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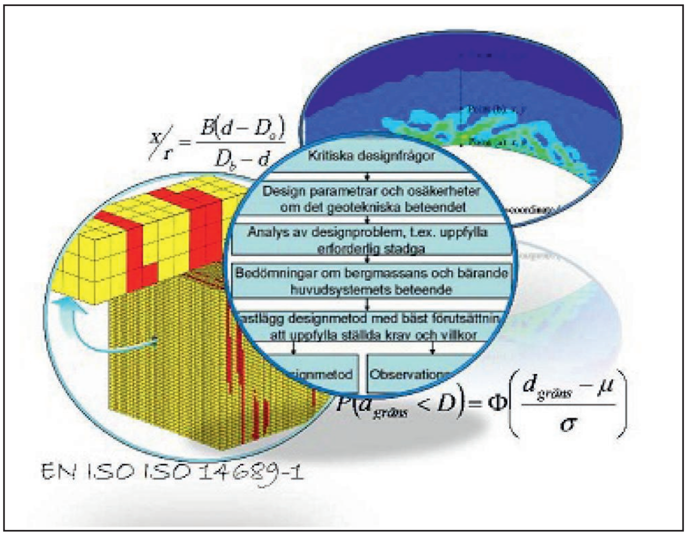

THE OBSERVATIONAL METHOD

Håkan Stille
Miriam Zetterlund
Kelvis Pérez
Mehdi Bagheri

BeFo Rapport 95

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Characterisation of Hard Rock Acccording to the Observational Method and Value of Information Analysis

Miriam Zetterlund, Chalmers University of Technology

Deformation and Failure of Hard Rock under Laboratory and Field Conditions

Kelvis Pérez, Luleå University of Technology

Model Uncertainty of Design Tools to Analyze Block Stability

Mehdi Bagheri, KTH Royal Institute of Technology

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INTRODUCTION

In desiging an underground excavation many parameters with varying degree of uncertainties must be taken into account. These uncertainties are related to sub-surface conditions and other site-specific requirements. Safety issues and providing underground structures with an economic design taking the geological setting into account, was the key considerations when the basis for the observational method was formulated. The observational method is one of the designated design methods in Eurocode, EN 1997-1:2004. This allows the designer to employ formal approaches towards design uncertainties as well as towards results from monitoring and observations made during construction.

According to EN 1997-1:2004, section 2.7; "when prediction of geotechnical behaviour is difficult, it can be appropriate to apply the approach known as the observational method, in which design is reviewed during construction". The concept of geotechnical behaviour is not specifically defined when applied to underground excavations. However, the difficulty to predict geotechnical behaviour is equal to the uncertain prospect of achieving a sufficiently accurate assessment of;

- the location of foreseen rock qualities,
- the quality of the applied rock support measures,
- the interaction and subsequent behaviour of rock mass and support elements.

The application of the observational method includes taking engineering decisions despite uncertainties in sub-surface conditions, as well as to employ construction experience and information from monitoring, all with the aim to reduce uncertainties in the parameters that govern the design. In Sweden the observational method approach is known by the designation, active design. The basis is to establish a preliminary design, devise contingency actions for such a case that the structural behaviour deviates from the expected, select and execute relevant observations during construction and to conduct modification of design to suit actual conditions. This procedure in itself may be a source of faulty design and therefore requires stringent handling of the design uncertainties. The preparation of contingency actions before construction is a mean to mitigate this specific problem.

There are formal requirements in the Eurocode that the behaviour of the construction shall be monitored during construction. This implies that relevant design parameters that can be predicted and monitored must be devised. These parameters are designated as control parameters and define the acceptable limits of design. Maintaining high quality in the monitoring process and the subsequent analysis is a prerequisite for a qualified decision making process. The observational method are used for assessing the stability of the structural system, the rock mass and support, as well as for controlling the design requirements related to durability and serviceability.

Another way is to describe that the base of observational method is the cases when the geotechnical prerequisites of design will be better and more easily determined during construction than in advance. This will not imply that preliminary design can be omitted. Instead preliminary design has to be as correct as possible in order to be followed up and if required adjusted during construction.

The behavior of the geotechnical structures has to be measurable. The serviceability and ultimate limit states also have to be defined, if possible with the same variables in order to facilitate the use of the Observational Method. Another prerequisite of the method is that the uncertainties involved in the rock design must have its origin in lack of knowledge and can be reduced by few observations. If on the other hand the uncertainties are coming from an outcome of more true stochastic variables, single observations will not reduce specifically the uncertainties and thus the observational method will not be applicable.

A critical element in the design process is to establish relevant control parameters that expose significant events that influence the geotechnical behaviour during construction. One must be able to quantify such parameters in order to validate the design requirements. The control parameters may be linked to the quality of or to the structural behaviour of the rock mass and the support elements. The control parameters must be selected carefully and with a good understanding of the significance to the design situation. The monitoring plan must take into account the important aspects of documentation and analyses of monitoring results as well as means of communicating significant events so that contingency actions can be undertaken successfully.

A survey of current design practices and procedures reveals that design within the framework of the observational method (Holmberg and Stille 2009);

- is comparable with today's practice,
- implies that observations shall focus on assessing the current rock mass quality, controlling that the support measures meet the requirements of the technical specification and revealing whether the structural behaviour lies within the acceptable limits of behaviour,
- introduces additional demand on transparency and traceability,
- introduces additional demand on the contractual relations and documents.

The above discussed basic principles were the starting point for the BeFo´s projects 216-218 which was diveded in three parallel Ph.D projects with the overall objectives to study different and significant parts in the design process with Observational Method.

One project is studying how the value of a preinvestigation shall be estimated by comparing the cost of the investigations with the utility of the received information. It will give the very base for a descision if the Observational Method should be applied or the design should be based on calculations or emperical based knowledge.

The second project is studying the possibility to redefine the limit states in terms of measurable variables. This implies that the classical approach with loads and bearing capacities as input in the ultimate limit state function has to be changed to possible and acceptable behaviour defined as deformations and strains. It will give a direct intrepretation of measured behaviour.

The third project is studying the uncertainties of the rock mechanical models. If the model uncertainties will be too dominant (biased or unprecise) the behaviour will be more of type stochastic variable and the observationale method will have a limited application. The study has been directed towards block instability which is the most comon failure mode.

Professor Håkan Stille
Division of Soil- and Rock Mechanics
Department of Civil and Architectural Engineering
KTH Royal Institute of Technology, Stockholm

ROCK ENGINEERING RESEARCH FOUNDATION
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Characterisation of Hard Rock Acccording to the Observational Method and Value of Information Analysis

Miriam Zetterlund, Chalmers University of Technology

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Introduction

The Observational Method

Terzaghi and Peck (1948) first introduced the Observational Method for application when uncertainties in the prior investigations are high. The method was later defined and described in more detail by Peck (1969), where he states eight conditions that need to be fulfilled for complete application of the method:

a) Exploration sufficient to establish at least the general nature, pattern and properties of the deposits but not necessarily in detail.
b) Assessment of the most probable conditions and the most unfavourable conceivable deviations from these conditions. In this assessment geology often plays a major role.
c) Establishment of the design based on a working hypothesis of behaviour anticipated under the most probable conditions.
d) Selection of quantities to be observed as construction proceeds and calculation of their anticipated values on the basis of the working hypothesis.
e) Calculation of values of the same quantities under the most unfavourable conditions compatible with the available data concerning the subsurface conditions.
f) Selection in advance of a course of action or modification of design for every foreseeable significant deviation of the observational findings from those predicted on the basis of the working hypothesis.
g) Measurement of quantities to be observed and evaluation of actual conditions.
h) Modification of the design to suit actual conditions.

Consequently, the method is only suitable in projects where the design can be altered as the construction proceeds. The principles of the Observational Method are presented in Figure 1. In his article, Peck emphasises that it is very important for the implementation of the method that there are pre-planned actions for all possible outcomes. If not, the method has not been used completely.

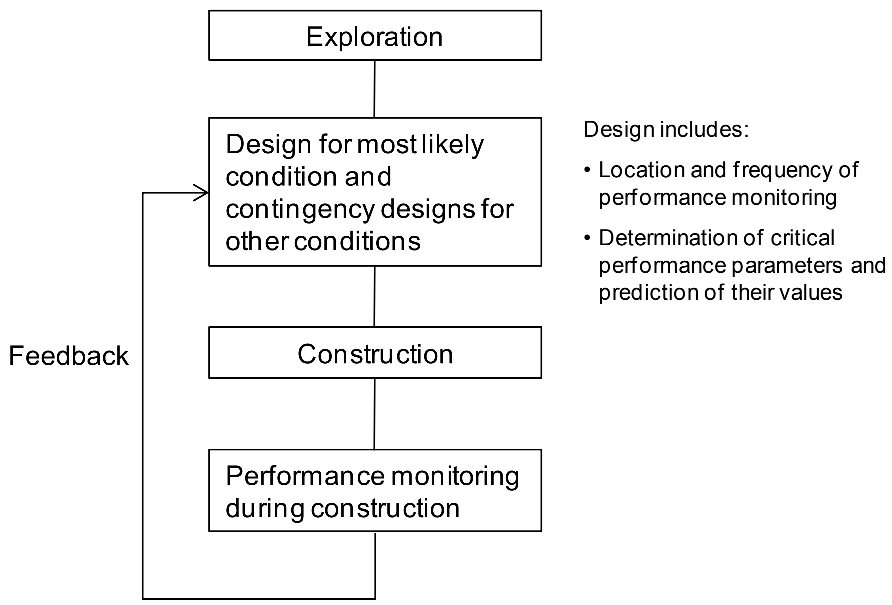

Figure 1. The principles of the Observational Method (Modified from Einstein and Baecher, 1982).

In cases when the conceptual model includes considerable uncertainties, the Observational Method could be the only reasonable method of design. On the other hand, it is not preferable to use the Observational Method in cases when the probability of rock failure is low, or when the cost of a conservative design is lower than the cost of applying the Observational Method (Stille et al., 2005).

Peck (1969) identified two ways of applying the Observational Method. In the first one, *ab initio*, the Observational Method is applied from the initiation of the project, when traditional design would most probably result in an overly conservative design and observations combined with planned courses of action may result in lower costs without reduced safety. The second application, the *best way out* procedures, may be used when some unexpected behaviour arises during construction and there is no economically acceptable alternative (Powderham, 1994).

Looking closely, there are some differences between the original formulation (Peck, 1969) and the text in Eurocode 7 (CEN, 2004). For example, the concepts of *ab initio* and *best way out* are not mentioned in the Code. The Eurocode 7 states five requirements that should be met before the construction is started when the Observational Method is applied:

- acceptable limits of behaviour shall be established;
- the range of possible behaviour shall be assessed and it shall be shown that there is an acceptable probability that the actual behaviour will be within the acceptable limits;
- a plan of monitoring shall be devised, which will reveal whether the actual behaviour lies within the acceptable limits. The monitoring shall make this clear at a sufficiently early stage, and with sufficiently short intervals to allow contingency actions to be undertaken successfully;
- the response time of the instruments and the procedures for analysing the results shall be sufficiently rapid in relation to the possible evolution of the system;
- a plan of contingency actions shall be devised, which may be adopted if the monitoring reveals behaviour outside acceptable limits.

In addition, the Eurocode points out some requirements for the use of the method, such as

- monitoring should be carried out as planned during construction;
- the results of the monitoring should be assessed at appropriate stages and the planned contingency actions shall be put into operation if the limits of behaviour are exceeded;
- monitoring equipment should be either replaced or extended if it fails to supply reliable data of appropriate type or insufficient in quantity (CEN, 2004).

Some of the benefits of the Observational Method are, among others, a stronger link between design and construction and an improved understanding of the interaction between geology and structure. One key requirement for the application of the Observational Method is that an acceptable level of risk must be identified and controlled. It is also argued that the Observational Method can be viewed as an aid in risk management and that a correct implementation of the method can lead to increased safety, for instance by focusing on good communication, planned procedure and control (Powderham, 1994).

When planning the observational programme, identification of the critical observations for the system is essential. To find these, an understanding of the processes involved and the level of accuracy is needed (Powderham, 1994). It is also important to consider the time for feedback and assessment of the measurements to either confirm a prediction or to alert for unfavourable trends in the data. The time between warning and the effect of the contingency action need to be shorter than the time for the deviation in the working hypothesis, e.g. failure. This means that the method is more suitable in rock masses where ductile failure mechanisms occur, since it is then possible to measure the failure development and to plan for courses of action during that time. In rock masses with brittle failure mechanisms, the Observational Method is only an aid to localise or limit failure and in doing so reduce the risk (Nicholson, 1994).

The initial design should be based on the most probable conditions (Peck, 1969). With the monitoring system it must be possible to measure the whole range between the most probable and the unfavourable conditions. The formulation of contingency plans needs to be explicit and with clear instructions on how to proceed if the trigger values are exceeded (Nicholson, 1994). Often, the physical quantities are not possible to measure directly. For instance, in the case of groundwater-bearing conditions in rock, it is the flow and the pressure, not the conductivity, that is measured (Holmberg and Stille, 2007). It should be noted that the observations could lead both to an increase or a decrease in the amount of, for example, rock support.

According to statements by Peck (1969) and CEN (2004), the Observational Method can be summarised into the following main activity elements (Stille et al., 2005):

- The decision problem
- Limits for acceptable behaviour
- Intervals for probable behaviour
- Probability of exceeding the stipulated limits
- Monitoring systems
- Observations and updating
- Control programmes and planned courses of action/contingency plans.

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Geological Characterisation and the Observational Method. Application of Value of Information Analysis

(Licentiate thesis ISSN 1652-9146; nr 2009:5)

1. Background

The implementation of the new European standards for geotechnical design, Eurocode 7 (CEN, 2004), calls for new practice in the underground construction industry in Sweden. The code proposes the Observational Method as an alternative method when the geotechnical behaviour is difficult to predict.

When working according to the Observational Method, the design of the underground construction can be based on the most probable geological conditions (based on pre-investigations), instead of an overly conservative design based on the worst case scenario. During the construction phase it is possible to modify the design according to prepared contingency plans when the geological conditions prove to be different from what was expected.

In Sweden, the concepts of *Active design* and *Design as you go* have been frequently used in tunnelling since the 1970s. Active design is based on the same ideas as the Observational Method. The thought behind it is to make a preliminary design and then use observations to gradually modify the design during the construction phase (Stille, 1986).

The Observational Method differs from both Active design and Design as you go since it is a defined method where the contingency plans prepared in advance are important. Even though the choice of design is made during construction based on observations, the alternatives are developed in the design phase. This method has previously been used in geotechnical engineering and there are some examples in the literature where the method has been used in tunnelling. However, the examples are few and a link to rock mass characterisation is lacking.

2. Aim and objectives

The aim of the thesis is to create a platform for rock mass characterisation according to the Observational Method. The objectives of the thesis are as follows:

- To describe how Bayesian statistical methods can be useful in rock mass characterisation according to the Observational Method.
- To show how Value of Information Analysis can be used as a tool when working according to the Observational Method.

3. Limitations

Rock mass characterisation is a broad field and some limitations are necessary.

- The purpose of the study is not to develop a new rock mass classification system, but to develop a method of rock mass characterisation in accordance with the intentions of the Observational Method.
- A distinction is made between the concept of characterisation and classification. Hence, of the numerous classification systems in use today, brief descriptions are given of just a few.
- The study is valid primarily for tunnelling projects.
- The focus of the thesis is on crystalline rocks.
- Even though a clear link to management and contractual issues is seen, the thesis does not include anything that deals with this subject in greater depth.

4. Value of Information Analysis

One of the challenges in rock mass characterisation is to make the optimal number of investigations. Most investigations are expensive and hence there is often a request for limitations on the investigations in order to reduce the cost. At the same time investigations that are too limited lead to large uncertainties in the results. Value of Information Analysis (VOIA), sometimes also referred to as Data Worth Analysis, is a central element in decision-making in complex problems and can help to create a rational design strategy for investigation programmes (Bedford and Cooke, 2001; Freeze et al., 1992). Key questions in such a strategy are:

- What should be measured?
- Where should measurements be made?
- How many measurements should be made?

The method is based on Bayesian statistics and cost-benefit analysis and is suitable for problems when different alternatives are evaluated and compared, e.g. the design of an investigation programme when the number of measurements or investigations needs to be determined. In VOIA the value of new information, from measurements for example, is assessed by estimating the uncertainties in the present information compared to the expected reduction in uncertainty following collection of new information. The cost and the time it takes to obtain better information must be compared to what can be saved by modifying the investigation programme. New information is only interesting when it can change the outcome of the decision and thus is of value for the decision-maker. The cost of conducting an investigation or making a measurement should be less than what is expected to be saved, otherwise the investigation should not be made (Bedford and Cooke, 2001). Hence, the added value does need to be a monetary benefit; it can also be a reduced total uncertainty from the newly gained knowledge compared to the uncertainty in knowledge of the present state (Back, 2006, among others).

As indicated above, VOIA can be seen as a form of cost-benefit analysis where different alternatives are compared. Irrespective of the total number of alternatives, one alternative is the null alternative where nothing is done, and in consequence the costs are zero but the risk costs can be significant. The other alternatives are when something is done at a certain cost, leading to reduced uncertainty and risk cost.

In short, the working order in a VOIA consists of the following steps:

1. Prior analysis
   Analysis based on the present state of knowledge. Results in an expected total cost or benefit.
2. Preposterior analysis
   Analysis based on the expected information from the investigation programme. The analysis is performed following definition of the programme but before the investigations have taken place. It results in an expected value of information (EVI). Estimation of the expected value of perfect information (EVPI), and the net value of information (NEVI) are also part of the analysis.

Thereafter, the investigations are carried out if they are finacially justified. Strictly speaking, the VOIA is completed after stage 2, although it can be supplemented with a posterior analysis performed where the value of information gained from the investigations performed is calculated. In an updating process, the posterior value will serve as prior information in a new, updated VOIA (Back, 2006; Freeze et al., 1992).

4.1 Prior analysis

The prior analysis is the start of the VOIA and is focused on the choice between the alternatives; the value of the prior analysis is the value of the best alternative. If P(F) is the probability of failure, C_F the cost if failure occurs, and C_I the cost for preventing failure, the value of the prior analysis can be calculated as the maximum of the null alternative and the other alternatives, where the risk cost, $C_F \cdot P(F)$, can be reduced totally by the C_I. Failure is defined as an undesired state of nature or event. If two alternatives are compared, the value of the prior analysis is given as:

$$\Phi_{prior} = \max_i \Phi_i = \max(0, C_F \cdot P(F) - C_I) = \max(0, \alpha \cdot C_I \cdot P(F) - C_I) \qquad (1)$$

The factor α describes how much more it would cost if failure occurs compared to the cost of preventing failure, i.e.

$$\alpha = \frac{C_F}{C_I} \qquad (2)$$

4.2 Preposterior analysis

The preposterior analysis focuses on the information that can be gained from further investigations. Can more information change the choice made after the prior analysis or are the uncertainties already reasonably low?

An event tree illustrates the outcomes given different scenarios in the decision analysis, see Figure 2. In the figure, the two main scenarios are illustrated in the first two branches, either that an event (for example failure) happens F, or, an event does not happen F’. In the forthcoming branches, the conditional outcomes are shown. Given that an event happens, it can either be detected, D│F, or not be detected, D’│F. Analogously, given that an event does not happen it can be detected, D│F’, or not be detected, D’│F’. The conditional probabilities P(D’│F) and P(D│F’) describe the errors in the investigation method.

