample geometry for photogrammetry (Figure 11). The sample was thoroughly cleaned and photographed during an overcast cloudy day. A total of 414 photographs were taken using a Canon EOS 600D DSLR and a Canon EF 35 mm F/2 IS USM objective.

Most photos were taken at close range with small translation movements and large overlaps. Between the linear series, the camera angle was changed. The camera angle was varied vertically (pitch) from 30 to 80 degrees and horizontally (yaw) from -45 to 45 degrees; some of the pictures were taken at a different sensor angle (roll) from 0 to 45 degrees to reduce sensor bias. At the end of the shoot, some images were taken at longer distances, up to 4 m away to include the whole slab in a single image to allow crossmatching of each partial image.

The images were processed to a point cloud using VisualSFM software 0.5.25 using default settings (Wu, 2007; Wu, 2011; Wu et al., 2011; Wu 2013). The resulting cloud was cleaned up, cropped and meshed in CloudCompare 2.5.5.2. A section of 170 cm x 60 cm of uncompromised rock was selected and cropped for further analyses. Seventeen subsamples were taken as indicated in Figure 12, giving a 1:10 scale range from 17 cm x 6 cm to 170 cm x 60 cm. Next, all subsamples taken from the original sample (Figure 13a) were downscaled to a target testing size of 17 cm x 6 cm, and 3D printed using Stratasys Objet30 Scholar as plastic casting moulds (Figure 13b) for mortar replicas (Figure 13c) using JB1000/3 mortar with uniaxial compressive strength of C60/75. To quantify the loss of features by the production process, the geometry was captured from the original rock slab (Figure 13d), from the plastic 3D printed casting moulds (Figure 13e), and from the mortar cast replicas (Figure 13f).

Figure 11. The rock slab and the subsampling pattern used in the research.
Figure 12. The subsampling pattern 1:1 170 cm x 60 cm (a), 1:1.333 128 cm x 45 cm (b), 1:2 85 cm x 30 cm (c), 1:4 43 cm x 15 cm (d), 1:10 17 cm x 6 cm (e). The circles denote the centrepoint and the quarterpoints.

Figure 13. The rock slab (a), 3D printed plastic casting mould (b), and mortar cast replica (c), digital rock slab (d), digital 3D printed plastic casting mould (e), digital mortar cast replica (f). Photographs: Pauliina Kallio (a, b, c), and Joni Sirkiä (d, e, f).

4.3 Observed loss of geometry

Total of 34 sample halves was scanned. The replicas show a significant loss of geometry up to -40 % in the 10:1 scaling, measured as points per square millimetres (Kallio, 2015). On average, a replica sample loses 2 % of surface roughness on casting mould production, and 6 % of surface roughness in the casting of a replica sample measured as the ratio between the true surface area and the nominal surface area Rs values (El-Soudani, 1978). Top and bottom scan pairs differ 3.5 % to 9.6 % in Rs values (Sirkiä, 2015). The observed loss of geometry associated with rescaling is shown in Figure 14 using the Barton and Bandis (1982) correction for different scales. The typical loss of geometry attributed to each working stage of the replication process is shown in Figure 15 using the directional 3D roughness evaluation introduced by Tatone and Grasselli (2009).

There is no universally accepted metric for evaluating the successfulness of rock surface replication, so the roughness (Rs) of 1 mm x 1 mm window
normalised surfaces were used as a metric. Using this metric, it was subjectively evaluated that only 5 out of the 33 analysed cases retained the geometry sufficiently. While a minor success was achieved using replicas, it was decided to use an entirely digital scale series in conjunction with numerical modelling in the continuation research.