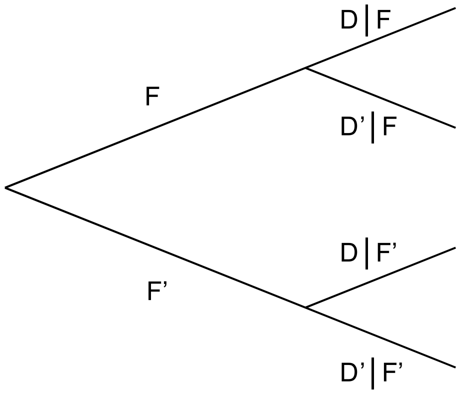

Figure 2. Example of an event tree for preposterior analysis.

The expected value of the preposterior analysis is calculated as:

$$\Phi_{prepost} = \max(0, \alpha \cdot C_I \cdot P(F|D') - C_I) \cdot P(D') + \max(0, \alpha \cdot C_I \cdot P(F|D) - C_I) \cdot P(D) \quad (3)$$

Bayes' theorem,

$$P(D|F) = \frac{P(F|D)P(D)}{P(F|D)P(D) + P(F|D')P(D')} \quad (4)$$

and the law of total probability

$$P(D) = P(D|F) \cdot P(F) + P(D|F') \cdot P(F'); \qquad P(D') = 1 - P(D) \quad (5)$$

give the conditional probabilities P(F│D') and P(F│D).

The expected value of information (EVI) is calculated as:

$$EVI = \Phi_{prepost} - \Phi_{prior} \quad (6)$$

4.3 Expected value of perfect information (EVPI)

There is an upper boundary for the value of new information when the investigations are as good as they can possibly be i.e when there are no errors in the investigation method. If the error probabilities are set at zero in the calculations, P(D'│F) = 0, and P(D│F') = 0, the expected value of perfect information (EVPI) can be estimated using the same procedure as the preposterior analysis.

Since investigations are never worth doing if the cost of performing them exceeds the expected value of perfect information it is advisable to calculate EVPI as a check before the preposterior analysis.

4.4 Net value of information (NEVI)

When the data value is calculated it should be compared with the costs of making the investigations. It is worth carrying out the investigations as long as the data value exceeds the costs of the investigation. The net value of information (NEVI) is calculated as

$$NEVI = EVI - C_M \quad (7)$$

where C_M is the investigation cost.

5. Rock Mass Characterisation and Value of Information Analysis

VOIA theory within tunnelling and geological and hydrogeological characterisation for grouting in hard rock was tested in two hypothetical examples (Zetterlund et al., 2008, and Zetterlund et al., 2009). A schematic illustration of how VOIA can be incorporated into rock mass characterisation in the pre-investigation phase of a tunnelling project is presented in Figure 3. VOIA theory can be applied to different types of decisions, although a VOIA is only valid for the particular decision for which it was made. Each decision has its own critical factors and specific conditions, and the definition of failure in each analysis should correspond to the purpose of the analysis. In a tunnelling project, for example, one VOIA can be made for grouting purposes and one for rock mechanical aspects. Even though it is theoretically possible to make a combined VOIA for a number of aspects, it is advantageous to keep the complexity level as low as possible.

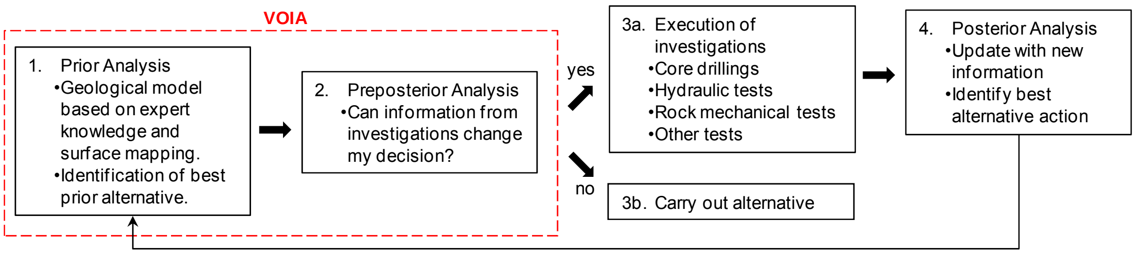

Figure 3. The steps in VOIA related to the pre-investigation phase of a tunnelling project.

6. Publication I

The aim of the first example was to present the first steps towards a methodology for rock mass characterisation in accordance with the Observational Method and decision theory. The focus was on how to obtain the necessary information in a pre-investigation of a tunnel by means of VOIA.

The exemplified tunnel, approximately 90 metres in length, was to be constructed in Precambrian diorite. In order to reduce the inflow of water into the tunnel, pre-excavation grouting is planned for the whole tunnel length. Initially, a basic grouting design is planned although there is a possibility that the design will not be sufficient in certain sections of the tunnel. These sections will then be grouted a second time prior to excavation. The second round of grouting can be seen as a project risk, i.e. if the time for the grouting is not included in the project budget and time schedule it will be associated with delays, and cost increases.

6.1 Prior analysis

In the prior analysis, the main decision is whether it is sufficient to plan for only a basic grouting design, or if a second round of pre-excavation grouting should be planned from the start of the project. To decide, the decision-maker needs to have an opinion about how likely it is that the basic grouting design will meet the stated requirements of water inflow into the tunnel and how likely it is that the requirements would not be met. Consequently, the decision is between two alternatives (0) the null alternative, when only a basic grouting design is planned from the start of the project, and (1) an alternative where a second grouting round is planned in addition to the basic design.

The prior analysis is basically a cost-benefit analysis of the two alternative risk costs. Failure is defined as an undesired state of nature or event; in this example the need for a second grouting round as a consequence of too large inflow into the tunnel.

The costs involved in the two alternatives are related to the grouting, such as material, staff and equipment, as well as the costs of a delay if an unforeseen transmissive fracture zone is detected. Subsequently, the benefit of alternative 1, when a second grouting round is planned from the start, is the reduced risk of unplanned costs due to the grouting compared to the risk costs in the null alternative, when no second grouting round is planned. The costs are showed in Table 1 and Table 2.

Table 1. Costs for alternative 0.

Costs, C_0 **(With no second round of grouting planned)**	**Costs, C_{F0}** **(At failure)**
No costs	Risk of water inflow into the tunnel Cost of material and execution of grouting Cost of a stoppage in the process (staff and machinery, delay penalties)

Table 2. Costs for alternative 1.

Costs, C_I **(With two rounds of grouting)**	**Costs, C_{F1}** **(At failure)**
Costs of material and execution The time for grouting is included in the time table from the start of the project	Risk of water inflow into the tunnel Costs of material and execution of grouting Costs of stoppage in the process (staff and machinery, delay penalties)

The cost of unplanned grouting, or the cost of failure, C_F, was assumed to be a factor α times higher than the cost of grouting planned from the start of the project. Included in the factor α are all costs that will be added to the cost of the basic design if a fracture zone with a transmissivity exceeding the critical were to be found unexpectedly. Hence, α includes the cost of machinery, equipment and staff not directly involved in the grouting procedure but which are put on hold when the second grouting round takes place. The factor also includes potential penalties if the additional grouting round causes a delay in the total project. Of course, α is project specific and increases with project size. The probability of a second grouting round was represented by the probability of finding a fracture zone with a transmissivity higher than the critical, i.e.

$$P(F) = P(T > T_{crit}) = P(Z) \tag{8}$$

Input data for the prior analysis is summarised in Table 3.

Table 3. Input parameters for prior analysis.

Grouting cost,	C_I = €167 200
Cost of an unplanned second grouting round (Failure cost)	$C_F = \alpha \cdot C_I$
Factor for cost increase	α = 5
Probability of a second grouting round	P(Z) = 0.33

The result of the prior analysis was calculated according to Equation 1 as:

$$\Phi_{prior} = \max_i \Phi_i = \max(0, C_F \cdot P_F - C_I) = \max(0, \alpha \cdot C_I \cdot P_F - C_I) = \ 108\ 680€$$

Note that

$$(\alpha \cdot P_F - 1) \cdot C_I > 0 \Leftrightarrow P_F > \frac{1}{\alpha} \tag{9}$$

Even though the factor α may seem difficult to estimate, the central problem in the prior analysis is to determine the probability of finding a fracture zone where the transmissivity exceeds the critical, $T > T_{crit}$. In Zetterlund et al. (2008) this was done exclusively with expert knowledge. A more refined model, presented in Zetterlund et al. (2009), will be discussed later on.

6.2 Preposterior analysis

The main decision in the pre-posterior analysis is whether information from further investigations, in this example a core-drilled borehole, can be of value in the decision-making process. The biggest challenge at this stage in the VOIA is to assign values to the error probabilities of the investigation method, P(D'│Z) and P(D│Z'). In this case the error probabilities describe the accuracy of the borehole as an investigation method and the ability of the borehole to represent hydrogeological properties of the total rock mass around the planned tunnel. More specifically, P(D'│Z) is the probability of missing a water-bearing fracture with the probing borehole, and P(D│Z') is the probability of falsely interpreting a fracture as water-bearing even though it is not. In addition to the errors in the investigation method and errors in interpretation, the error probabilities also include the possibility of human mistakes, and the measurement limit of the equipment. For input parameters for the preposterior analysis, see Table 4.

Table 4. Probabilities used in the preposterior analysis.

Probability of fracture transmissivity higher than T_{crit} (grouting is needed)	P(Z) = 0.33
Probability of fracture transmissivity lower than T_{crit} (grouting is not needed)	P(Z') = 0.67
Probability to detect a high fracture transmissivity that exists	P(D│Z) = 0.9
Probability to not detect a high fracture transmissivity that exists	P(D'│Z) = 0.1
Probability to detect a high fracture transmissivity that does not exist	P(D│Z') = 0.1
Probability to not detect a high fracture transmissivity that does not exist	P(D'│Z') = 0.9

The result of the preposterior analysis was calculated, according to Equation 3, as:

$$\Phi_{prepost} = \max(0, \alpha \cdot C_I \cdot P(Z|D') - C_I) \cdot P(D') + \max(0, \alpha \cdot C_I \cdot P(Z|D) - C_I) \cdot P(D) = \ 187\ 431€$$

According to Equation 6 the Expected Value of Information (*EVI*) was calculated as:

$EVI = \Phi_{prepost} - \Phi_{prior} =$ 187 431-108 680 = 78 751€

To find the upper boundary of the maximum value of information, the Expected Value of Perfect information (*EVPI*) was found by assigning the error probabilities no value, i.e. $P(D'|Z) = P(D|Z') = 0$, which gives

$EVPI =$ 112 024€

If the *EVPI* is less than the cost of the investigations, the investigations are not worth performing since they can never generate more value than what they cost.

6.3 Conclusions, Publication I

- Although VOIA is a useful tool to structure thoughts and to formalise a decision process, the conditions and requirements that affect the decision-maker's priorities and attitude to risk vary between projects.
- VOIA in rock mass characterisation is site-specific and although the key questions are the same the answers may vary.
- VOIA focuses the decision process in rock mass characterisation on the most essential parameters.
- By focusing on the parameters that are crucial to the purpose of the characterisation, VOIA contributes to a more transparent process where each step is openly evaluated and the value of further investigations is compared with the present state of knowledge about the underground construction site. This leads to an investigation programme that is well adapted to the statutes of the Observational Method.

7. Publication II

In Zetterlund et al. (2009) another example of rock mass characterisation for grouting is demonstrated. The aim was to develop a method for using VOIA in pre-investigations for grouting in tunnels in hard rock. The method was illustrated in a generic case of a feasibility study of a tunnel constructed in crystalline rocks of the Fennoscandian Shield. Two alternative grouting design choices were available, one conventional design with cement grout and one extensive design with cement in combination with Silica-sol. Two questions were asked: Which of the alternatives is best suited to the geological conditions on site? Is information from investigations of value in making the decision? The value of new information from a core-drilled borehole was compared to the cost of drilling and measurement. Specifications of the alternative grouting designs are shown in Table 5.

Table 5. Specifications of alternative grouting designs.

	Null Alternative (Reference)	Alternative 1. Basic grouting design	Alternative 2. Extensive grouting design

Pre-excavation grouting			
Number of boreholes in one grouting fan	-	20	20 + 10
Type of grout	-	Cement	Cement + Silica sol
Post-excavation grouting (if the stated inflow requirement is not met)			
Number of boreholes	30	10	10
Type of grout	Cement	Silica sol	Silica sol
Additional measures	Drains	Drains	Drains

7.1 Conceptual geological model

The focus of the study was the methodology and the conceptual hydrogeological and geological models were thus rather simplified. The geology was assumed to consist of either rock with a need for grouting, or rock without a need for grouting. Larger deformation zones were of more interest than specific fractures.

The aspect ratio of length and width of a deformation zone was assumed to be approximately 1:10. The proportion of rock mass belonging to a deformation zone was assumed to be 10 per cent of the total rock mass volume. The dominating strike was assumed to be perpendicular to the direction of the tunnel.

7.2 Uncertainty in the grouting result

Irrespective of the choice of grouting design, there is uncertainty that the grouting would succeed in sealing the fractures sufficiently to meet the stated inflow requirement into the tunnel. The uncertainty in this example is described using a specific beta distribution for each alternative grouting design. The parameters deciding the characteristics of the distributions are α and β, as well as the minimum and maximum for the function. The minimum and maximum are assumed to be 0 and 1 for all grouting designs. The value 1 means that the design is sufficient to meet the inflow requirements and the value 0 represents that the design is not sufficient.

For example, the number of deformation zones sealed using the first alternative grouting design is represented by a beta distribution with the parameters $\alpha_1 = 7.1$ and $\beta_1 = 1$.

7.3 Stochastic simulation of rock mass

A geological model was made in the software T-PROGS, which uses transition probabilities and Markov chains in three dimensions for geostatistical analysis and stochastic simulation of spatial distributions of, for example, geological units. Input data for T-PROGS was mainly based on expert knowledge representing studies of geological maps and previous studies in this area in a real project.

The rock mass was divided into two classes:

Class 1. Rock mass without a need for grouting.

Class 2. Rock mass <u>with</u> a need for grouting (e.g. occurrence of a transmissive deformation zone).

The deformation zones in class 2 are seen conceptually as quadratic discs in the rock mass, with an aspect ratio between thickness and extension of approximate magnitude 1:10. Ten per cent of the rock volume in each direction is assumed to consist of zones. Input data for T-PROGS is shown in Table 6, Table 7, and Table 8.

Table 6. Input data regarding characteristic deformation zone values to Markov chains in the x-direction (width)

Material	Proportion	Width
Class 1	0.9	18.0
Class 2	0.1	2.0

Table 7. Input data regarding characteristic deformation zone values to Markov chains in the y-direction (length).

Material	Proportion	Length
Class 1	0.9	198.0
Class 2	0.1	22.0

Table 8. Input data regarding characteristic deformation zone values to Markov chains in the z-direction (depth).

Material	Proportion	Depth
Class 1	0.9	198.0
Class 2	0.1	22.0

Embedded transition probabilities were used in the model, and the transition probabilities of embedded occurrences, between class 1 and class 2, were calculated. When there are only two classes, the transition probability is equal to one; hence the transition probability matrices in all three dimensions are equal:

$$\mathbf{T}_x = \mathbf{T}_y = \mathbf{T}_z = \begin{pmatrix} 0 & 1 \\ 1 & 0 \end{pmatrix} \tag{10}$$

Stochastic simulation resulted in 200 realisations of the rock mass, Figure 4.

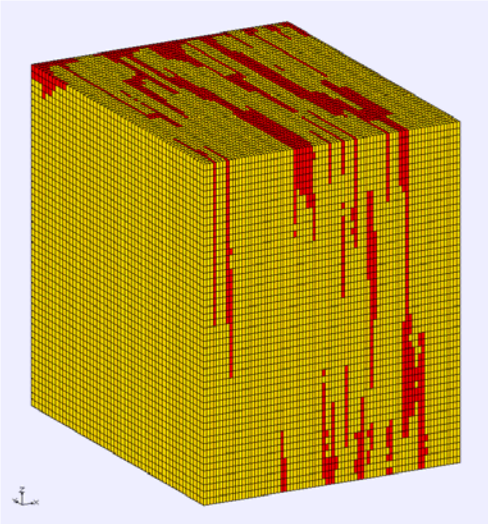

Figure 4. Example of one of 200 realisations of the rock mass.

7.4 Prior analysis

The purpose of the prior analysis was to decide which one of the alternative grouting designs was most suitable for the rock mass and the stated inflow requirements. A null alternative, with no pre-excavation grouting actions planned in-advance was used as a reference but was not treated as a possible alternative. The risk cost of the reference alternative includes the costs of post-excavation grouting in the null alternative, C_{E0}, and the costs of installation of drains, C_D. The risk cost was calculated as:

$$R_0 = q_0 \cdot C_{E0} + q_e \cdot C_D \tag{11}$$

where q_0 is the probability of post-excavation grouting, and q_e is the probability of drain installation, i.e. that post-excavation grouting does not seal the fractures successfully enough to satisfy the inflow requirement. These probabilities are based on the beta distributions, as well as q_i and q_{ie} below.

The risk cost of alternative i is:

$$R_i = q_i \cdot C_E + q_{ie} \cdot C_D \tag{12}$$

where q_i is the probability that the grouting design i fails, and that post-excavation grouting is needed, and q_{ie} is the probability of drain installation for alternative i. Note that $C_{E0} \neq C_E$ since post-excavation grouting in the null alternative involves much more work as no pre-excavation grouting has been carried out.

The benefits of alternative i are expressed as the difference between the reference risk cost, R_0, and the risk cost of the alternative, R_i:

$$B_i = R_0 - R_i \tag{13}$$

The object function of alternative *i* is expressed as the difference between benefits and costs for the alternative, such as:

$$\Phi_i = B_i - C_i \tag{14}$$

When two alternatives are compared, the prior value is:

$$\Phi_{prior} = \max(\Phi_1, \Phi_2) \tag{15}$$

The results of the prior analysis are shown in Table 9.

Table 9. Results from prior analysis in example II. All results are calculated per grouting fan (SEK).

Reference risk	R_0	277 000
Cost of pre-excavation grouting (alternative 1)	C_1	151 000
Risk cost (alternative 1)	R_1	108 000
Benefits (alternative 1)	B_1	169 000
Cost for pre-excavation grouting (alternative 2)	C_2	255 000
Risk cost (alternative 2)	R_2	13 000
Benefits (alternative 2)	B_2	264 000
Difference in benefit (alternative 1/alternative 2)	ΔB_{prior}	95 000
Difference in cost alt. (alternative 1/alternative 2)	ΔC_{prior}	104 000
Value alternative 1	Φ_{1prior}	18 000
Value alternative 2	Φ_{2prior}	9 000
Value prior analysis	Φ_{prior}	18 000

7.5 Preposterior analysis

The preposterior probability of deformation zones crossing the tunnel was found by means of ‘virtual’ drilling in all 200 realisations from T-PROGS, see Figure 5. The total length of the tunnel was divided into eight sections, each ten metres in length. The sections were divided into *high-risk rock* and *low-risk rock* based on the classes seen in the drillings (rock mass with a need for grouting/rock mass without a need for grouting). If there is any sign of a deformation zone in the section, the whole section is categorised as high-risk rock.