![Figure 14. Loss of JRC measured subjectively along the longer centreline using a plastic 25 cm long profilometer with the Barton and Bandis (1982) correction for different scales.](image1)

![Figure 15. Typical loss of digitally measured directional roughness during the replica manufacturing process from original rock surface to 3D-printed plastic casting mould, to mortar cast shear testing replica.](image2)
Figure 16. 170 mm x 60 mm replica shear tests for 1:1 (a), 1:2.5 (b), 1:5 (c), 1:7.5 (d) and 1:10 (e) scales under constant normal load conditions with normal pressure of 0.50 MPa.
4.4 Replica shear test results

For validation, a total of 17 shear tests for mortar cast replicas was carried out using constant normal load conditions and with a normal pressure of 0.50 MPa. Most of the results show a shear stress-displacement curve, typically attributed to poorly mated surfaces, and only a few replicas produced a clear peak strength (Figure 16). This interpretation is supported by the measured negative dilatation indicating vertical compression in most of the shear tests. The basic friction was established using smooth samples allowing the roughness component containing both large-scale geometrical components and small-scale asperity components to be studied separately. Despite the accuracy loss during the replication process and the high scatter in the testing results, a weak negative scale effect can be seen in the results (Figure 17). The replication process and the shear testing had technical challenges (Tolvanen, 2015), and more research is needed before conclusions about the scale effect can be drawn.

In later research, the mortar replication approach was not continued, and it was substituted with the photogrammetric digitalisation of the rock joint surfaces followed by numerical modelling of simulated shear tests (Kivivirta, 2017). The small tests had high variability, and larger test surfaces may have more homogenous shear behaviour.

![Figure 17. Observed scale effect for peak strength and residual strength of the mortar replicas. The measured friction component is also shown.](image-url)
5 Large rock sample joint shear tests

5.1 Background and motivation

The motivation to carry out large-scale rock joint shear tests comes from two sources. First, the large tests are needed to validate the photogrammetric roughness predictions at as large scale as is possible to execute in laboratory conditions. Secondly, Johansson and Stille (2014) published a new conceptual method which offers an explanation of the observed scale effect for fresh, unweathered rock joints. Later in 2016, Johansson published follow-up research for 60 mm by 60 mm and 200 mm by 200 mm samples. In our research, the samples were 500 mm x 250 mm and 2000 mm x 950 mm. This choice of scales allows testing the conceptual model near the scale when the scale effect is expected to even out. The hypothesis was that there should not be any observable difference between the 500 mm long and the 2000 mm long samples. The hypothesis is that there should be a negative scale effect associated with the increase in size.

5.2 Workflow description

Both of the samples used a shared workflow but had a different shearing arrangement. In the smaller 500 mm test, the shearing occurs as the top slab is pushed along the longer direction and for the larger 2000 mm test, the top slab is pulled along the longer direction. The small slab was sheared 25 mm (5 %) and the top slab 50 mm (2.5 %), which is also the defined canister shear displacement limit. First, the slabs were thoroughly cleaned using a vacuum cleaner and a soft brush and photographed before testing. Then, both slabs were tested in four stages with 4.4 kPa, 6.6 kPa, 9.2 kPa and 4.4 kPa normal pressure, using static weights and including the slab self-weight. The pull method was chosen, as it was easier to implement – however, it later turned out to cause stick-slip behaviour. Between the loading stages, the top slab was lifted and the location of any visually observable damage was recorded, and the spalled rock was removed from the surface. The slab was repositioned and retested with a different normal pressure. After all loading stages, full photogrammetry was carried out again for the damaged samples.
5.3 Medium scale testing arrangements

The medium scale 0.50 m by 0.25 m test was tested in a push configuration, and the in-plane rotation was unrestricted, but the upper slab was guided using ball bearing plates (Figure 18). Steel weights were used to increase the normal pressure. There was almost no change in the peak shear strength as the pressure was increased. However, the last test produced an unexpectedly low-peak shear strength result. The residual values produce almost linear relation with the normal stress as expected. The dilatation values produce the characteristic curve for shear tests except in the first stage where the dilatation first grows fast and then declines towards the end of the stage. After the four stages were completed, the top slab was rotated 180 degrees, and the residual strength was obtained. The stick-slip phenomenon can be observed in all tests, but there are no major jumps or associated loss of data.