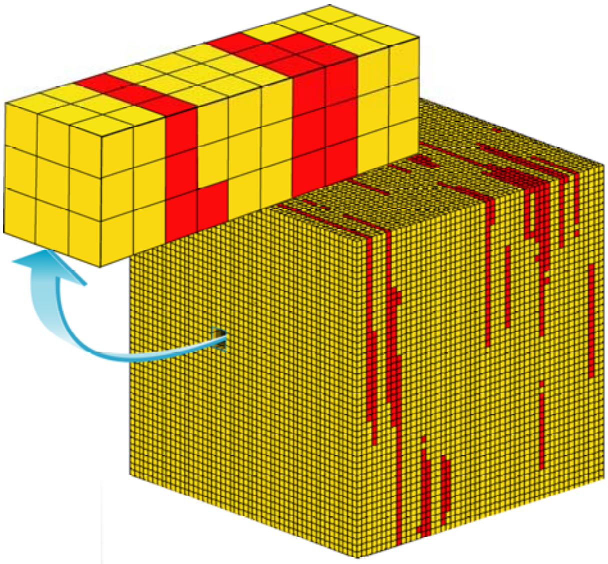

Figure 5. **Each section of the planned tunnel was represented by 3x3 cells along the y- and z-axes, and 10 cells along the x-axis. Virtual drilling was performed in the central cell of the planned tunnel section (from Zetterlund et al., 2009).**

Hence, data *X* consist of the two categories of rock that can exist in the tunnel, high risk rock, *H*, or low risk rock, *L*, i.e. $X = L,H$. All the prior probabilities can be calculated given *X*, and the reference risk in the preposterior analysis is:

$$R_0 = R_{0|L} \cdot P(L) + R_{0|H} \cdot P(H) \quad (16)$$

Where

$$R_{0|X} = q_{0|X} \cdot C_{E0} + q_{e|X} \cdot C_D \quad (17)$$

The risk cost of alternative *i* is calculated in the same way. The benefits of alternative *i* are:

$$B_{i|X} = R_{0|X} - R_{i|X} \quad (18)$$

in the cases of $X = L,H$.

The expected value of the preposterior analysis is calculated as:

$$E\Phi_{posterior|X} = \Phi_{posterior|L} \cdot P(L) + \Phi_{posterior|H} \cdot P(H) \quad (19)$$

The resulting expected Value of Information from Investigations (*EVI*) is calculated as:

$$EVI = E\Phi_{posterior|X} - \Phi_{prior} \quad (20)$$

The results of the preposterior analysis are shown in Table 10, and the results of the VOIA in Table 11.

Table 10. Results of the preposterior analysis in example II. All results calculated per grouting fan (SEK).

Null alternative		**High risk rock**	**Low risk rock**
Risk cost (null alternative)	$R_{0\|H}$; $R_{0\|L}$	444 000	138 000
Posterior Risk cost (null alternative)	$R_{0\,post}$	278 000	
Alternative 1			
Risk cost (alternative 1)	$R_{1\|H}$; $R_{1\|L}$	198 000	33 000
Posterior Risk cost (alternative 1)	$R_{1\,post}$	109 000	
Benefits (alternative 1)	$B_{1\|H}$; $B_{1\|L}$	246 000	105 000
Posterior Benefits, (alternative 1)	$B_{1\,post}$	169 000	
Posterior value (alternative 1)	$\Phi_{1\,post\|H}$; $\Phi_{1\,post\|L}$	95 000	-46 000
Posterior value (alternative 1)	$\Phi_{1\,post}$	18 000	
Alternative 2			
Risk cost (alternative 2)	$R_{2\|H}$; $R_{2\|L}$	26 000	2900
Posterior Risk cost (alternative 2)	$R_{2\,post}$	13 500	
Benefits (alternative 2)	$B_{2\|H}$; $B_2 \|L$	418 000	135 000
Posterior Benefits (alternative 2)	$B_{2\,post}$	265 000	
Posterior value (alternative 2)	$\Phi_{2\,post\|H}$; $\Phi_{2\,post\|L}$	163 000	-120 000
Posterior value (alternative 2)	$\Phi_{2\,post}$	9 900	
Posterior value	$\Phi_{post\|H}$; $\Phi_{post\|L}$	163 000	-46 000

Table 11. Results of VOIA in example II (SEK).

Difference in benefit (alternative 1/ alternative 2)	ΔB_H ; ΔB_L	172 000	30 000
Value prior analysis	Φ_{prior}	18 000	
Expected value posterior analysis	$E(\Phi_{post\|X})$	50 000	
Expected value of information	EVI	32 000	

7.6 Conclusions, Publication II

The main conclusions from this work are:

- VOIA can contribute to good structure in geological surveys when the geology is difficult to predict and when repeated updating is necessary during the course of a project.

- The prescribed method provides a tool to design well-motivated investigation programs where geotechnical value is weighed up against execution costs.

- The prescribed method also serves as a good basis for updating problems by quantifying the reduction in uncertainty in monetary terms. In doing so, the method facilitates the use of the Observational Method in underground construction projects.

The suggested method for application of VOIA in rock mass characterisation is:

1. Formulate the purpose of characterisation. Identify key questions.
2. Preliminary study of all available geological information for the area, such as maps, earlier investigations, etc.
3. Make a first conceptual model of the geological and hydrogeological conditions
4. Perform field mapping
5. Update the conceptual model
6. Perform a prior analysis
7. Make a stochastic model of geology in, for example, T-PROGS
8. Preposterior analysis
9. Compare the Expected value of information with investigation costs.
10. Make a decision regarding further investigations

8. Experience from the methodological examples

The methodological examples provided valuable experience of the work order of VOIA and in particular the statistical way of thinking. There is great potential in the use of the method in industry today; a well-performed VOIA is a good basis for decision-making in infrastructure projects and leads to decisions that are justified both finacially and geologically. At a first glance the statistical notations may be unfamiliar and discouraging for the engineer or geologist, which could obstruct implementation of the model. However, the mathematics behind the notations are elementary and it is worth spending some time on the notations.

The main difficulties are putting numbers to the probabilities and identifying the key parameters. To avoid complicating the task, the focus must be kept firmly on the key issues for the specific question for which the VOIA is being performed. The focus must also be kept on the important issues for each step in the design. Although it is tempting to try to solve everything at once, the VOIA must be solved step by step. Limitations are necessary, as are certain simplifications in the models.

9. Discussion

9.1 VOIA in Rock Mass Characterisation

Rock mass characterisation can be performed for many reasons and purposes although it has been shown earlier in this thesis that most problems in tunnelling can be traced back to two main sources, stability and water. In the initial phase of the characterisation process, a clear aim and purpose for the characterisation should be set up, and if a VOIA is to be made, the key parameters or critical factors for that purpose should be identified. The factors can be found by definition of failure, defined as an undesired state of nature or event, which in turn should be well thought-out in order to correspond to the purpose of the analysis. A good basis for identifying key questions, critical parameters and underlying processes in a specific project are lists of headings such as the one by Gustafson (2009) for hydrogeological purposes:

1. Construction
   - Understanding of the rock mass (geological model and prognosis)
   - Stability and groundwater inflow
   - Sealing of rock
   - High water pressure
   - Identification of parameters possible to observe and measure
2. Environment inside the tunnel
   - Working environment
   - Water-soluble gases (radon)
   - Requirements regarding dripping and moisture in a finished tunnel

3. Effect on the surroundings
   - Groundwater depression
   - Spreading of grout and contamination
   - Salt water intrusion and other water chemical effects
   - Discharge of process water and seepage of groundwater
4. Durability
   - Durability of grout, shotcrete and bolts
   - Corrosion and groundwater quality
   - Groundwater issues during operation and maintenance (infiltration)

The list of questions should be made early in the project, and the more effort that is put into the list the more problems can be minimised or even avoided.

The probabilistic approach, which is the strength of VOIA, is in fact also the most difficult part in the analysis. The probabilities are a way of expressing uncertainties regarding the geology and uncertainties in the investigation methods. When numbers are assigned to the probabilities, the whole process is considered and only by asking all the questions that need to be answered in this process is one of the aims of the method achieved, i.e. a contribution to a more structured and rational decision. To determine the probabilities, the key parameters of the problem need to be identified and translated into characteristic data for stability and water properties. In areas where quite extensive geological information is available, such as in urban areas where there are already many underground constructions, the probabilities can be assigned with more accuracy than in remote areas where no previous investigations have been conducted or where geological information is sparse. In the latter case, expert judgements and opinions are necessary.

It is difficult to go round the fact that many geological features/problems are spatially dependent. The data value of a measurement or an investigation is very much dependent on *where* the measurement is performed. In an investigation programme with limited resources, the whole investigation area should be assessed with as few measurements as possible. The investigation programme then often needs to target the expected weaknesses in the rock mass, which can lead to a negative bias. The value of the information gained from these investigations, such as core-drilled boreholes, not only depends on the cost of the drillings and the cost of failure; it is also affected of where the boreholes are located in the rock mass. For example, the data value of investigations in a borehole may be significant if it is close to a suspected fracture zone, but unimportant in another location where there is greater certainty regarding the geology. This fact complicates the VOIA but if the investigation locations are determined before the calculations of the data value commences, the VOIA will be valid for investigations at those particular spots.

9.2 Implications of the Observational Method

Does the Observational Method lead to any differences in the characterisation procedure compared to the usual procedure used in industry today? The requirements of a geological description in Swedish standards for tunnelling (Vägverket, 2004; Banverket, 2005) are not in contradiction with the Observational Method. However, there are other implications of the method which will be discussed below. A common misconception is that when working according to the Observational Method the effort put into the pre-investigations does not need to be as great as would normally be the case. This is not true. The original definition of the method (Peck, 1969) includes the need to fulfil two conditions. Whilst these are not mentioned explicitly in Eurocode 7 (CEN, 2004), they are no less important. These are:

a) Exploration sufficient to establish at least the general nature, pattern and properties of the deposits (geology, *authors comment*), but not necessarily in detail.
b) Assessment of the most probable conditions and the most unfavourable conceivable deviations from these conditions. In this assessment geology often plays a major role.

In order to work out the design for the most probable geological and geotechnical conditions, the geology needs to be well known and investigations are as important as in any tunnel project. The main difference is that the initial design should be adapted to the most probable geotechnical/geological conditions and not to a worst case scenario. This is the main advantage of the method, as it will reduce expensive, overly-conservative designs when the tunnel is designed for the most probable conditions of the rock mass. This will be followed up by measurements (observations) of the critical parameters and continuous updating of the geological model. If the observations show deviations from the predicted behaviour, an in-advance prepared contingency action will replace the initial design alternative, as stated in point f) in the list by Peck (1969). This is a new approach which should not be confused with the concept of *design as you go* since the contingency plans should be well defined before the start of construction.

Two other points in Peck's (1969) list of conditions that need to be fulfilled are to do with the observations:

a) Exploration sufficient to establish at least the general nature, pattern and properties of the deposits but not necessarily in detail.
g) Measurement of quantities to be observed and evaluation of actual conditions.

When implementing the Observational Method, this could involve some difficulties. Since many parameters in tunnelling projects can only be measured indirectly, it could be difficult to find relevant observable and measureable parameters, e.g. hydraulic conductivity is represented by flow and pressure. Failure in ductile rock is predicted with deformation measurements. However, in rock masses with brittle failure mechanisms this can be problematic since failure is not preceded by deformations. It is important that an alarm level is set early to allow sufficient time in the system to take action before the critical level is reached. These parameters need to be thought of in the pre-investigation phase.

In a project where the Observational Method is applied, updating of the geoscientific conditions are made repeatedly and there is a need for close contact and good co-operation between the site geologist, the design engineer and the contractor. The exchange of information and knowledge between different stakeholders in the project is important to get the updating procedure to work smoothly. The work order in a project where there is such close contact between the design engineer, the site geologist and the contractor requires a great deal of effort regarding the financial aspects of the contracts. The updating procedure and the alteration of the design make the tendering process difficult and call for new contract forms. The contractual issues are discussed in more detail in Kadefors and Bröchner (2008).

When disagreements in a tunnelling project arise, the geological prognosis and its interpretation are often a major topic. Naturally, it is not possible to make a general statement of the reasons for these disagreements, yet it is worth bearing in mind that these prognoses are communicated between a variety of people with different skills and different backgrounds. When the Observational Method is applied, one of the main challenges is the communication between all the stakeholders in the project, e.g. the geologist, the contractor, the proprietor, and the design engineer. As stated above, it is absolutely vital for the updating process that this communication is smooth and easy.

The use of classification systems is wide spread and accepted in the industry. According to Swedish standards for tunnelling (Vägverket, 2004; Banverket, 2005) geological prognoses should include information about the rock mass quality in a classification system. The advantage of classification is that it is a way of simplifying the geological information in the project. On the other hand, the result of the classification is a generic number, where the original properties of the rock mass are concealed. During the course of the project, the classification should be seen as part of a working hypothesis and it should be updated in the same way as the other geological information. There are uncertainties involved in the initial classification of the rock mass and there is also an uncertainty in the fact that support measures prescribed for a class could be found to be inappropriate for some or all sections belonging to that particular class.

Mapping in the tunnel is made by the site geologist and the interpretation of the rock mass and the level of detail can be affected when the time pressure is high. The geologist is usually engaged by the client but should act independently. Nevertheless, situations may easily arise where there is pressure on the geologist to change his or her mind for financial reasons. The geological mapping is delivered to the management of the tunnel site and to the contractor as drawings, which are the basis for the design of the reinforcements. Close contact between the geologist, the management and the staff in the tunnel is vital when the Observational Method is applied. Everyone involved needs to be aware of the importance of the geological conditions, and it is important that everyone involved strives towards a technical solution that is as optimal as possible.

10. Conclusions

The thesis presents how a decision-maker can prepare for a continuous up-dating process at the exploration phase of a project.

'Characterisation' should not be confused with 'classification'. In rock mass characterisation the condition of the undisturbed rock mass is described and parameters governing or influencing the rock mass behaviour are described and quantified. Classification is performed for direct application to an underground construction. It is a way of simplifying the geological information and can be a help in organising and obtaining a better overview of data. If classification has been carried out properly, it may simplify the process and ease communication.

Communication and transfer of knowledge between different persons and stakeholders in the project is in fact one of the difficulties when the Observational Method is applied. However, this knowledge transfer is essential for an efficient updating procedure.

A rock mass characterisation process needs to be focused on the problem to be solved and the parameters needed for that specific problem. The question of whether further investigations, or measurements, are beneficial or not is not only governed by the value of the latest measurement, but also by the uncertainties in the process itself. In some cases the uncertainties can be of such magnitude that they override the value of making further investigations.

VOIA makes the decision process in rock mass characterisation more focused on the most essential parameters. By focusing on the parameters crucial to the purpose of the characterisation, VOIA contributes to a more transparent process where every step is evaluated, and the value of further investigations is compared with the present level of knowledge regarding the underground site.

The structure of a VOIA and the mathematics it involves are quite straightforward and yet the statistical notations may be unfamiliar. Development of computer aids for VOIA calculations would simplify the use of the method in project planning and construction and in doing so facilitate implementation of the method.
The costs included in the cost-benefit analysis are presumably already known by the decision-maker but as stated previously, the difficult part is to assign values to the probabilities involved. In one example in this study, the probabilities were based solely on expert knowledge. In a second example, it was shown that a stochastic model of the rock mass can be a basis for the probabilities. In the latter example, the uncertainty in the grouting result was represented by a beta distribution.

A purpose driven rock mass characterisation will, using VOIA, contribute to a transparent decision procedure, and to an investigation programme that is well adapted to the statutes of the Observational Method.

The theory of decision analysis is already well developed, although the link to rock mass characterisation is not as developed. Very few examples have been found where decision analysis has been used to its full extent in rock engineering projects. A test of the method in real, on-going projects is necessary.

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ROCK ENGINEERING RESEARCH FOUNDATION
STIFTELSEN BERGTEKNISK FORSKNING

Deformation and Failure of Hard Rock under Laboratory and Field Conditions

Kelvis Pérez, Luleå University of Technology

BeFo Rapport 95

Deformation and failure of hard rock under laboratory and field conditions

(Licentiate Thesis ISBN 978-91-86233-61-7)

PREFACE

This is a summary of the licentiate thesis work. The licentiate work forms part of a project portfolio consisting primarily of three PhD research projects related with the requirements of Eurocode 7. This work is concentrated on deformation and fracture process of hard rock masses. The project portfolio is a collective research effort between the Rock Engineering Research Foundation (BeFo), the Swedish Rail Road Administration (Banverket), Svensk Kärnbränslehantering AB (SKB), the Royal Institute of Technology (KTH), Chalmers University of Technology and Luleå University of Technology (LTU). The financial support was provided by Banverket, SKB and BeFo.

There are a number of people who deserve my thanks. This licentiate thesis work would not have been possible without them. Special thanks go to my supervisors Professor Erling Nordlund at the Division of Rock Mechanics (LTU), and Adjunct Professor Jonny Sjöberg at LKAB for their supervision and direction as to how to develop this licentiate work. I would like to express my gratitude to my project reference group for their suggestions and interesting discussions. They are Tomas Franzen and Mikael Hellsten (formerly and currently at BeFo, respectively), my supervisors, Olle Olofsson (Banverket), Rolf Christiansson (SKB), Håkan Stille (KTH), Anders Fredriksson (Golden Associates), Lars O. Ericsson (Chalmers), Mats Holmberg (Tunnel Engineering), Miriam Zetterlund (Chalmers), and Mehdi Bagheri (KTH). I wish to thank those who contributed with data for the underground cases: Professor Derek Martin of the University of Alberta, Health and Safety Executive (HSE), John Anderson and Guy Lance, Jimmy Töyrä at Banverket, and Christer Andersson at Vattenfall Power Consultant AB. My gratitude goes also to Bo Carlsson (Banverket) for data from laboratory tests carried out at LTU. I am also grateful to Professor Inge Söderkvist (LTU), for help with the polynomial fit of crack volumetric strain data.

SUMMARY

The understanding of the fracture mechanisms and failure processes of the rock is an important requirement for the design of mining excavations and civil engineering constructions. The fracture process is necessary for the excavation and fragmentation of rock, but fracture of rock must be avoided and controlled to preserve the integrity of the construction. This licentiate thesis work is focused on hard rock masses and conditions typical on Fennoscandia. This work was initiated with a review of literature. Following the literature review, information about real underground excavations with deformation monitoring was collected. Laboratory test data was also collected after the survey of cases. The last task for the licentiate work was to perform numerical analysis simulation using the program *Phase2* and evaluate the strains due to possible failure.

The literature review showed that fracture of brittle rock is the process by which new surfaces in the form of cracks are formed in rock-like material, or existing crack surfaces are extended. Five stages of deformation are distinguished in the fracture process of brittle rock: crack closure, linear elastic deformation, fracture initiation, fracture propagation and post-peak behaviour. The most common reason for stability problems in underground excavation is structurally controlled failure and stress-induced failure. The ground response curve is a technique for describing the response of rock under parameters such as deformation and stress. Thus, the response of the rock mass response can be evaluated and related to the distance to the face of the excavation. The failure criteria reported in the literature are formulated in terms of stresses and include one or several parameters that describe the rock mass properties. Only a few failure criteria were formulated in terms of strains. Since macroscopic failure surfaces are characterized by strain concentrations, fallout criteria should be expressed in terms of strain quantities. Further studies have to be done in order to be able to formulate strain-based fallout criteria. The four underground cases with hard rock mass and conditions typical of Fennoscandia are: Mine-by Experiment, Instrumented drift at the Kiirunavaara mine, Arlandabanan tunnel and Äspö Pillar Stability Experiment. These cases contain very good information regarding rock properties, geology and stress state. These cases are a good example of *in situ* deformation measurement. For some cases, the failure occurred and the measured deformation is related to the failure. Laboratory tests of hard rock specimens were performed at Luleå University of Technology and by Posiva Oy. The tested rocks are Fennoscandian types such as limestone, quartzite, diorite, norite, gabbro, diabase, syenite porphyry, mica gneiss, tonalite gneiss and a variety of granites. In these tests, the rock properties and stages of deformation (crack closure, crack initiation and crack damage) were measured and determined. The evaluation of the laboratory tests showed that the stages of deformation vary between rock types and depend on factors such as grain size and mineral composition. Therefore, it may be better if each rock type is treated individually. Failure (i.e., intersection of shear bands forming a v-notch) of a real case and fictitious case was simulated using *Phase2*. The evaluation of predicted quantities such as maximum and minimum principal, volumetric and maximum shear strains along the depth of the v-notch showed good agreement with the point where the v-notch ended.

Keywords: Hard rock, failure process, deformation stages, strain, deformation measurement, underground cases, laboratory tests, numerical modelling.

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1 INTRODUCTION

1.1 Background

The design of underground excavations such as tunnels and rock caverns is dependent on the material itself, the imposed disturbance due to the excavation development, state of stresses and rock properties. The disturbance caused during the excavation process generates deformation (i.e., strains) and growth of fractures in the rock mass. It may result in stability problems leading to minor failure and/or to fallout and collapse of the excavation (Figure 1.1). For that reason, it is important to increase the knowledge of the failure process in order to facilitate the evaluation of the excavation stability.

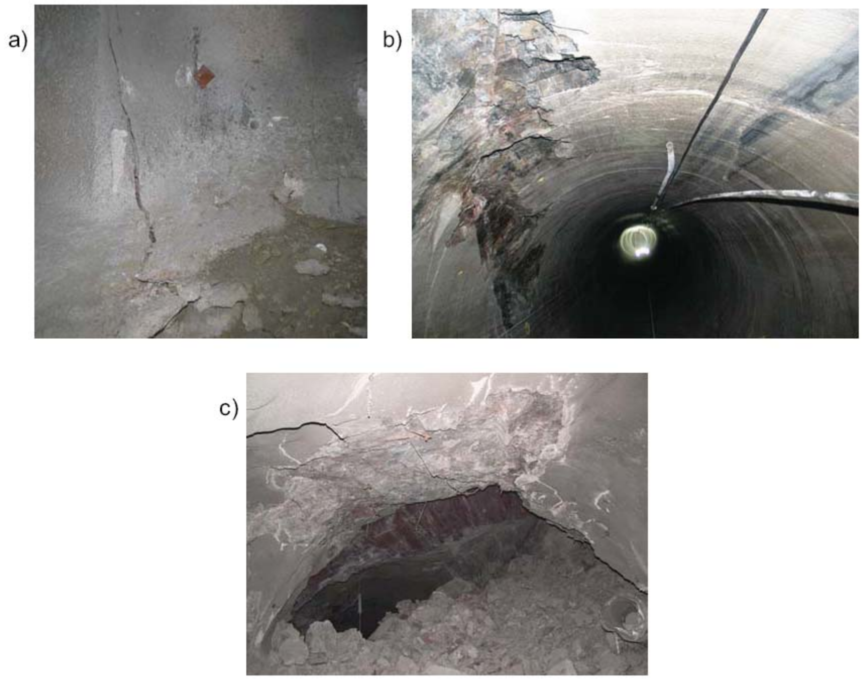


Figure 1.1 Different failures: a) visible cracks in rock support at the Kristineberg mine, b) spalling failure in Garpenberg mine raise, and c) collapse of a Kiirunavaara mine drift (Edelbro, 2008).

The growth of fractures is described by the failure process. The failure process of brittle rock has been studied by many researchers such as Bieniawski (1967); Martin and Chandler (1994); Hakala and Heikkilä (1997a,b); Heikkilä and Hakala (1998a,b); Eberhardt et al.,(1998); Eloranta and Hakala (1998, 1999a,b); Eberhardt et al., (1999); Read (2004) among others. Laboratory tests and observations *in situ* have been the platform of these studies, which have shown that the failure process can be divided into different stages of fracture.

The stress-strain behaviour in Figure 1.2 illustrates the fracture process of a specimen of intact granite that was tested in laboratory by compressive loading. The failure process of this particular specimen begins with closing of cracks and finish with its maximum strength. In studies at the Underground Research Laboratory (URL) the crack initiation and crack damage stress were used to better quantify rock damage. The progressive failure process due to spalling in the Mine-by Experiment is shown in Figure 1.3.

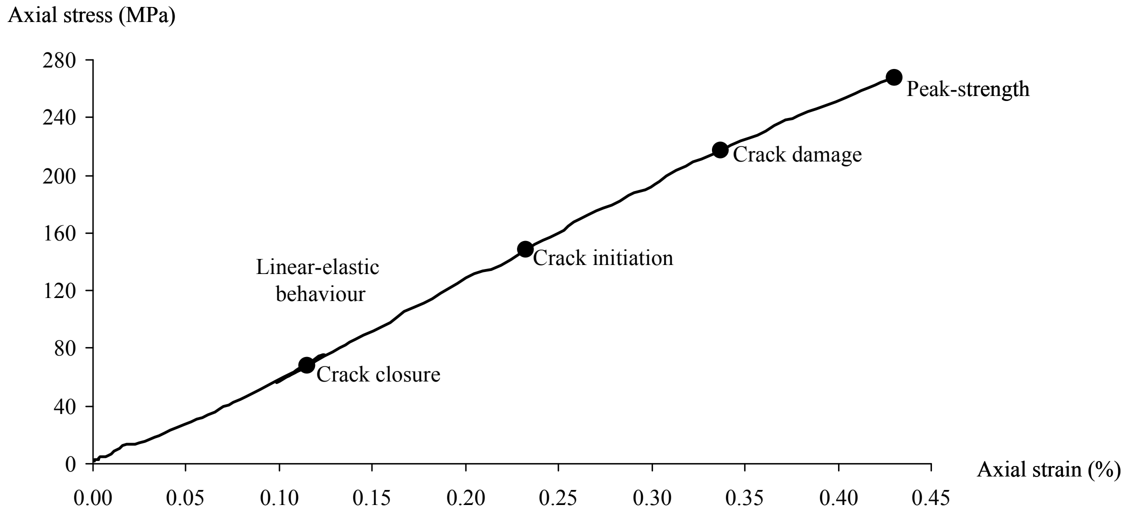


Figure 1.2 **Axial stress-axial strain curve showing stages of deformation of a specimen of granite from Kuru, Finland tested uniaxially in laboratory.**

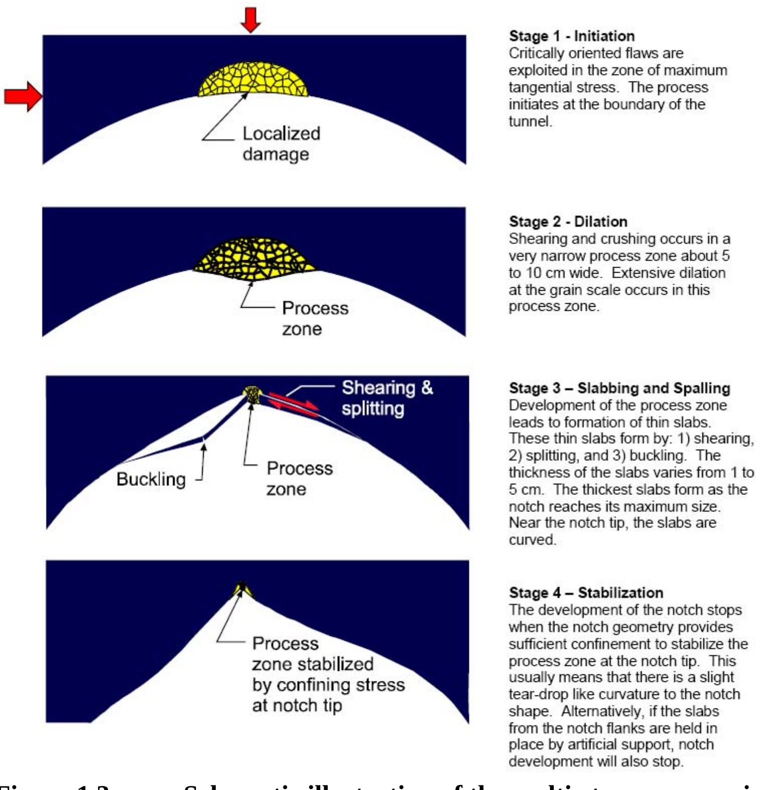


Figure 1.3 **Schematic illustration of the multi-stage progressive failure process observed in the MBE test tunnel (Read, 2004).**

The behaviour of a rock construction is normally assessed by deformation monitoring and damage mapping, which is often conducted in underground excavations. Therefore, a connection between observable and predictable behaviour could be assessed. But, the behaviour evaluation would also require a failure criterion based on deformation quantities. However, a review and evaluation of the literature regarding existing rock failure criteria and their respective parameters showed that the majority of the failure criteria are formulated in terms of stresses (shear stress-normal stress or major principal stress-minor principal stress relations). Furthermore, they include one or several parameters that describe the rock mass properties. The study also showed that only a few failure criteria had been formulated in terms of deformation quantities.

1.2 Objectives

The objective of the licentiate thesis is to increase the understanding of the deformation at different stages of failure process. There is a relation between the deformation (i.e., strain) caused by loading and/or excavation process and the failure process which produces rupture and/or failure of the material like rock and/or the underground excavation. The objective of the licentiate work was also to evaluate strains due to deformation for (i) real underground excavation with failure, (ii) rock specimens loaded up to peak stage in laboratory tests, and (ii) underground excavations where failure is predicted using numerical simulation.

1.3 Scope and limitations

The licentiate thesis concerns to deformations related to the rock failure process. The work is focused on hard-brittle rock masses and conditions typical for Fennoscandia (i.e., high-strength crystalline rock). The thesis work is primarily concerned with stress-driven failure mechanisms such as shear and spalling failure. Gravity driven failures such as falling and sliding of blocks are only treated in the review of cases studies. The cases and laboratory tests presented in the licentiate thesis aimed at describing and representing the rock strain behaviour related to failure in (i) macro scale such as tunnels and drifts in underground excavations, and (ii) small scale such as rock specimen in laboratory tests. The underground cases were selected on the basis that measurement of deformation was conducted. The laboratory tests were selected on the basis that the stress-strain behaviour of the rock-specimen was measured during the loading procedure. In the licentiate thesis work, the stress-strain behaviour comprises the pre-peak behaviour of the rock such as crack closure, crack initiation, crack propagation and peak-strength. The post-peak behaviour of the tested rock was not studied. The tested rock types comprise only rock types from Swedish and Finnish sites such as limestone, quartzite, diorite, norite, gabbro, diabases, syenite porphyry, mica gneiss, tonalite gneiss and a variety of granites. The numerical analyses aimed at evaluating strain due to spalling and/or shear failure. In this work, a geomechanical sign convention is used, i.e., compressive stresses are positive and tensile stresses are negative.

1.4 Approach

Figure 1.4 shows the general outline of the licentiate thesis work according to the objectives. It includes a review of literature and underground cases and evaluation and interpretation of laboratory tests as well as numerical simulations.

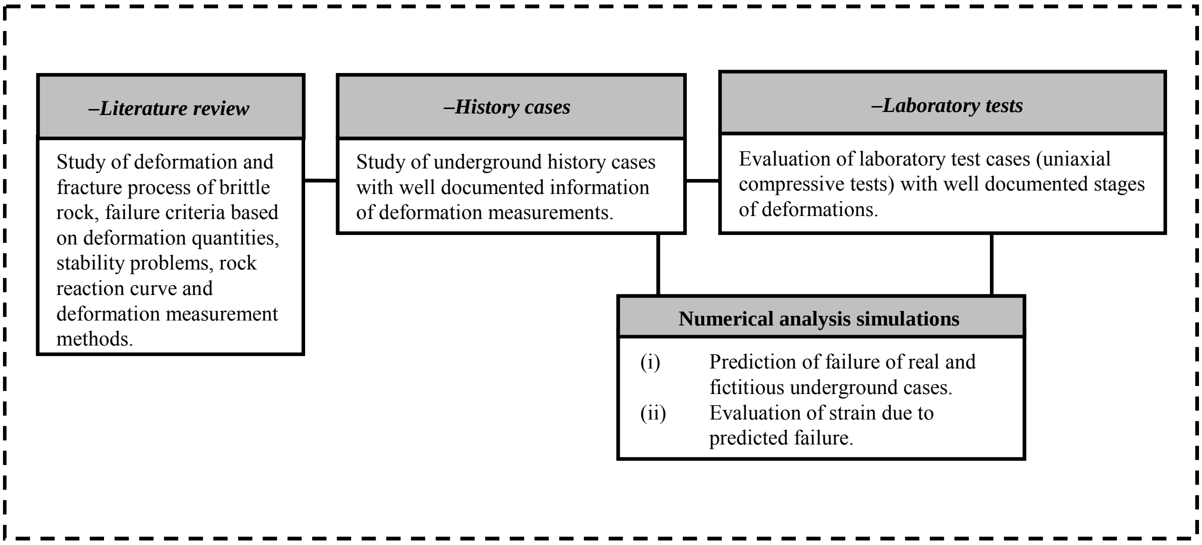


Figure 1.4 General outline of the licentiate thesis.

1.5 Outline of thesis

The licentiate thesis comprises seven chapters. The following main chapters were arranged as described below.

- Chapter 2 is a literature review of the fracture process, failure criteria based on strain quantities, causes of stability problems in underground excavations, ground reaction curve, and methods for deformation monitoring.
- Chapter 3 is a case study review of deformation measurements in tunnels and drifts.
- Chapter 4 is a laboratory tests review of deformation stages of the fracture process of some hard rock specimens. This chapter also comprises evaluation of the laboratory test data.
- Chapter 5 is a numerical analyses simulation of failure using the program *Phase2*. Two cases are presented: (i) a real case of failure in a vertical raise, and (ii) a fictitious tunnel case. In this chapter the principal strains that are caused by the predicted failure are evaluated.
- Chapter 6 include a discussion for each chapter and conclusions of the entire work. It is also reserved for the licentiate thesis recommendation. This part was aimed at giving suggestions for further rock mechanics research within this field.

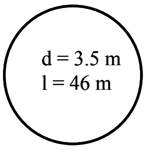
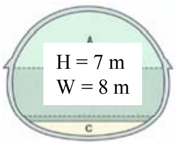
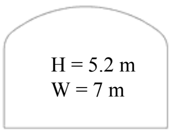
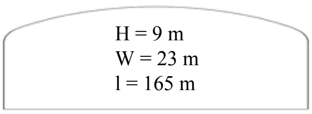
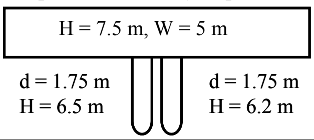
2 CASE STUDIES REVIEW

This Chapter is a review of underground cases. Each case is described in as much detail as possible with respect to rock properties, geology and geological structures, state of stress, failure and deformation measurement. The cases were selected on the basis of the requirement that measurement of deformation has been conducted. Five cases in total are presented: (i) Mine-by Experiment, (ii) Heathrow airport tunnel collapse, (iii) Instrumented drift at the Kiirunavaara mine, (iv) Arlandabanan tunnel – Shuttle station 2, and Äspö Pillar Stability Experiment. The first two cases are tunnels from abroad (Canada and England). The tunnel in Canada is an experimental excavation. The last three cases are from Sweden, comprising one mine and two tunnel cases. One of the Swedish tunnels, the Äspö Pillar Stability Experiment, is an experimental excavation.

2.1 Observed failure and measured deformations

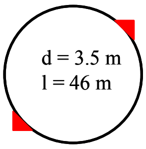
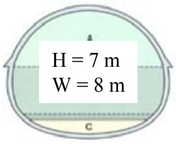
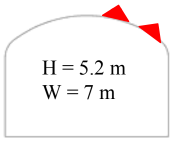
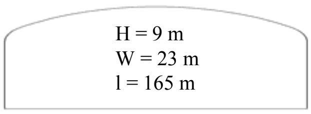
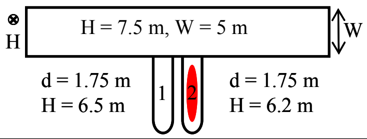
Based on the information presented in Table 2.1 and Table 2.2 the conclusion is that these cases provides good information about the stress state, failure development and deformation. Moreover, the failure at the Mine-by Experiment and the Äspö Pillar Stability Experiment was spalling (i.e., stress induced failure). The Heathrow airport tunnel collapse case is a good example of collapse of large dimension and economical consequences in softer ground. The Arlandabanan tunnel is a case with monitoring of displacement in various sections along the station during the construction of the station. Therefore, the failure and displacement data of these cases can be used as a reference for numerical simulation. Furthermore, the overburden for each case is different as well as the geometry of the excavation, rock types and state of stress. As a result of these factors, the failure mechanism is different for each case.

Table 2.1 Summary of information concerning rock type, geology, structure and excavation technique for each case.

Case	Depth	Rock type	Geology & Structure	Excavation technique
MBE 	420 m level	Granite	Massive without joints	Non explosive
Heathrow airport tunnel collapse 	30 m to the invert	Soil	London clay	-
Instrumented drift at Kiirunavaara mine 	514 level (285 m below horizontal ground surface)	Syenite porphyry	Three joint sets	Drilling and blasting
Arlandabanan tunnel 	11 m overburden	Mica schist and mica gneiss	Two large structure	Drilling and blasting
Äspö Pillar Stability Experiment 	450 m level	Äspö diorite	Slightly fractured	Drilling and blasting (drift) TBM (holes)

d = diameter, l = tunnel length, W = width, H = height.

Table 2.2 Summary of information concerning rock properties, stress state, failure and deformation monitoring for each case.

Case	Rock properties*	Stress states	Failure	Displacement monitoring
MBE 	Table 3.2	σ_1 = 60 MPa σ_2 = 45 MPa σ_3 = 11 MPa	Spalling (v-notch) in the roof and floor	Extensometer and convergence arrays in the round and chainage with spalling
Heathrow airport tunnel collapse 	London clay	Modelled using FEA	Collapse of the entire tunnel	Extensometer near the area of collapse
Instrumented drift at Kiirunavaara mine 	Table 3.13	σ_1 = 0.041z = 12 MPa σ_2 = 0.031z = 9 MPa σ_3 = 0.021z = 6 MPa	Block falling and open fractures in footwall abutment	Extensometer, distometer and telescopic extensometer in the abutment with block falling
Arlandabanan tunnel 	Table 3.19	σ_H = z/5.27 = 2 MPa σ_h = z/10 = 1.1 MPa σ_v = z/ρgz MPa	No failure	Extensometer and convergence array.
Äspö Pillar Stability Experiment 	Table 3.24	σ_1 = 25 to 35 MPa σ_2 = 15 MPa σ_3 = 10 MPa	Spalling (v-notch) on the wall of hole 2	LVDT on the wall of hole 2 with spalling

d = diameter, l = tunnel length, W = width, H = height. *Licentiate thesis.

3 LABORATORY TESTS REVIEW

This chapter presents a review and evaluation of two groups of laboratory tests of hard rock specimens typical of Fennoscandia. One group of laboratory tests was performed by the Rock Test Laboratory (RTL) at Luleå University of Technology (LTU) [Carlsson and Nordlund (2009a,b); Carlsson et al., (1999); and Carlsson (2009)]. The other group of laboratory was commissioned by Posiva Oy and performed by the Laboratory of Rock Engineering (LRE) at Helsinki University of Technology (HUT) [Hakala and Heikkilä (1997a,b); Heikkilä and Hakala (1998a,b); Eloranta and Hakala (1998, 1999a,b)]. No laboratory tests were carried out in this work, only the results of previously conducted test were used and evaluated. The tests were evaluated in order to determine the strain at each deformation stage.