5.4 Large-scale testing arrangements

The large-scale 2.00 m by 0.95 m test was tested in pull configuration (Dzugala, 2016), and the in-plane rotation was unrestricted, but the upper slab was guided using ball bearing plates (Figure 19). Gravel bags were used to increase the normal pressure. The first and last phase produce the same peak strength as expected, and there is a nonlinear increase of peak strength, possibly indicating small-scale damage. The first test also has a negative initial dilatation, indicating vertical compression (Figure 20). The other stages have the expected dilatation curve. The residual values produce almost linear relation with the normal stress as expected. There were significant jumps forward after the peak strength was reached, and all of the tests suffered from stick-slip phenomenon. Due to the data loss during jumps and the unexplained results in the first stage, it was decided to repeat the large-scale experiment in push configuration later.
Figure 19. Pull shear setup for the large 2.00 m by 0.95 m sample with ball bearing plates for guidance and LVDTs to monitor the displacements. Image: Lauri Uotinen

Figure 20. The dilatation observed in each testing stage for the large sample.

5.5 Shear testing results and scale effect

The testing results for both slabs are shown in Figures 21-23 and Table 1. The medium slab does not show a clear relation between the peak strength and the normal pressure (Figure 21). A non-linear peak strength can be seen for the larger slab (Figure 22). A comparison of the results is shown in Figure 23 and Table 1. It should be noted here that the normal pressures used (4.0 to 9.2 kPa) were relatively small.

For the peak strengths (Figure 24), there is a moderate negative scale effect, which corresponds more to the hypothesis of declining strength with increasing scale. While the Kuru grey granite is very homogenous, it should be remembered that only two tests were conducted. For the residual strengths (Figure 25), a minor but observable negative scale effect can be seen.
Figure 21. The shear testing results for the 0.50 m by 0.25 m slab as a function of normal pressure and as a function of shear displacement.

Figure 22. The shear testing results for the 2.00 m by 0.95 m slab as a function of normal pressure and as a function of shear displacement. (Publication 5)
**Figure 23.** The shear testing results for both the large 2.00 m by 0.95 m and the medium 0.50 m by 0.25 m slabs as a function of normal pressure.

**Table 1.** The shear testing results for slabs for each loading stage.

<table>
<thead>
<tr>
<th>Stage</th>
<th>Normal pressure</th>
<th>Large 2.00 m by 0.95 m</th>
<th>Medium 0.50 m by 0.25 m</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Normal pressure</td>
<td>Peak</td>
<td>Residual</td>
</tr>
<tr>
<td>1</td>
<td>kPa</td>
<td>kPa</td>
<td>kPa</td>
</tr>
<tr>
<td>4.0</td>
<td>6.7</td>
<td>2.7</td>
<td>2.8</td>
</tr>
<tr>
<td>2</td>
<td>6.6</td>
<td>15.1</td>
<td>4.3</td>
</tr>
<tr>
<td>3</td>
<td>9.2</td>
<td>17.2</td>
<td>6.4</td>
</tr>
<tr>
<td>4</td>
<td>4.0</td>
<td>7.2</td>
<td>3.1</td>
</tr>
</tbody>
</table>
Figure 24. The peak shear strength at different loading stages.

Figure 25. The residual shear strength at different loading stages.
6 Geotechnical risk management

6.1 Geotechnical risk assessment guideline

Geotechnical risk assessment guideline for underground mines has been developed at Aalto University as part of the I²Mine (The Innovative Technologies and Concepts for the Intelligent Deep Mine of the Future) project in 2011-2015 (Mishra and Rinne, 2014; Mishra and Rinne, 2015). The aim was to develop a method for geotechnical risk evaluation of geotechnical hazards in deep underground mines.

The methodology presented in the guideline consists of several phases (Figure 26). After the collection of data, the first phase is the assessment of the Geotechnical Hazard Potential (GHP), which evaluates based on the combination of rock mass competency and the mining method. The GHP aims to achieve preliminary information of geotechnical risk level in a mine. The next phase is the Geotechnical Risk Assessment (GRA), which is a formal risk evaluation stage. The risk is defined as a product of the likelihood of the hazard in question and the consequence if the hazard were to be realised.

![Figure 26. Geotechnical risk management, GHP, and GRA. (Publication 6)](image-url)
6.2 Dynamic control of underground mining operations

The method developed in the Dynamic control of underground mining operations (DynaMine) research project in 2014-2016 allows the real-time monitoring of stress state changes, using direct observations of displacements surrounding the rock spaces (Kodeda, 2014; Kodeda et al., 2015). First, the proposed method was benchmarked and shown to work with synthetic data. Then it was tested in the Kylylahti mine using 12 multipoint extensometers of 12 metres in length and 6 displacement sensor in each, but the results were inconclusive due to insufficient instrument reading accuracy (Ritala, 2016; Ritala et al., 2016). Based on I²Mine and DynaMine results, the real-time monitoring risk assessment concept using stress monitoring was first described in Mishra et al. (2016).

The DynaMine approach is based on matching the observed data to pre-calculated simulated behaviour. The three-dimensional stress state tensor change requires six independent components. For each component, a unit response is defined and stored. For the observed ground behaviour, the corresponding linear combination of unit responses can be found using multiple linear regression. The concept is illustrated in Figure 27 in 2D, where only three independent stress components (σ_x, σ_y, τ_xy) exist, and any state can be reconstructed by finding the multipliers A, B and C. The current approach requires knowledge of the geology surrounding the sensors and elastic behaviour of the rock mass. Only stress state changes can be observed and the initial in situ stress must be either measured using another method or back calculated from several excavation stage stress changes.

The developed stress inversion method is suitable for a semi-automated or fully automated risk management concept. As publication 6 concludes, it would fit well in the analysis stage of the measured data and would produce a stress state change, which could be used as input in the risk management guideline. The method currently requires knowledge of local geology near the sensors and elastic rock mass behaviour. It should be later extended to support excavation sequencing and to account for rock mass damage. The method is flexible with the type and amount of input data, and more input types could be programmed to allow the usage of existing mine sensor infrastructure.

\[
\begin{align*}
\sigma_x &= \begin{bmatrix} 1 & 0 & 0 \\ 0 & 0 & 0 \\ 0 & 0 & 0 \end{bmatrix}, \\
\sigma_y &= \begin{bmatrix} 0 & 0 & 0 \\ 0 & 1 & 0 \\ 0 & 0 & 0 \end{bmatrix}, \\
\tau_{xy} &= \begin{bmatrix} 0 & 1 & 0 \\ 1 & 0 & 0 \\ 0 & 0 & 0 \end{bmatrix}
\end{align*}
\]

\[
\sigma = A \cdot \sigma_x + B \cdot \sigma_y + C \cdot \tau_{xy}
\]

**Figure 27.** Unit stress components (σ_x, σ_y, τ_xy) and multiple linear regression of the stress tensor (σ) in 2D using arbitrary multipliers (A, B, C).
6.3 Fault Tree - Event Tree analysis

In publication 6, a Fault Tree (Figure 28) Event Tree (Table 2) example is shown using strainburst as an example case. For the demonstrated example case, the risk can be reduced from 60.9% to 12.2% by using a warning system. The risk can be visualised using a modified FN diagram (Figure 29). In this example, the summed risk reduces from 88,000 € to 11,000 €. The method appears to work well with the example data, and the approach should next be tried with real data from case studies.