3.1 Critical strains

From Figure 3.1 and Figure 3.2 the following can be concluded:

- The critical normalized axial strain values was found to be at the same level for all tested rock types, with some exceptions such as (i) crack initiation axial strain for limestone and syenite porphyry, and (ii) crack damage axial strain for limestone, Romuvaara tonalite gneiss and syenite porphyry.

- The normalized crack initiation axial strain for the metamorphic rocks seems to be at the same level which is not the case for the crack damage axial strain.
- The critical normalized axial strain values among igneous rocks shows more scatter compared to the metamorphic rocks. It may be due to the small number of tested metamorphic rock.

The same analysis for critical normalized lateral strain values shows that these parameters are very dependent on the rock type and the rock characteristic. There is more scatter for the normalized lateral strain values compared to the normalized axial strain values.These findings together with the standard deviation of the strain value show that each rock type behaves very differently. Each rock types must therefore be treated individually. Thus, critical values of axial and lateral strain for each rock type and for each stages of the failure process can be summarized as illustrated in Figure 3.1 and Figure 3.2. The maximum exhibited strain, for each rock type, at the peak strength stage of the failure process is also shown in Figure 3.3.

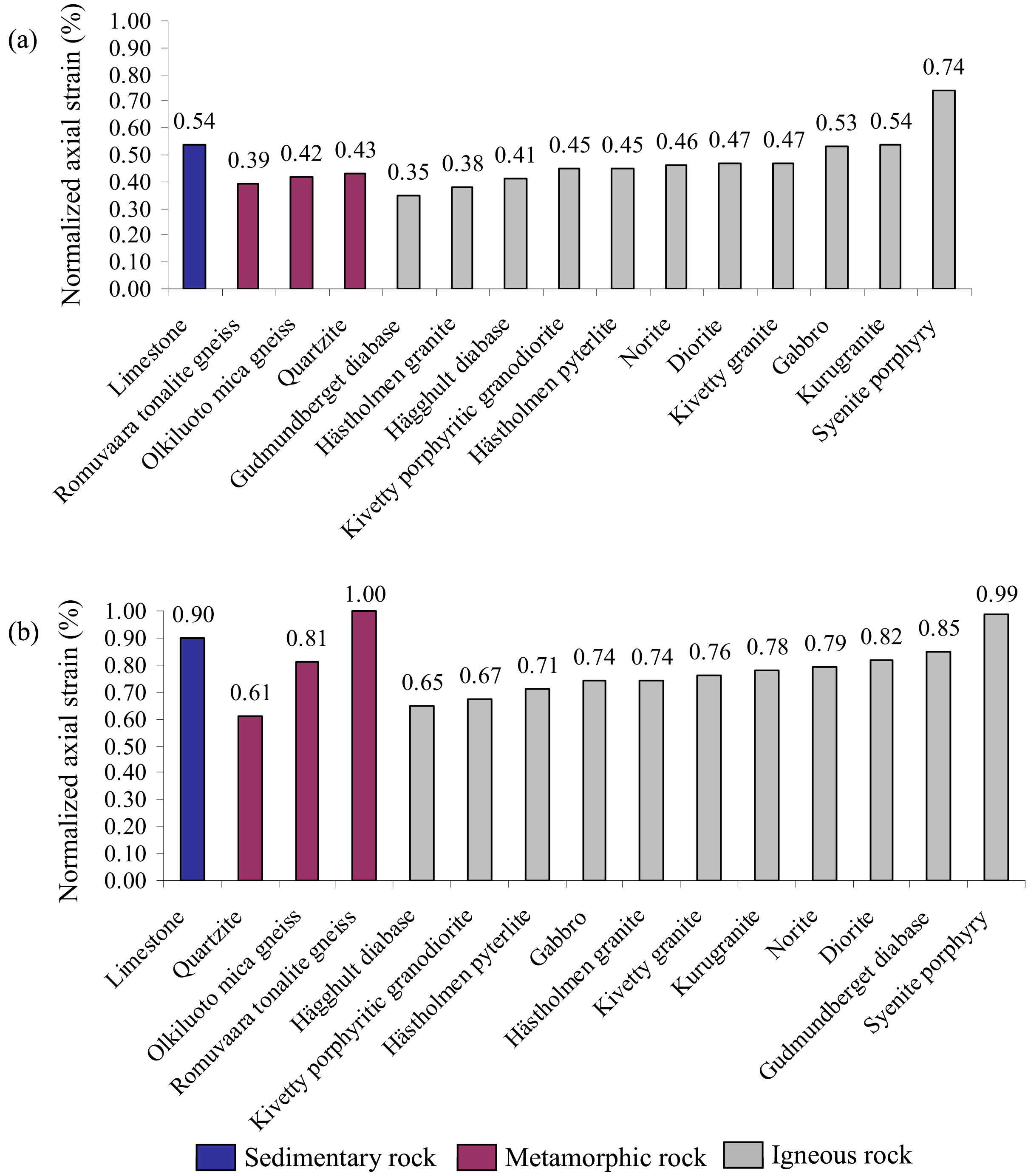


Figure 3.1 **Critical normalized axial strain values at (a) crack initiation, and (b) crack damage stages.**

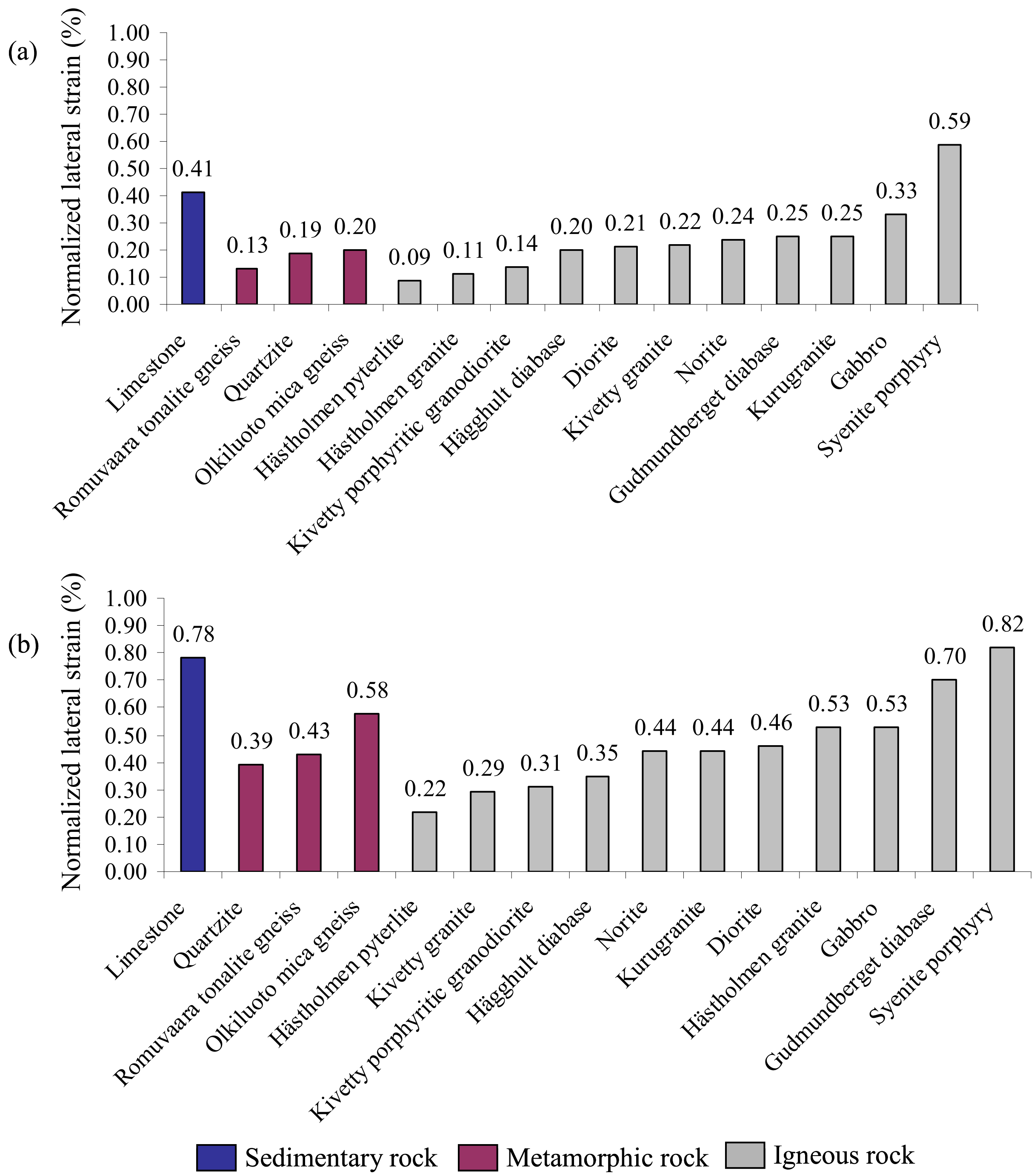
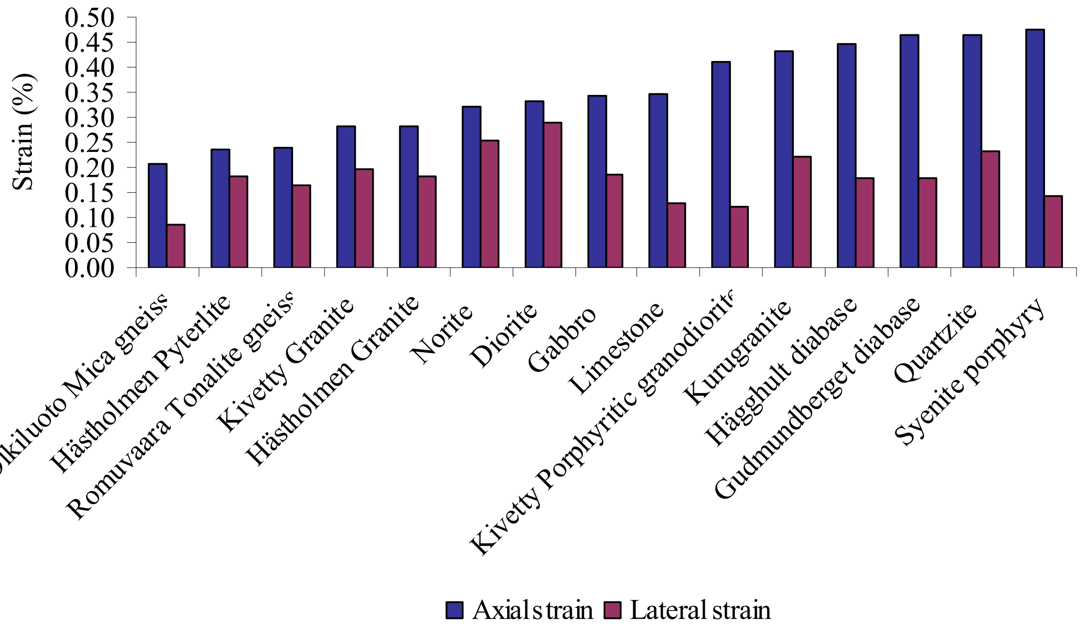


Figure 3.2 **Critical normalized lateral strain values at (a) crack initiation, and (b) crack damage stages.**



Rock type	ε_{1p} (%)	ε_{3p} (%)
Olkiluoto Mica gneiss	0.207	0.084
Hästholmen Pyterlite	0.236	0.183
Romuvaara Tonalite gneiss	0.240	0.163
Kivetty Granite	0.282	0.195
Hästholmen Granite	0.282	0.181
Norite	0.321	0.253
Diorite	0.331	0.288
Gabbro	0.343	0.184
Limestone	0.348	0.129
Kivetty Porphyritic granodiorite	0.411	0.123
Kurugranite	0.431	0.221
Hägghult diabase	0.445	0.180
Gudmundberget diabase	0.463	0.180
Quartzite	0.466	0.231
Syenite porphyry	0.475	0.144

Figure 3.3 Maximum axial and lateral strains at peak strength stage.

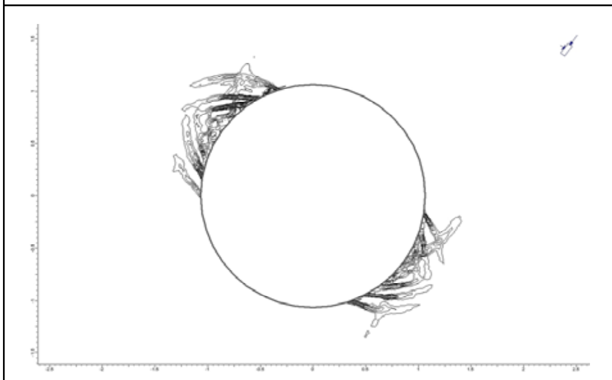
4 NUMERICAL MODELLING

In this chapter the strains around underground opening are evaluated. The failure is predicted in a fictitious case and real case using a linear-elastic, linear-elastic perfectly plastic and, linear-elastic brittle plastic material models. *Phase2* (Rocscience, 2009) is a two-dimensional elasto-plastic finite element program. This program was chosen because it is easy to use and widely applied to study mining and geotechnical problems. For the real case, the model for the Garpenberg raise performed by Edelbro (2008) was used. The strain at spalling failure in the raise was evaluated. This case was chosen because the excavation geometry was simple, and because the failure had been fairly successfully replicated in the work by Edelbro (2008). For the fictitious case the virgin stress state and rock mass properties correspond to those of a Zinkgruvan mine case studied by (Edelbro, 2008). The volumetric strain and the maximum shear strain can be calculated in *Phase*2. In this study the volumetric strain and the maximum shear strain have been used to calculate the principal strains ε_1 and ε_3.

4.1 Garpenberg raise I

The spalling of the rock is a gradual process that ends up in a final form that is most often v-notch shaped. Fallout due to shear is assumed to occur when two shear bands intersect the excavation boundary forming a coherent arch (Edelbro, 2008). In this work, the term v-notch is used as the intersection of shear bands. The use of this term is independent of the failure mechanism (spalling and/or shear failure) created by the shear bands. For the case of spalling, the primary failure mechanism is extensional splitting. However, as discussed by Edelbro (2008), a secondary failure mechanism may be shearing, thus justifying the use of shear bands as a failure indicator. Figure 4.1 shows the predicted v-notch formed in the roof of the raise. A perpendicular line to the boundary was defined in the v-notch region in order to collect data of volumetric strain and maximum shear strain and thus to calculate the maximum and minimum principal strains. Three points were defined along the line at (a) boundary, (b) assumed maximum v-notch depth, and (c) a point far from the boundary where the v-notch ends.

V-notch



V-notch with perpendicular line

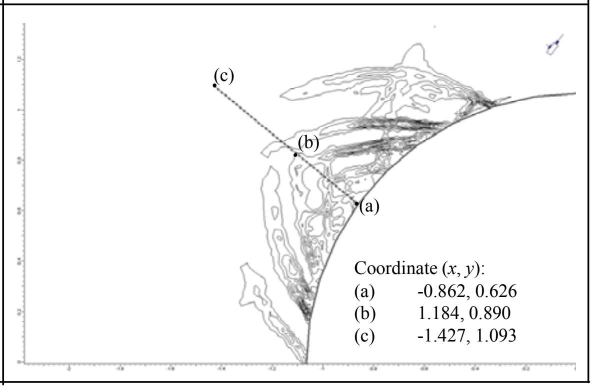


Figure 4.1 **V-notch and line in the roof of the raise using a linear-elastic brittle plastic material model.**

The maximum and minimum principal, volumetric and maximum shear strains versus distance from the raise boundary are plotted in Figure 4.2 and Figure 4.3 for the linear-elastic and elastic brittle model, respectively.

Linear-elastic material model

The evaluation of the linear-elastic material model shows that these quantities are very smooth along the depth of the v-notch. However, they are slightly large close to the boundary.

Linear-elastic brittle plastic material model

The elastic brittle model shows that these quantities have large values close to the boundary of the raise. They decrease with the distance from the boundary of the raise. Moreover, the absolute value of ε_1, ε_3 and γ_{max} are similar, i.e., with almost simultaneously maximum and minimum values along the depth of the v-notch depth. These quantities show this behaviour (maximum and minimum) as long as the v-notch (point b) exists. The intersection between minor shear bands within the v-notch produces this behaviour. After point (b) these quantities tend to be constant. The volumetric strain is negative from the boundary up to the point (b).

The predicted depth of the v-notch in point (b) is 0.41 m and the observed failure depth was 0.05 m.

The minimum principal strain was chosen as the critical quantity because it is a good indicator of crack development. The minimum principal strain using the linear-elastic and the elastic brittle-plastic model are compared (Figure 4.4). The comparison shows a smooth elastic curve while the elastic brittle curve has maximum and minimum values along the v-notch.

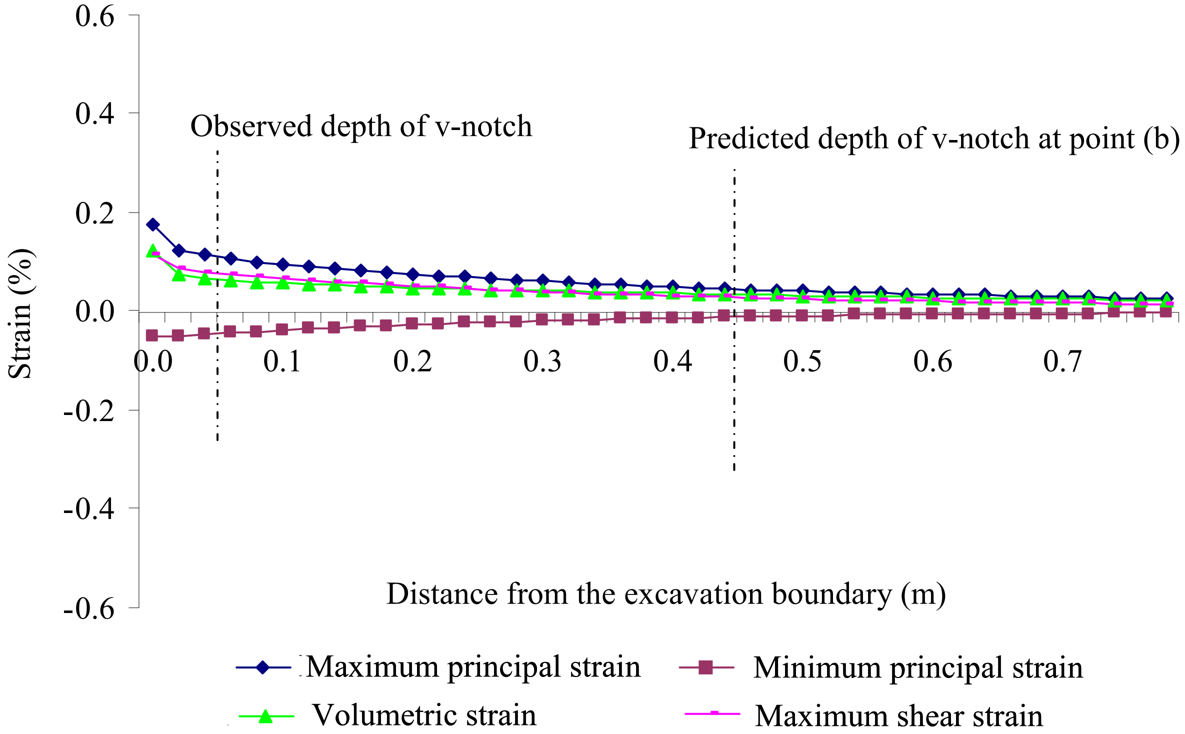


Figure 4.2 **Calculated principal strain along a line in the roof using a linear-elastic material model.**