![Figure 28. Example Fault Tree diagram showing faults leading to a strainburst. (Modified after Publication 6)](image)

<table>
<thead>
<tr>
<th>Strainburst (receives probability from Fault Tree)</th>
<th>Manned Shift</th>
<th>Injury</th>
<th>Fatality</th>
<th>Property Loss</th>
<th>Production Loss</th>
<th>Outcomes P(Outcome)</th>
</tr>
</thead>
<tbody>
<tr>
<td>H - 0.5</td>
<td>H - 0.65</td>
<td>H - 0.4</td>
<td>a1</td>
<td>4%</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>NH - 0.6</td>
<td>a2</td>
<td>6%</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>NH - 0.35</td>
<td>a3</td>
<td>21%</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>H - 0.6</td>
<td>H - 0.65</td>
<td>H - 0.4</td>
<td>a4</td>
<td>4%</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>NH - 0.6</td>
<td>a5</td>
<td>6%</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>NH - 0.35</td>
<td>a6</td>
<td>21%</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>H - 0.7</td>
<td>H - 0.65</td>
<td>H - 0.4</td>
<td>a7</td>
<td>18%</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>NH - 0.6</td>
<td>a8</td>
<td>27%</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>NH - 0.35</td>
<td>a9</td>
<td>14%</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>NH - 0.4</td>
<td>a10</td>
<td>10%</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>NH - 0.2</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

| Table 2. Example Event Tree showing various consequences arising from a strainburst. H = event occurs, NH = event does not occur. (Publication 6) |
Figure 29. Example of a modified FN diagram, where the probability is expressed on the vertical axis and the financial loss on the horizontal axis. The scenarios a1-10 without warning system and M1-M10 with warning system are shown in Figure 28 and Table 2. (Modified after Publication 6)

6.4 Bayesian network approach

The Fault Tree Event Tree approach works well in cases where the causality is well known, and probabilities are either known or can be estimated. In actual use, cases with multiple influencing factors, such trees can become overly complicated. Simultaneous incidents are not taken into account, and the nodes cannot be interdependent. To tackle these issues, in Publication 7, we have demonstrated the use of Bayesian networks (BN) in conjunction with the geotechnical risk management (Figure 30).

The main benefits are that BNs can work with a low to high amount of data and can learn the parameters from the observations. At the high amount of data, even the nodes themselves can be learned, leading eventually to an artificial neural network (ANN). BNs can be characterised as belief networks or low-data artificial neural networks. One benefit of the BN is that it can be used in real-time geotechnical risk management (Figure 31).

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1 In Publication 6 the Objective Risk line is incorrect. Here, the error has been corrected.
Figure 30. Decision-making model using the Bayesian network. (Publication 7, modified after Smith, 2006)

Figure 31. Real-time risk management process using Bayesian networks (BN). (Publication 7)
7 Discussion

In this thesis, advancements are shown in the fields of in situ experiment design, predictive numerical modelling, small-scale laboratory testing, medium and large-scale laboratory testing, concept development for geotechnical risk management and the use of Bayesian networks in conjunction with real-time risk management. Each of these research topics represents a snapshot of ongoing research: the in situ experiment reporting is ongoing, large-scale retesting is taking place, and the real-time geotechnical risk management concept is being tested at Kemi mine. For each research topic, there are observations worth discussion.

For the in situ test, it was carried out in the same hole as the preceding heating experiment. While minimal damage was observed, it is still possible that this has influenced the ICSE results. The rock mass is also highly heterogeneous, and the massive type wedge-shaped spalling has not been observed in ONKALO. This may result in structure localised stress damage, which can only be predicted by including such features in the numerical modelling. Finally, it is important that the heating pattern is even and that the heating is executed rapidly. In slower heating, the resulting local stress increase will be milder. The possible concrete-rock adhesion loss during the cooling stage could not be modeled accurately as it occurs after plasticity. However, in the KBS-3V concept, the rock spaces will be filled with expansive clay, and any loss of adhesion is unlikely to cause operational problems. Finally, it is noted, that while the thermal increase will not be sufficient to cause damage in the Olkiluoto repository, it is still important to establish a safety factor for the stress damage. For this purpose, the load-bearing capacity of the reinforcement options must be obtained.