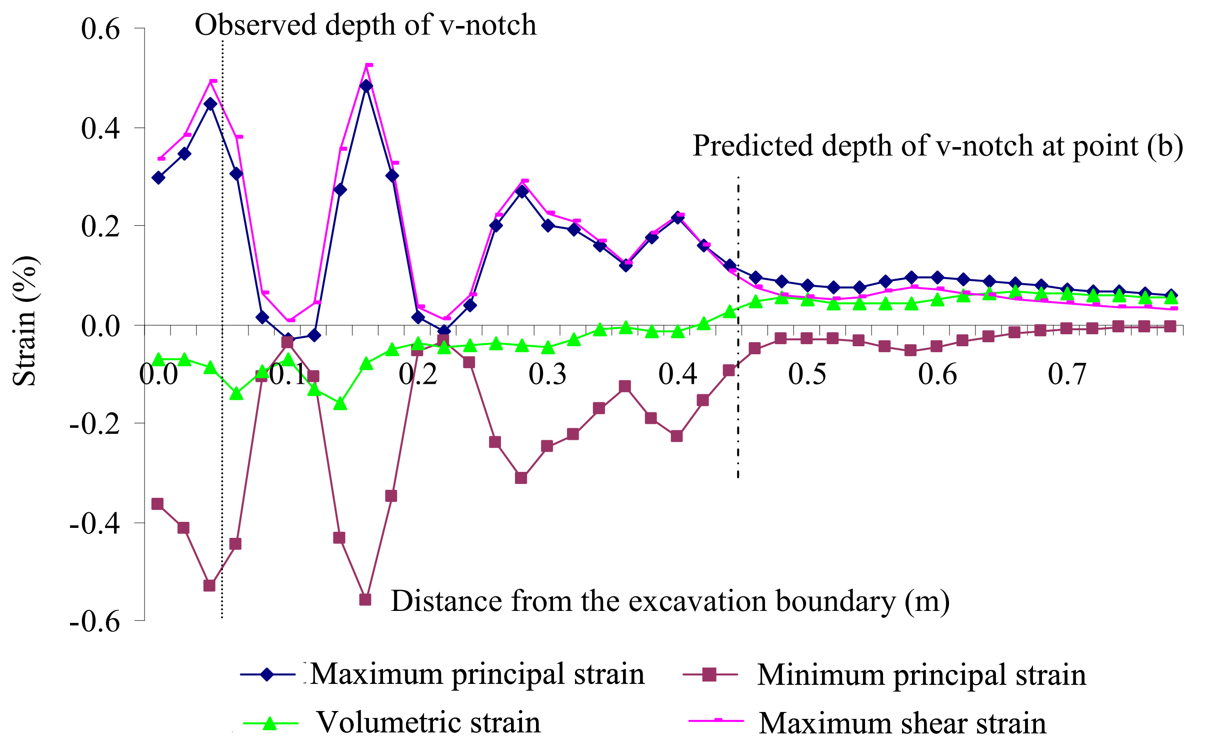


Figure 4.3 **Calculated principal strain along a vertical line in the roof using a linear-elastic brittle plastic material model.**

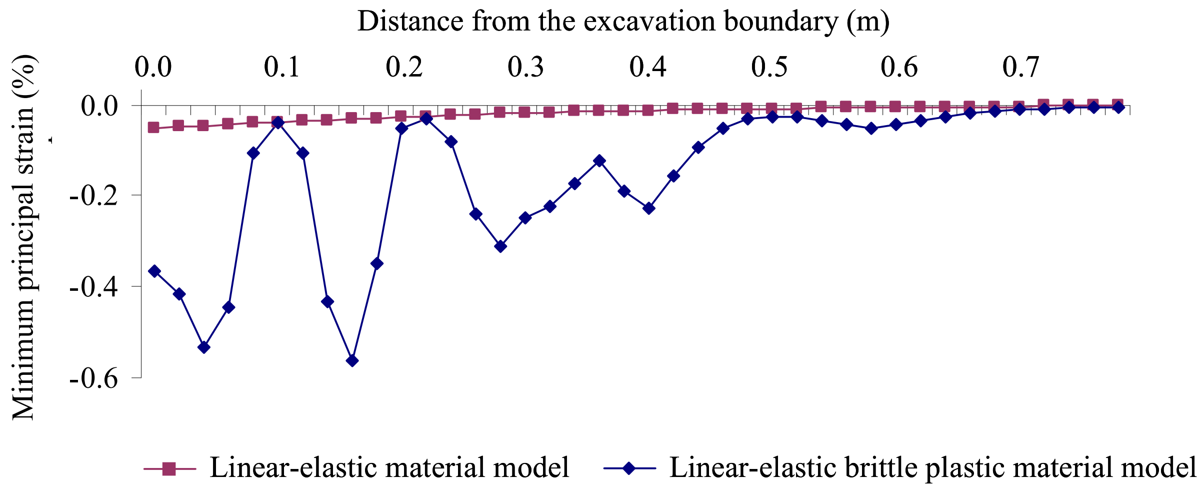


Figure 4.4 **Comparison between the minimum principal strain for the linear-elastic and the linear-elastic brittle plastic material model.**

4.2 The fictitious case

Contours of the maximum shear strain are shown in Figure 4.5. Shear bands forming v-notches are obvious when the major horizontal stress is oriented perpendicular to the excavation ($\sigma_{\perp} = \sigma_H$).

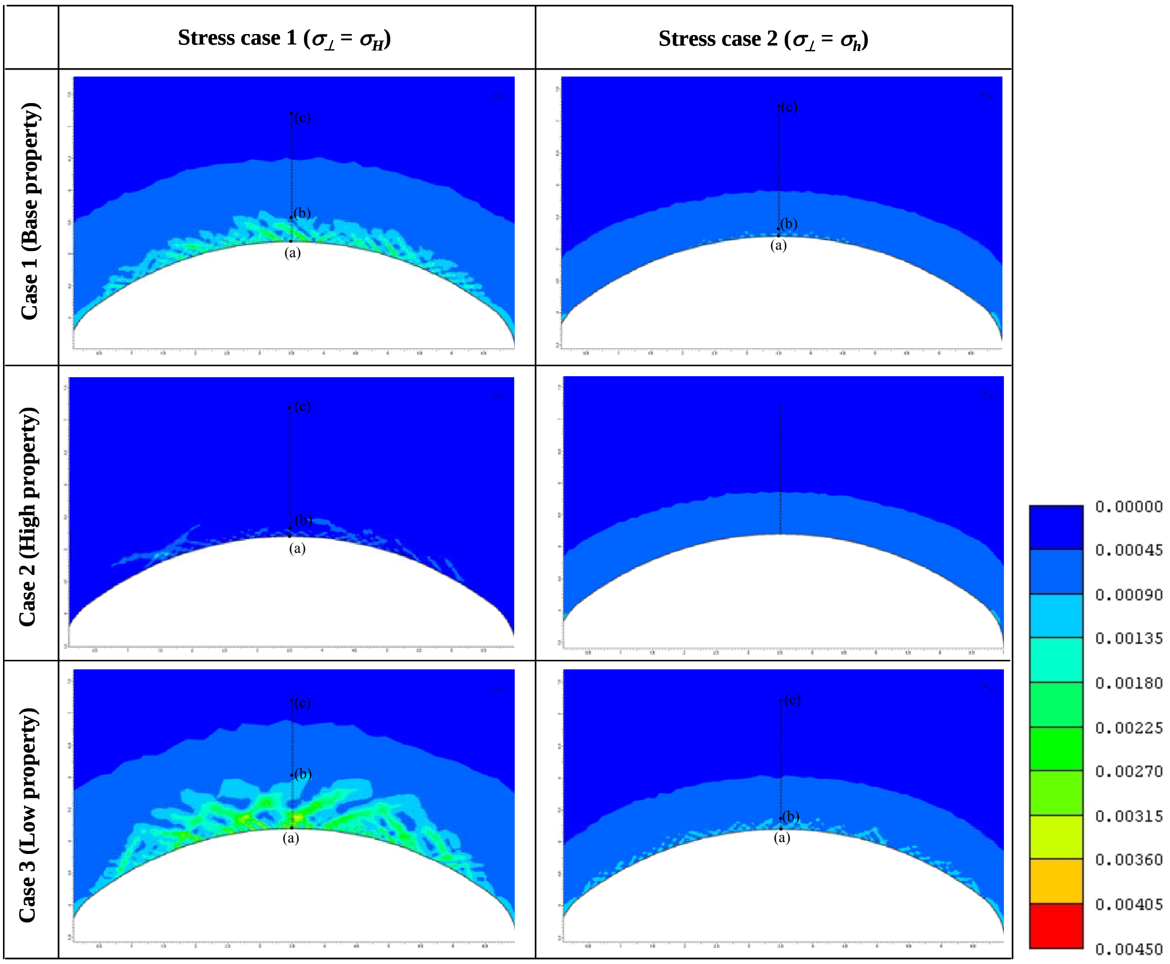


Figure 4.5 **Predicted maximum shear strain using a linear-elastic perfectly plastic material model in *Phase2*. The coordinates at point (a), (b) and (c) along the vertical line are given in Table 4.1.**

The principal strains (ε_1 and ε_3) were calculated. A vertical line was defined in the roof (as illustrated in Figure 4.5). Three points were defined along the vertical line as described for Garpenberg raise case. The values of volumetric strain and shear volumetric strain were obtained from *Phase2* (Rocscience, 2009) between these points.

Table 4.1 ***x, y*** **coordinates at point (a), (b) and (c) along the vertical line in the roof.**

Stress case	1 ($\sigma_\perp = \sigma_H$)	2 ($\sigma_\perp = \sigma_h$)
Property case	Coordinate (x, y) at point (b)	
1 (Base)	3.5 m, 5.614 m	3.5 m, 5.253 m
2 (High)	3.5 m, 5.306 m	-
3 (Low)	3.5 m, 6.068 m	3.5 m, 5.362 m

Coordinate at point (a): 3.500, 5.200; coordinate at point (c): 3.500, 7.200.

Linear-elastic material model
The principal strains at point (b) for each case is shown in Table 4.2. The principal strains are illustrated in Figure 4.7, and the volumetric and maximum shear strains in Figure 4.8. These quantities have large absolute values close to the excavation boundary, but decreases with the distance from the excavation. The maximum principal strain and the maximum shear strain

are similar at case $\sigma_{\perp} = \sigma_H$, as well as the maximum principal strain and the maximum shear strain at case $\sigma_{\perp} = \sigma_h$. The maximum shear strain is greater than the volumetric strain.

Table 4.2 Predicted strain due to v-notch in the roof using the linear-elastic material model.

Stress	1 ($\sigma_{\perp} = \sigma_H$)			2 ($\sigma_{\perp} = \sigma_h$)		
	*Depth (m)	Strain (%)		*Depth (m)	Strain (%)	
Property	at point (b)	ε_1	ε_3	at point (b)	ε_1	ε_3
1 (Base)	0.414 0.08		-0.06	0.053 0.07		-0.06
2 (High)	0.106	0.10	-0.08	-	-	-
3 (Low)	0.868 0.06		-0.04	0.162 0.06		-0.06

*Illustrated in Figure 4.6.

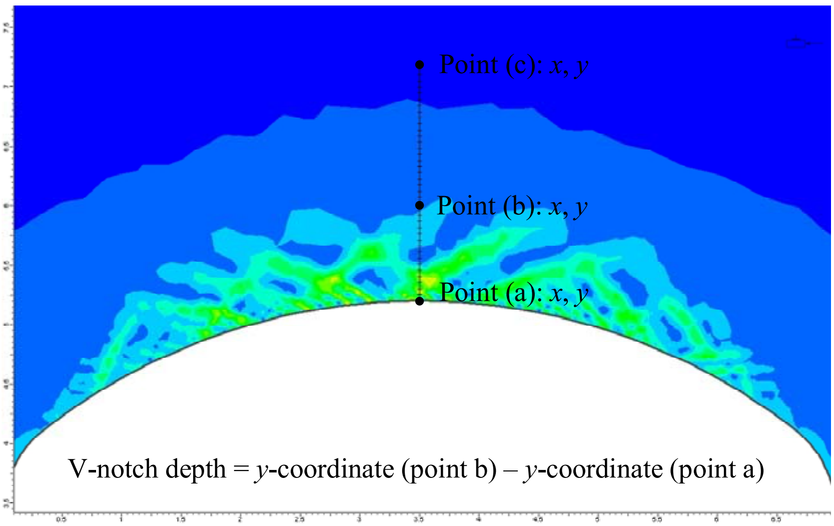


Figure 4.6 Illustration for determining depth of the v-notch.

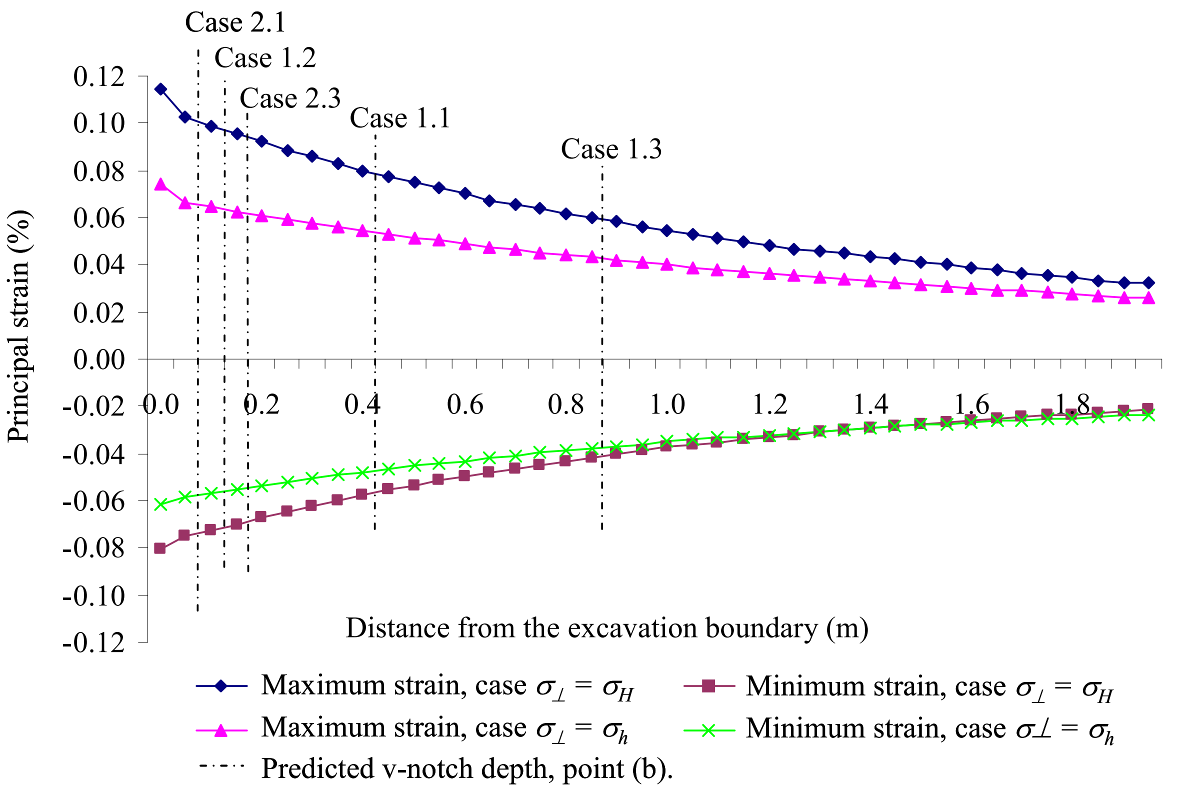


Figure 4.7 **Calculated principal strain along a vertical line in the roof using a linear-elastic material model.**

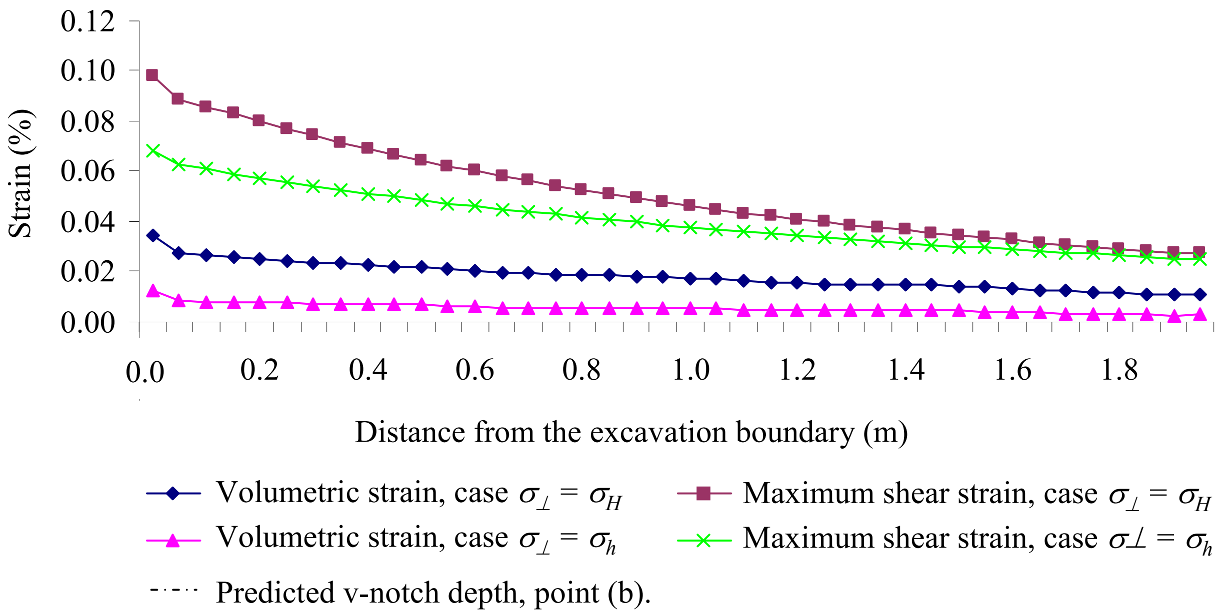


Figure 4.8 **Calculated volumetric and maximum shear strains along a vertical line in the roof using a linear-elastic material model.**

Linear-elastic perfectly plastic material model

The principal strains at point (b) for each case is presented in Table 4.3. The principal, volumetric and maximum shear strains are illustrated in Figure 4.9 and Figure 4.10 with respect to the distance from the excavation boundary. For all cases the absolute values of the principal strains and the maximum shear strain are large close to the excavation boundary, but decreases with the distance from the excavation. These plots show that the principal strains and the maximum shear strain are similar. The volumetric strain is negative from the boundary up to the end of the v-notch (point b), except for case $\sigma_{\perp} = \sigma_h$ with high property conditions. Moreover, for some cases, the peak of the principal strain curves coincides with the v-notch depth (point b).

As it was done for the Garpenberg raise case, the minimum principal strain was chosen to be plotted versus the distance from the excavation boundary in Figure 4.11. This plot shows that the largest minimum principal strains occur close to the excavation boundary when the stress perpendicular to the excavation is the major horizontal stress ($\sigma_{\perp} = \sigma_H$) and the properties of the rock are high. The base case was chosen to compare the linear elastic model with the elastic perfectly-plastic model in Figure 4.12. The comparison shows that close to the boundary, the minimum principal strain is largest for the perfectly plastic model when $\sigma_{\perp} = \sigma_H$.

Table 4.3 Predicted strain due to v-notch in the roof using the linear-elastic perfectly plastic material model.