The new photogrammetric method has an accuracy that challenges the commercial solutions on the market. During the research, we found that it is highly sensitive to how the photographs are taken. It is recommended to follow a preset pattern and remove the human influence from the photogrammetry setup. Due to parallelism, there is still some minor variability in the measurements, but the repeatability of the method is good. The method was shown to work well in laboratory conditions, and the current research is aimed at producing a portable system which is suitable for field use. The method has also been tested for open pit mines (Iakovlev, 2015; Iakovlev et al., 2016; Sirkiä et al., 2016), and it may prove to be useful in the reduction of subjectivity with rock mass
characterisation methods (Barbens et al., 2014). The proposed mortar replication method had too high of a loss of roughness due to the manufacturing process, and it was abandoned. Instead, it is suggested that the photogrammetry could be used to produce digital samples, which would then be sheared in numerical modelling with either particle flow codes or with hybrid finite element discrete element codes. This approach is currently being investigated by both Aalto University and KTH Royal Institute of Technology.

The mystery of the observed scale effect was not solved in this thesis, but we still took steps forward. Two large-scale laboratory shear tests were carried out for a medium, 0.50 m by 0.25 m, slab in push and a large, 2.00 m by 0.95 m, slab in pull, under constant normal load conditions, using displacement resetting. The larger slab test is the largest of its kind in the world. The current shear tests show a negative scale effect, but a vertical compression was observed in the first stage. This can be an indication of surface mismatch. Lack of matedness has been suggested as a possible cause for the scale effect (Zhao, 1997a; Zhao, 1997b). The slab was tested using the displacement resetting method, and any damage carries over to each subsequent stage. The amount of damage was checked and mapped between each stage, and for the larger, 2.00 m by 0.95 m, slab, the last and end tests show no significant difference. Photogrammetry was used before and after the testing, and the directional roughness shows no significant difference. For future research, it is imperative that the larger slab size be retested using the push configuration. Depending on the results of the retest, testing other scales to establish scale effect curve shape or further retesting of current sizes to establish repeatability of the results will be needed.

The geotechnical risk management concept was developed towards a real-time system. It begins with the I2Mine developed GHP and GRA, followed up by more traditional risk management with real-time monitoring. Use examples were shown with Fault Tree and Event Tree methods, and the risk can be visualised using the modified FN diagram method presented in Publication 6. However, the authors no longer recommend the FT-ET methods, as the required probability values for each event are hard to set objectively, and the mutual occurrence or dependencies are omitted. Instead, Publication 7 suggests using Bayesian networks, which provides an evaluation that can develop from qualitative assessment to quantitative assessment as more data becomes available. The probabilities do not need to be entered and can be learned from available data. Spross et al. (2014, 2016) have pointed out that the Observational Method lacks a society-acceptable way of defining the probability of failure. Using Bayesian networks may solve this problem. From the studied cases in Publication 7, no single distribution would fit across several mines. Possible causes for this include different site-specific geology and the human effect. In future research, the Bayesian method should be used for an operating mine using their site-specific geology and site-specific protocols.

For each research topic, future research is needed. We will know more with the ICSE observations, the large-scale, push-shear retesting, Kemi mine stress change measurements and risk management data. Research is already ongoing on the further development of the Fractuscan fracture surface scanner and the numerical modelling of shear tests using digitized rock joint surfaces.
8 Conclusions

The three main conclusions each address the stress-driven damage prediction and mitigation. The stress-driven damage can be reduced using support pressure generated by thin concrete liners. A new method was developed to capture rock joint geometry using photogrammetry and to manufacture mortar replicas for laboratory scale shear testing. Finally, the use of Bayesian networks, together with real-time geotechnical risk management concept, was demonstrated. In more detail, the conclusions are:

1. An in situ experiment was designed to induce stress damage using heating. The In situ Concrete Spalling Experiment (ICSE) comprises a circular hole bored in the rock mass, and half of it reinforced with a concrete liner. Based on the numerical modelling results, increasing the temperature around the circular openings using linear heaters is a potent way of increasing the local rock mass stress.