Stress	1 ($\sigma_{\perp} = \sigma_H$)			2 ($\sigma_{\perp} = \sigma_h$)		
Property	Depth (m) at point (b)	Strain (%) ε_1	Strain (%) ε_3	Depth (m) at point (b)	Strain (%) ε_1	Strain (%) ε_3
1 (Base)	0.414	0.09	-0.08	0.053	0.08	-0.09
2 (High)	0.106	0.11	-0.15	-	-	-
3 (Low)	0.868	0.08	-0.06	0.162	0.06	-0.07

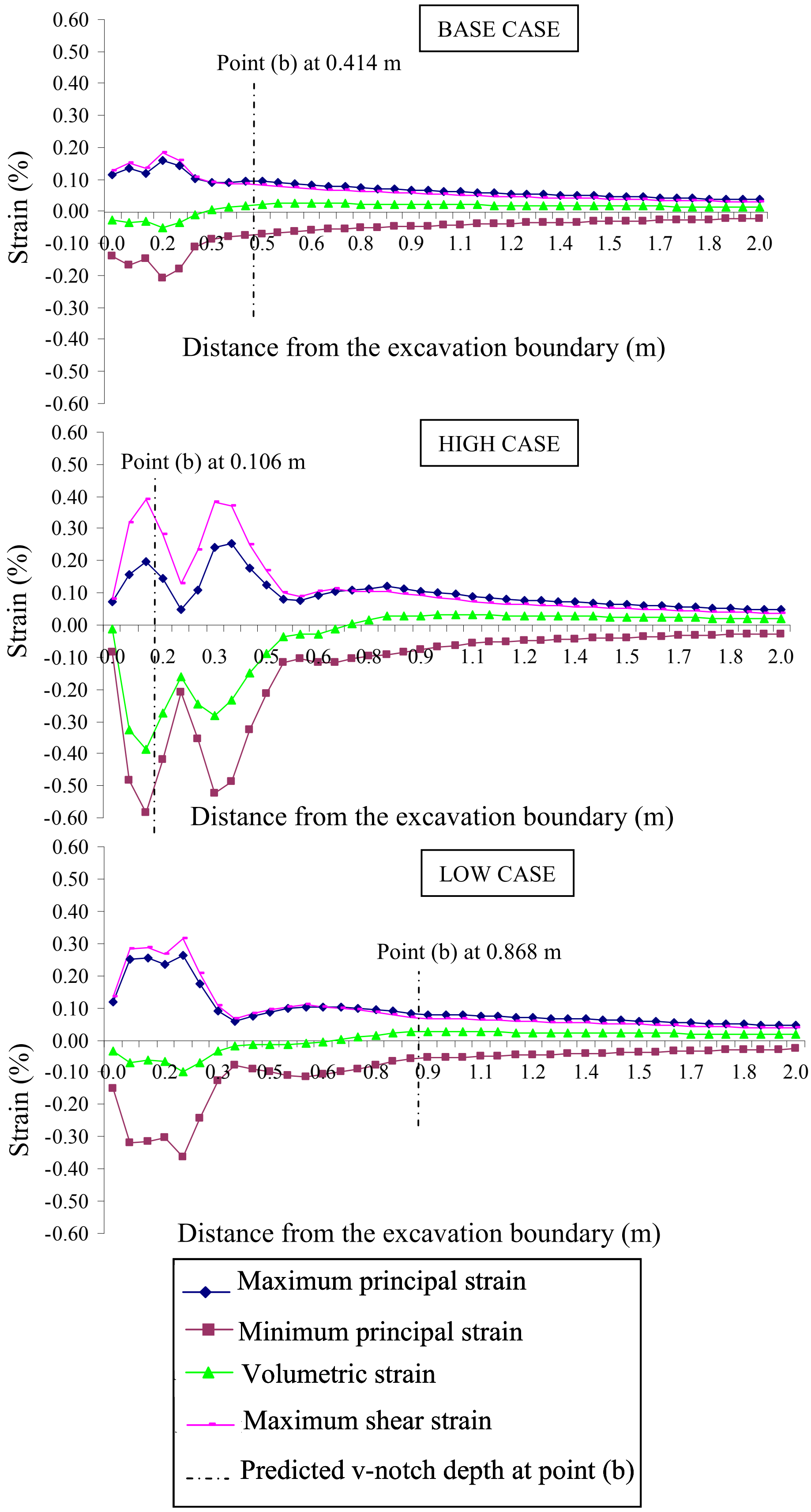


Figure 4.9 **Calculated principal, volumetric and maximum shear strains along a vertical line in the roof (case $\sigma_{\perp} = \sigma_H$, base, high and low property cases) using a linear-elastic perfectly plastic material model.**

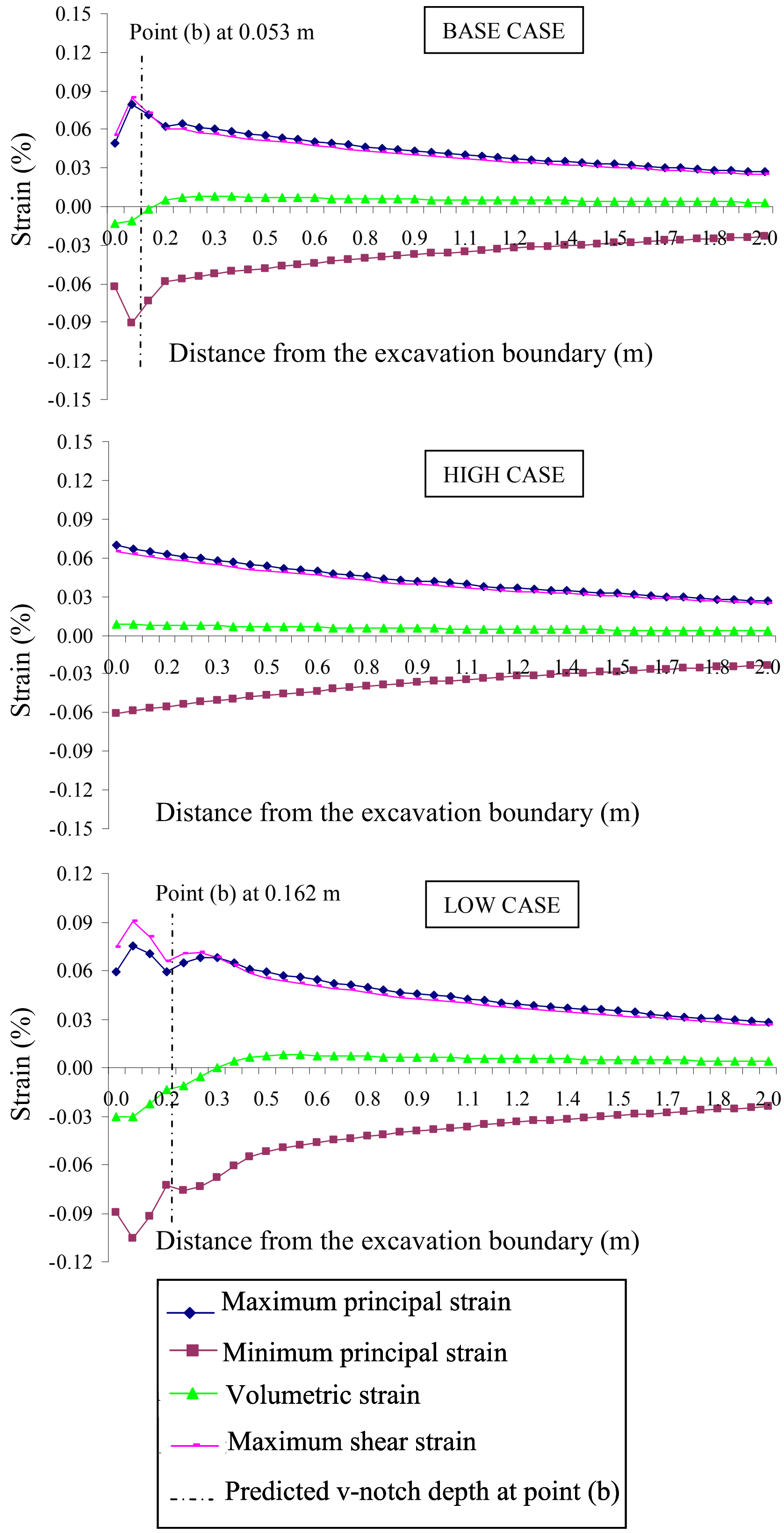


Figure 4.10 **Calculated principal, volumetric and maximum shear strains along a vertical line in the roof (case $\sigma_{\perp} = \sigma_h$ at base, high and low property cases) using a linear-elastic perfectly plastic material model.**

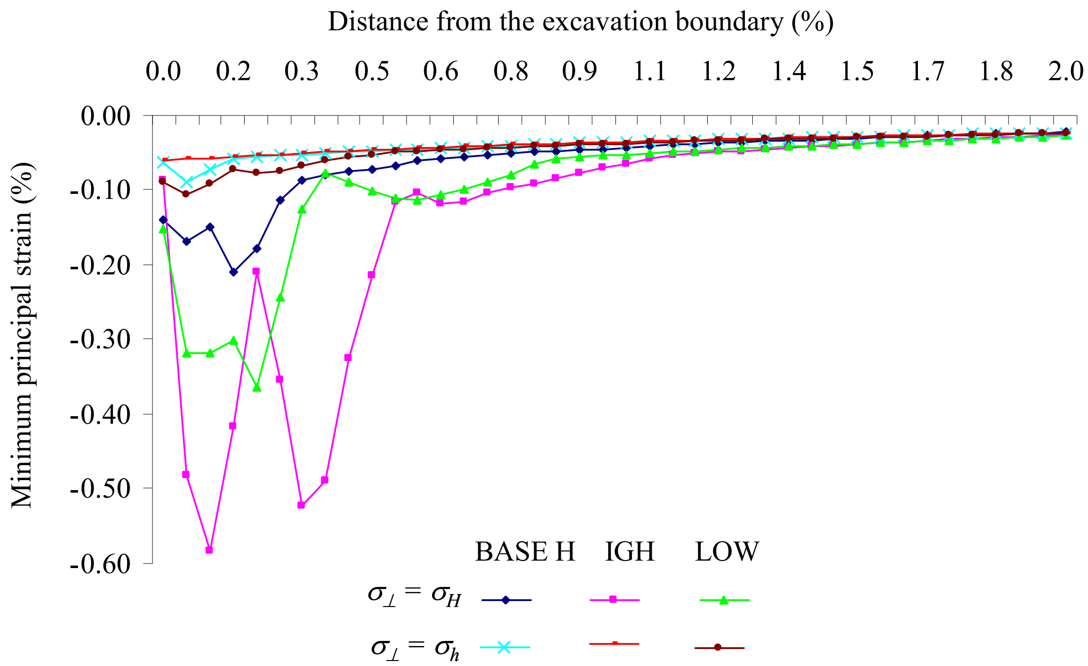


Figure 4.11 **Minimum principal strain for six fictitious cases using a linear-elastic perfectly plastic material model.**

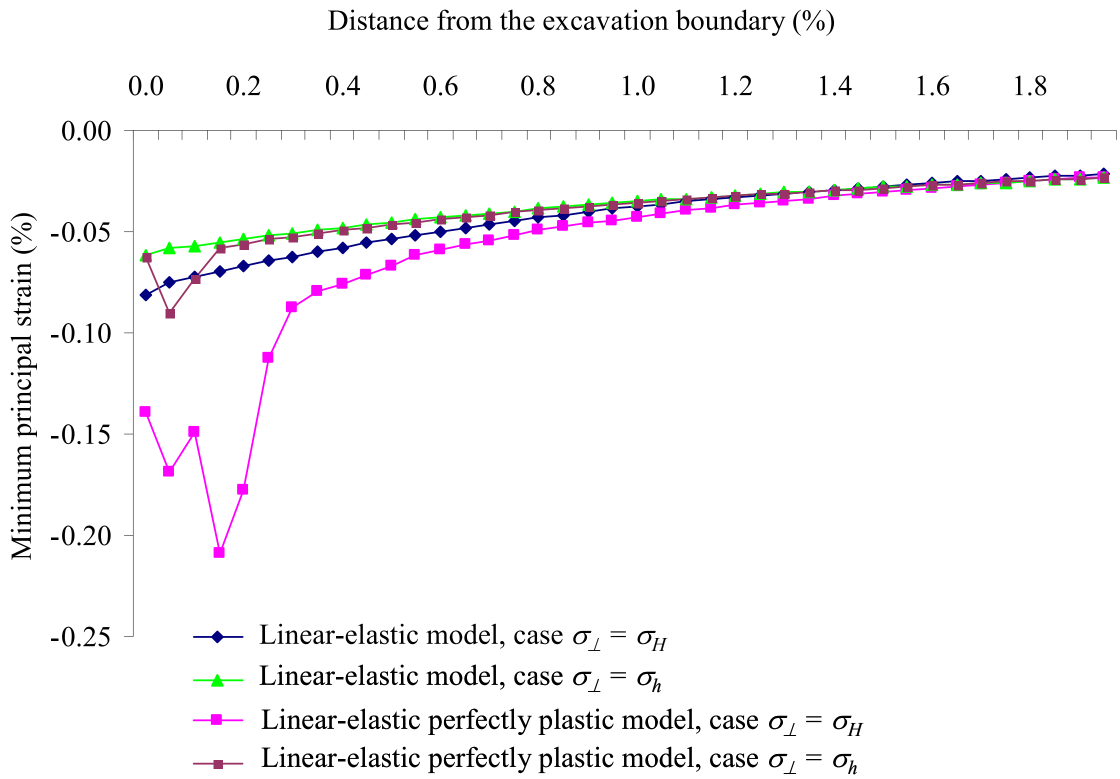


Figure 4.12 **Comparison between the minimum principal strain for the linear-elastic and the linear-elastic perfectly plastic model, case $\sigma_{\perp} = \sigma_H$ and $\sigma_{\perp} = \sigma_h$ for the base case property.**

5 DISCUSSION AND CONCLUSIONS

This Chapter presents a discussion of the literature, cases and laboratory tests review, and numerical modelling. The discussion is based on how the collected data from the literature, cases and laboratory test as well as the evaluation of laboratory tests and numerical modelling have achieved the objectives of this licentiate research. Moreover, the quality of the collected data is assessed with respect to whether or not it is judged sufficient as input data for future works.

5.1 Discussion

Most of the laboratory studies, presented in the literature, have been focused on the pre-peak behaviour. This would indicate that there is a need of laboratory studies of the post-peak behaviour, since stability problems and fallouts involve pre- as well as post-peak deformations. However, in the pre-peak laboratory studies, the crack damage strain (See for example, Martin and Chandler, 1994) can be determined. Since it is defined as the crack volumetric strain at the onset of unstable crack growth and dilation, it may be a key quantity to define the onset of localized failure (creation of macro cracks and shear bands).

The literature review also revealed that there is a lack of failure criteria based on deformation quantities. Furthermore, the failure criteria presented in the literature are only formulated as limits between elastic and non-elastic behaviour. None of the presented criteria were formulated in terms of onset of fallout or macroscopic collapse. Thus, there is a need for fallout criteria which could help in improving the usefulness of the results from numerical analyses.

The strain criterion for fracture initiation presented by Stacey (1981) requires determination of the critical value of the extension strain with great sensitivity. Moreover, the levels of extension strain in a uniaxial compressive test correspond to a stress level of 30% of the uniaxial compressive strength. However, these stress levels are lower than those at crack closure for rocks tested by LTU. The average value of the axial crack closure stage for rock types tested by LTU was of 45% of the uniaxial compressive strength. This observation raises some questions regarding the definition of critical extension strain according to Stacey (1981). It may also suggest that this criterion is not applicable to predict initiation of failure for all rock types. The advantage with the shear strain criterion used by Sakurai (1995) is that the critical shear strain can be determined from laboratory test data. However, some questions may be raised regarding this criterion: (i) what type of stability problem is evaluated?, (ii) what type of laboratory tests needs to be performed to determine the critical strain?, and (iii) no application of the criterion has been presented. The advantage with the strain-strength criterion proposed by Chang (2006) is that quantities such as volumetric strain and maximum principal strain can be easily determined from laboratory tests. However, it is not clear how the constants κ and ε_c were determined and under which conditions they apply. Additionally, the plasticity term is very subjective. It could be interpreted as (i) failure and fallout of rock, or (ii) formation of fractures without involving failure.

Two of these failure criteria (found in the literature review), formulated in terms of strain quantities are all based on strength data from laboratory tests. The quality of the input in terms of measuring accuracy is therefore often high. However, the use of laboratory strength data as input for prediction of failure around underground openings may be questionable, due to differences in failure mechanisms and rock volume involved in the failure. Small scale spalling may be well predicted using laboratory data, whereas it is unlikely that a core-based sample would behave similar to a large volume of a highly fractured rock mass during the failure process.

Monitoring of deformations around an underground opening during the excavation phase as well as during the operation can give valuable information regarding the performance and stability. However, the choice of monitoring instrument and the location of installation can be crucial for the quality of the information from the measurements. In order to ensure that the monitoring programme can identify localisation of failure (shear bands, etc) and the magnitude of localized deformations, it has to be based on a comprehensive study including, for example, numerical analyses. On the other hand, if the excavation is in a highly fractured and weathered rock mass with high stress magnitudes, the monitoring of deformations will always result in useful information, regardless of a detailed understanding of the failure process. Deformation monitoring under such conditions will give information about potential large scale fallouts and total collapse.

The comparison between cases and displacement magnitudes among cases showed that they deform very differently. The difference is due to the effect of different factors such as rock type, geological structures, rock properties, state of stresses, failure type and deformation measurement technique. In the Arlandabanan case the displacement was directly related to the response of the excavation process and/or the blasting technique since fallouts did not occur. For the Heathrow airport tunnel collapse the displacement reflect the collapse in soft ground. The Mine-by Experiment and the Äspö Pillar Stability Experiment are good cases if spalling failure is the objective. The Kiirunavaara drift case is another good case for evaluating block falling. The cases presented in this thesis can be used to support the development of numerical modelling techniques, since they provide input data, deformation measurements as well as descriptions of the failure and stability problems encountered.

Critical axial and lateral strain at crack closure, crack initiation, crack damage and peak strength stage were identified for each rock type tested in the laboratory studies by LTU and Posiva Oy. Since the laboratory tests were performed up to the peak, the plastic behaviour, i.e., post-peak stage, was not studied. The evaluation of laboratory test data provided a valuable database of critical strain values for a number of different rock types. The numerical modelling of the real case (Garpenberg raise) and the fictitious case were evaluated using critical strain values obtained in the evaluation of the laboratory tests. The maximum and minimum principal strains, and maximum shear strain calculated in the numerical analyses showed a large variation within the v-notch but decreased monotonically behind it. Furthermore, the calculated minimum principal strains at the v-notch were compared with critical strains from the laboratory tests. The strain data from the laboratory tests were chosen to match the rock types in the numerical models (information from the sites). The comparison showed that the critical lateral strains seem to be good indicators of crack development and onset of dilation.

5.2 Conclusions

The following conclusions can be drawn from this work:

- The evaluation of the underground cases showed that the rock mass deform differently based on factors such as rock properties, stress state and deformation measurement technique among others.

- The laboratory test data evaluation showed that the stages of deformation vary between rock types and depend on rock features such as grain size and mineral composition. The rock types should therefore be treated individually.
- The evaluation of strains from numerical analysis showed that the maximum and minimum principal strains, and the maximum shear strain indicate the position of localized failure (macroscopic failure surfaces such as shear bands).
- The absolute values of maximum and minimum principal strains, and maximum shear strain are larger close to the excavation boundary. The absolute values of the strains decrease with the distance from the excavation boundary.
- The critical strains measured in the laboratory tests seem to give information which can help to improve the understanding and the description of the failure process around underground openings such as the Garpenberg raise and the fictitious tunnel.
- The minimum principal strain was a good indicator that can be used to identify different stages of the failure process due to crack development.

5.3 Recommendations for further research

This licentiate research has shown some issues, which needs to be further studied in order to improve the understanding of deformation and the different stages of the failure process:

- Develop criteria for prediction of failure based on deformation quantities. Caracteristic deformation quantities for the failure process in hard rock have to be identified. The data of the laboratory tests presented in this work can be used as input data.
- The real cases presented in this thesis should be used to calibrate numerical models in order to support the development of numerical modelling techniques and study the sensitivity of uncertainty in the input parameters.
- It would be of great value to follow an ongoing excavation process where monitoring of deformations and damage mapping is carried out. Such a case could then be used for failure and fallout prediction using different failure criteria and taking into consideration uncertainty in the input data.
- Carry out numerical modelling and simulate failure in order to propose a failure criterion in terms of deformation quantities that predicts fallout such as spalling and shear failure.

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ROCK ENGINEERING RESEARCH FOUNDATION
STIFTELSEN BERGTEKNISK FORSKNING

Model Uncertainty of Design Tools to Analyze Block Stability

Mehdi Bagheri, Royal Institute of Technology, KTH

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Model Uncertainty of Design Tools to Analyze Block Stability

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SUMMARY

Block failure is one of the most common failure modes in tunnels. Design tools to analyze block stability have some simplifications and, therefore, they also have some model uncertainties. The purpose of this project is to assess the model uncertainty for different design tools (KLE, analytical solution) in order to estimate block stability.

Different approaches of kinematic limit equilibrium (KLE) including conventional KLE, limited joint length, limited joint length and stress field consideration and probabilistic KLE were compared to that of DFN-DEM. In this approach, the results of the calibrated DFN-DEM with field mapping were considered to be of true value. The results show that the conventional KLE is overdesign due to it's over simplification. By considering fracture length and stress field, the volume of predicted unstable blocks is reduced. The probabilistic approach of KLE by considering finite joint length and stress field predicts the volume of unstable blocks to be lower than DFN-DEM approach. Therefore there is a great model uncertainty of our standard design tools for block stability analysis.

The results from analytical solution based on joint relaxation process have also been compared to those of DEM at different condition of depth, K0, apical and friction angle, Kn and Ks value, and ratio of Kn/Ks. The comparison shows that for shallow depth with K0 less than 1, analytical solution leads to an overestimation of block stability. The analytical solution predicts that the block is stable, while the analyses from numerical solution show the block is unstable. The analyses show that by increasing K0, accuracy of analytical solution also increases. Moreover, for the cases with close value of friction angle to semi-apical angle, the use of analytical solution is not recommended. As the ratio of Kn/Ks increases, the accuracy of analytical solution decreases. Increasing the angle ratio (ratio between semi-apical angle to friction angle) is one source of increasing uncertainty in the model. The analytical solution is very uncertain in cases with a low value of K0, and a high value of stiffness ratio and angle ratio. On the other hand, the analytical solution is more certain in conditions with a high value of K0 and a low value of stiffness ratio and angle ratio. According to current information (K0, angle ratio, stiffness ratio), one can determine the value of model uncertainty by using the diagrams presented in Chapter 6 of the thesis (Bagheri, 2009). The analyses show that by having more information about the key parameters, the model uncertainty could be identified more precisely. However, having more information means spending more money, and this increase in cost must be compared to the cost of failure or delay in the project or overdesign.

List of Publications

This thesis incorporates 3 papers on the subject of model uncertainty design tools to estimate block stability:

- G. Nord, M. Bagheri, A. Baghbanan and H. Stille ’Design consideration of large caverns by using advanced drilling equipments’, *Felsbau* Vol 25(5) PP131-136, 2007
- M. Bagheri, H. Stille, Investigation of model uncertainty for block stability analysis, Accepted by International Journal for Numerical and Analytical Methods in Geomechanics, Doi: 10.1002/nag.926
- M. Bagheri, A. Baghbanan, H. Stille, ‘Some aspects on model uncertainty in the calculation of block stability using Kinematic Limit Equilibrium’, *ARMAROCKS 08*, US, 2008
- M. Bagheri, H. Stille, ‘Some aspects of model uncertainties of block stability estimation’, *ARMS2008*, Tehran, PP 675-681, 2008

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1-Introduction

1-1 Background

Several failure modes may occur around underground openings. One of the most common observed failure modes in underground openings is block failure. Fractures cross each other in the perimeter of excavation and they make blocks with different sizes, which may have the potential to fail. The excavation alters the magnitude and direction of stress, and this creates changes in the forces that act on the located blocks in the perimeter of excavation. The potential unstable blocks could slide, fall out from the roof or rotate (Mauldon and Goodman, 1990). Stability of blocks depends on block shape, size, and stresses around the opening. Block shape and size depends on the fracture pattern. The stresses around the opening depend on the shape of the opening and in-situ stresses. In order to assess the stability of the opening, potential unstable blocks must be recognized and stresses around the opening analyzed. In the case of instability, required rock support must be estimated. The block stability includes the interactions between blocks, block geometry, forces, and support. Analyzing this type of failure mode is a complex problem.

The purpose of design of an underground opening is to predict the stability with a certain amount of confidence. The reliability of the predictions is influenced by the uncertainties involved. Three different kinds of uncertainties are normally geometric uncertainty, parameters uncertainties, and uncertainties in the design tools. Model uncertainty plays an important role in the reliability analysis and the design of rock support. One example of the influence of model uncertainty on the design could be seen in the design based on ultimate limit state. The design based on the ultimate limit state requires a definition of a performance function. Performance function is usually based on a standard deterministic design tool. Model uncertainty is associated with imperfect representation of reality and simplifications in the design tool. The designer needs to know how to properly represent model uncertainty in a limit state design. According to the Eurocode (Eurocode, 1997), there is no recommendation for the design of openings against block failure based on reliability analysis or observational method. Based on the author's knowledge, no publication exists on model uncertainty for block failure in underground openings. For this reason, the model uncertainty for the different block failure design tools is evaluated in this report.

1-2 Objectives and limitations

The objective of this report is to identify the advantages and disadvantages of the different design tools used to analyze block stability, as well as to assess the model uncertainty of the different design tools. Available design tools used to analyze block failure could be divided into design tools to estimate block volume (kinematic analysis and Discrete Fracture Network) and design tools to analyze the equilibrium of the block (analytical solutions and numerical solutions).

Limitations

Model uncertainty can only be quantified either by comparison with other more involved models that exhibit a closer representation of the nature or by comparison with collected data from the field or the laboratory (Ditievsen, 1982). The author has not found any recorded case in which failed block geometry, volume, resistance parameters, stresses were measured. Therefore, the results of different design tools have been compared to those, which are more closely representative of nature.

The model uncertainty is estimated for static design tools. Effects of dynamic loading on the block stability are not considered. However, this is out of the scope of this report.

1-3 Design Tools to Analyze Block Stability

In order to analyze block stability, two questions must be answered. ***Do we have any block?*** And, ***if there is one, is it stable or not?*** The first question relates to the block existence and block volume. Block volume and its existence are related to the fractures and opening orientation, and fracture length. Priest (1993) mentioned that the kinetics feasibility for a given block can determine the potential of movement, and this is not based upon forces analysis. The second question is related to forces that act on the block. Forces acting on the block are block weight, induced stresses, dynamic loads, resistance forces from fracture friction and forces from support. To answer the first question, design tools such as kinematic analysis and DFN are available. To answer the second question, analytical solutions based on limit equilibrium and joint relaxation and numerical solutions are available. Each design tool has certain assumptions on the rock mass behaviour and some simplifications on the block geometries and presence of fractures in rock mass; it is important to understand how to use these tools efficiently and both the strengths and weaknesses of the tools (Starfield and Cundall, 1988). Table1-1 shows various combinations of different design tools to answer the question regarding block existence and analyzing forces that act upon the block.

Table 1-1. Different Design Tools to Analyze the Geometry and Stability of Blocks

Block Volume	***Kinematic Analysis***		***DFN***
	Conventional	***Finite Joint Length***	

Estimation			***Analysis***	***Deterministic***	***Probabilistic***	
Analytical Solution	***Based on Limit Equilibrium 1***	No Stress Field	A1	B1	C1	D1
		Considering Stress Field	------------------	B'1	C'1	D'1
	Based on Joint Relaxation 2		A2	B2	C2	D2
DEM 3			A3	B3	C3	D3
DDA 4			A4	B4	C4	D4

In the table, alphabet (A-D) refers to the design tools used to estimate the block existence and its volume estimation. Numbers 1-4 refer to the design tools used to analyze forces around a block. In Chapter 2, a short description will be given of design tools to analyze block volume. In the Chapter 3, a short review will be given of analysis methods 1-4. Model Uncertainty of Kinematic Limit Equilibrium Analysis **(**A1, B1, C1, B'1 and C'1) is discussed on chapter 4. This has been done by comparing to the results of D3. Analytical solution based on joint relaxation (2 in the table) will be compared to DEM (3 in the table) in Chapter 5.

1-4 Uncertainties

Uncertainty deals with safety and economics of a project. This is therefore a very important issue in the design process. There are different uncertainties involved in block failure analysis such as block geometric uncertainty, model uncertainty, and parameter uncertainty. The other failure modes also deal with mechanical parameter uncertainty and model uncertainty. Geometric uncertainty makes block failure different from other types of failure modes. All aspects of uncertainties affect the results of analyses. Considering model uncertainty which is an issue that plays an important role in the design and decision-making about rock support, ignoring the model uncertainty could be very dangerous. The designer should be aware of the model uncertainty, and should correct the outcome of model regarding to the model uncertainty factor. The general aspects on the model uncertainty have been explained in this chapter. The application will be discussed in Chapter 4 and 5.

2-Design tools to estimate block volume

Different design tools have been described to estimate block existence and volume in licentiate thesis. The advantage and disadvantage of each are also described. Figure 2-1 shows a schematic view of the predicted block volume by different approaches. By infinite fracture length, a large volume of potential unstable blocks is predicted, while by limiting the fracture length to observed maximum fracture length, the volume of potential unstable blocks is reduced. The probabilistic approaches of kinematic analysis (finite joint length and orientation) and DFN are based on stochastic nature of fractures in mass, and will result in a distribution for potential unstable blocks.

The main difference between DFN and kinematic analysis is that the kinematic analysis takes into account blocks that are formed by the conjunction of three joint sets, while in DFN, blocks can be formed by the conjunction of more than three joint sets. In another way, it could be said that, in kinematic analysis, blocks are assumed to have a tetrahedral shape while other polyhedral shape of blocks are possible in DFN approach. Another difference is that kinematic analysis has the purpose of finding the maximum block while DFN does not have this aim.

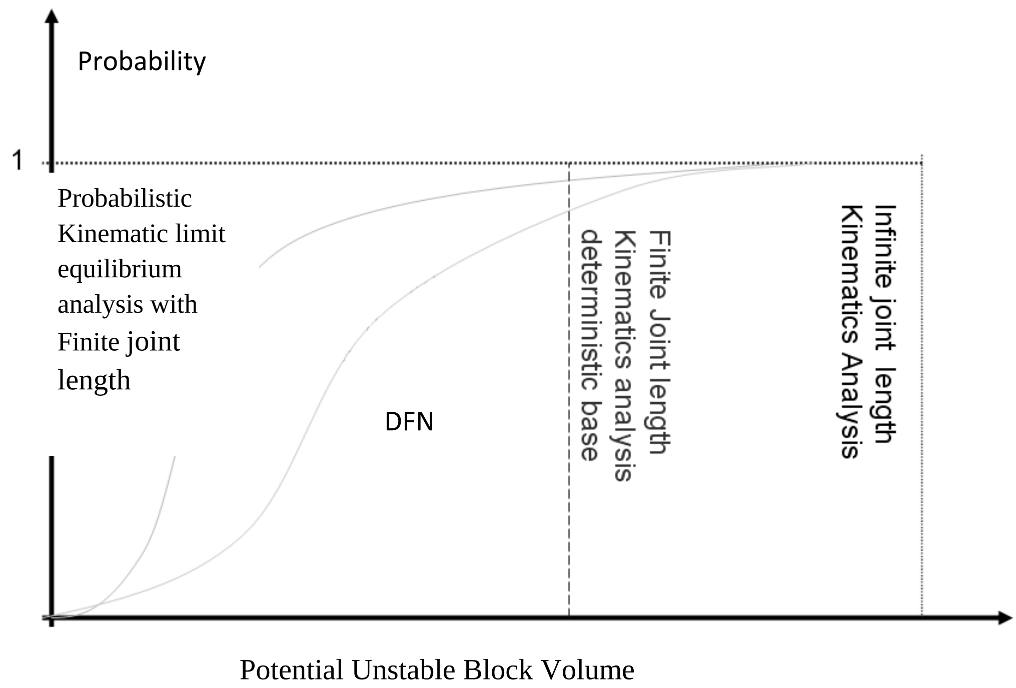

Figure 2-1. Schematic view of different design tool to predict potential unstable block volume

One of the most significant uncertainties in block stability analysis is the block volume estimation. This comes from the fact that the true value for block volume could not be directly measured. As is shown in Figure 2-1, the design tools could be compared to each other. Each of the methods has some assumptions that make that the model predictions differ from reality. However, among them, the calibrated DFN may predict block volume closer to reality.

3-Design tools to analyze block stability

Although the use of kinematic-limit equilibrium or key block theory- limit equilibrium (Goodman and Shi, 198) are quite simple, a system consisting of an assemblage of blocks cannot be studied. Discrete element methods (Cundall, 1971) could consider the system assembly of blocks. On the other hand, it is impossible to have exact joint locations and geometries in practice. Therefore, the use of numerical method is used more to understand the failure mechanism and effect of in-data changes on the results of analysis (Barbour and Krahn, 2004). The analytical solution based on Limit equilibrium mechanics without consideration of clamping forces is conservative. The analytical solution which takes into account the fracture stiffness and joint relaxation may lead to a better estimation of failure mechanism and a better prediction of required rock support.

Crawford and Bray 1983, proposes the solution of a more detailed analysis that considers the effect of fracture stiffness. For simple cases that follow the plain strain, the failure is sliding or falling or in the cases such as the persistence fractures, the use of analytical solution could be useful if their model uncertainty has become quantified.

There are many parameters that are required to perform the DDA analysis. It is not clear how to obtain these parameters in practice (Ohnishi, et al, 2006). Still, DDA is underdeveloped and the use of DEM is recommended instead. The progressive failure in combination with fracturing propagation is a phenomenon that cannot solve by the current technology of DEM.

DEM incorporates a careful stress analysis in order to analyze all block failure modes (rotation, falling or sliding). Therefore, DEM analysis is the most accurate analysis by today’s knowledge that can be performed by analyzing the block stability for non-progressive failure.

4-Model Uncertainty of Kinematic Limit Equilibrium Analysis

Results of DFN-DEM analysis, which have been confirmed by a William-Watson test (Batschelet, 1981) were considered to be real. The conventional KLE which doesn't consider the fracture length and field stresses is on safe side and leads to a conservative design. By considering the fracture length in KLE, the estimated unstable block volume is reduced. But still this approach is on safe side and it is overdesign. Considering of stress field together with fracture length will reduce the unstable block volume further. But still this approach is on safe side and it is overdesign. It can also be concluded that a kinematic analysis based on a Monte Carlo simulation estimates block volume smaller than reality. The results of probabilistic approach analyses (both PKLE and DFN-DEM) could be shown in a distribution for the potential unstable block volume. This will show the designer the probability for forming block with a specific volume. The designer could decide about the acceptable unstable block volume related to its probability.

The results show that even considering limited joint length in kinematic analysis and the clamping forces in the limit equilibrium analysis, there is a great model uncertainty of our standard design tools for block stability analysis.

The analyses show that the results of probabilistic kinematic analysis are interesting and commercial software ought to develop to facilitate the calculation.

5-Model Uncertainty of Bray-Crawford Solution

Model uncertainty of analytical solution based on joint relaxation has been assessed. The analyses show that Bray-Crawford solution has good accuracy for the tunnels with negligible vertical in-situ stress and high value of K0.

The DEM considers the relaxation of in-situ stress, while the analytical solution does not. The relaxation of in-situ stress gives the joint normal displacement which makes reduction of clamping force. This is not considered in analytical solution; therefore, the analytical solution overestimates the block stability.

With decreasing of K0, the mean value of model uncertainty factor decreases. This corresponds to that the outcome of the analytical solution is more biased. The standard deviation of model uncertainty factor increases with decreasing of K0. Neglecting key parameters such as vertical stress, joint shear and normal stiffness together with relaxation of in-situ stress generates model uncertainty. Thus the analyses show that the vertical stress plays important role in estimation of block stability in crown of openings.

Three important parameters to identify model uncertainty have been recognized. They are K0, ratio between joint normal and shear stiffness, and ratio between block semi-apical angle and friction angle. As the amount of information about the in-situ stress state, joint stiffness, apical and friction angle increases, the variation of model uncertainty factor decreases and the model uncertainty factor could be determined more precisely. Information about the all identified key parameters is required in order to assess acceptable precision.

The results of the analyses indicate that, by increasing the ratio between joint, normal stiffness, and shear stiffness, or the ratio between semi-apical angle and joint friction angle the outcome of model is more biased. Cases with higher value of vertical in-situ stress than horizontal stresses - especially for the shallow depth tunnels or the cases in which the friction angle is closed to semi-apical angle, the analytical solution overestimates the block stability. By having biased factor in an acceptable precision, the outcome of analytical model could be modified. The analytical solution could be used in combination with the tables for determining model uncertainty factor.

6-Remarks and Conclusions

The purpose of this research has been to quantify the model uncertainties of different design tools in order to calculate block stability. The author has described different design tools to estimate block volume such as kinematic analysis and DFN, and also design tools to estimate block stability such as analytical solutions and DEM.

Different approaches of Kinematic limit equilibrium with various assumptions in the joint length, stresses, and joint orientation have been applied to a cavern. These results have been compared to those of DFN-DEM, which show that the conventional KLE (unlimited joint length and without field stress) overestimate the unstable block volume. However, while by applying the joint length, the unstable block volume is reduced. By considering the joint length and field stresses around the largest unstable block, its volume is reduced. Monte Carlo could be used to define a representative value for joint length and the orientation which could be used in a Kinematics limit equilibrium which considers the clamping forces from in-situ stress. The comparison between this approach and DFN-DEM shows that this approach predicts the unstable block volume lower than DFN-DEM.

Another conclusion of KLE analysis is that the information about joint length and stresses could lead to a better design. Once again, the costs for obtaining the information about the joint length and stresses must be compared with the costs for overdesign. As an example that relates to the case study in conventional KLE analysis, the support must be design for a 5779 m^3 of block per 1 meter of tunnel length. While considering the joint length and stress field, it is reduced to 22 m^3 per tunnel length.

The analytical solution based on joint relaxation could be used together with kinematic analysis in order to estimate the stability of block. Model uncertainty of the analytical solution has been assessed. The analyses show that Bray solution has good accuracy for the tunnels with negligible vertical in-situ stresses and high value of K0.

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Further Research

Although block failure is a common failure mode in underground openings, there is still a need for more research on the probabilistic design against block failure. Further research could perform to analyze the effects of key parameters such as K0, angle ratio, and stiffness ratio on the reliability index.

The analyses show that there is a systematic error in Bray-Crawford solution. The solution needs to be improved in order to consider the effects of in-situ stress relaxation. The analytical solution based on joint relaxation could be revised in order to consider the joint stiffness changes due to changes of loading.

Moreover, current available commercial software cannot perform the probabilistic approach. More work is needed to develop the software that could be helpful in research and practice.

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Box 5501
SE-114 85 Stockholm

info@befoonline.org • www.befoonline.org
Besöksadress: Storgatan 19

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