2. In ICSE, both fracture mechanics modelling and the thermomechanical finite element method were used to produce numerical blind predictions. The results suggest that a thin concrete layer is useful in suppressing the stress-driven damage. A 40 mm concrete liner is predicted to produce support pressures up to 3 MPa, which reduces the tangential stress by a third and significantly suppresses damage.

3. The numerical models suggest that the interface between the concrete liner and the rock mass remains in compression during the entire heating stage, and no significant shear or loss of adhesion is expected. Adhesion loss during cooling takes place after the damage process, and it could not be modelled accurately as it occurs after plasticity.

4. The proposed new photogrammetric method can record rock surface geometry at submillimeter accuracy of 16 pts/mm² or point spacing of 250 μm. The method requires commonly available, mid-range price photographic equipment, and it is based on software freely available for academic or personal use.

5. A new methodology was developed for replicating fractures to facilitate testing under various loading conditions and scales with identical fracture surface. In this research, 17 subsamples obtained from an artificially induced tensile fracture were digitally scaled down to 17 cm long and 6 cm wide, 3D printed into casting moulds, and finally mortar replicas were shear tested under a constant normal load, with 0.50 kPa
normal pressure. A high scatter was observed in the results. During the casting mould production, 2% of the surface roughness was lost and 6% during the casting of replica sample compared to the original rock.

6. The Aalto Shear Pull Experiment for Rock Tensile Fracture (ASPERT) was a four-stage shear test with displacement resetting for a large, 2 m by 0.95 m, Kuru grey granite sample with an artificially induced tensile fracture. The bottom slab was held in place, and the top slab was sheared 50 mm by pulling, the same distance that spent nuclear fuel canisters are designed to withstand. A non-linear peak strength was observed for the larger, 2 m by 0.95 m, slab with increasing normal load. Compared to a medium-sized, 0.5 m by 0.25 m, shear test, a slight negative scale effect is observed.

7. Based on The Innovative Technologies and Concepts for the Intelligent Deep Mine of the Future (I²Mine) and Dynamic Control of Underground Mining Operations (DynaMine), a real-time geotechnical risk management concept for deep underground spaces was developed. The use of a Fault Tree - Event Tree methodology was demonstrated using strainburst as an example case. The method appears to work well with example data, and the approach is ready to be tested using real data.

It can be concluded that the six objectives set for this thesis were reached: the ICSE was designed, blind predictions were given, a new method was developed to digitise the roughness of rock joints, a new method was developed to manufacture arbitrary scale rock joint replicas, large-scale validation shear tests were carried out, and the geotechnical risk management concept was modified towards real-time monitoring and the suitability of using Bayesian networks to cope with the changing amount of data was evaluated. The results contribute towards the long-term goals to predict stress-driven damage in deep underground spaces and the continuation research is already ongoing. Based on the results, the following future research is recommended:

1. Prediction-observation study of the ICSE.
2. Push shear test for a 2 m by 1 m artificial tensile fracture.
3. Large-scale, in situ experiments with joint length of 5 m or more.
4. CNS testing for large samples of 0.50 m by 0.25 m and 2 m by 1 m.
5. Sensitivity studies for the accuracy of the photogrammetric method.
6. Calibration data for numerical modelling of rock joint shear testing.
7. Testing the real-time risk management concept with real mine data.
8. Testing the Bayesian networks with mine-specific data and protocols.
References


Kodeda, S. (†). (2014). Stress state change estimation using back calculation of strain sensor data, Manuscript for Master’s Thesis, Aalto University, p. 60


Failures in rock spaces may occur due to a variety of reasons, but the two most common are structurally controlled gravity-driven and stress-induced causes. Usually, near-surface rock spaces are subject to gravity-driven failure, but in deep excavations the in-situ stress of the rock mass increases and the risk of stress-driven problems grows. Five common stress-driven damage mechanisms are rockburst, spalling, convergence, shearing, and seismic. Excessive convergence is rarely a problem in hard, massive rock mass. In this thesis, the remaining four of these five failure mechanisms are addressed.