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Review of the Use of Engineering Geological Information and Design Methods in Underground Rock Construction

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SUMMARY

One of the main concerns in underground rock construction is the complexity and uncertainty of the ground conditions at the project site. Detailed knowledge of the actual ground conditions cannot be determined until they are revealed at excavation, and the risk of preparing insufficient design alternatives must be balanced against the cost of investigating the ground before construction. The handling of engineering geological information and the preparation of design alternatives of various construction measures consequently need careful consideration throughout the construction process.

The ground conditions are generally established by collecting and characterising data, creating geological models of the site and grouping material with similar engineering characteristics. Various design methods are employed to solve the engineering issues, e.g. numerical modelling, use of experiences from similar cases and the evaluation of observations carried out during construction. There are, however, problems associated with the handling of engineering geological information and implementation of design methods which are not always considered in the construction process. This study aims at identifying some of the most common problems associated with information handling in rock engineering design and explore some viewpoints on how these problems can be mitigated. The following areas are reviewed within this study:

- The framework of the underground construction process
- The investigation, conceptualisation and characterisation of ground conditions as a part of the handling of engineering geological information
- The concept of uncertainties and risks within rock engineering and how these are dealt with during the construction process
- Design methods commonly employed in rock engineering, with focus on empirical rock mass classification systems and the observational method
- A closer look on two design issues, rock support and grouting, and how the observational method is suggested to be applied in the design of these

Key words: Conceptualisation, Characterisation, Uncertainty, Empirical Rock Mass Classification System, Observational method, Rock Support, Grouting

TABLE OF CONTENTS

PREFACE

This literature review was carried out at the Division of GeoEngineering, Chalmers University of Technology with supervision from Prof. Åsa Fransson. Financial support was provided by the Rock Engineering Research Foundation (BeFo) and the Swedish Research Council for Environment, Agricultural Sciences and Spatial Planning (Formas).

Gothenburg, June 2013

Sara Kvartsberg

1 INTRODUCTION

Construction of underground facilities can be rather complicated due to uncertainties in the ground conditions that represent the bulk of the construction material, strict requirements on the acceptable environmental impact, geometrical restrictions, temporary organizations and complex contractual arrangements (Stille et al., 2003; Lundman 2011). Every project creates a unique result at a specific site, although routine and repeatable activities constitute a substantial part of the construction work (Lundman 2011). The complex and un-repetitive character of underground projects implies that engineering design decisions should be custom-tailored to the site-specific ground conditions and project-specific issues rather than standardised design solutions (Schubert, 2010b; Palmström and Stille, 2007). However, the site-specific ground conditions need to be converted into relevant engineering geological information that is useful for the various engineering works to be carried out in the project.

Empirical rock mass classifications systems attempt to facilitate the rating of the rock mass, although shortcomings are included in their use, for instance when they are used in design applications outside their original intent (Palmström and Broch, 2006). The complex behaviour of geological materials also implies that it is not possible to obtain complete knowledge of actual conditions before construction (Sturk, 1998). Underground construction design will therefore need to deal with unavoidable uncertainties in the ground conditions.

The European standard for geotechnical designs, Eurocode 7 (EN-1997-1) presents the observational method as an appropriate design approach for situations where ground properties and geotechnical behaviour are difficult to predict. The observational method allows for an active adaption of the final design to suit actual ground conditions (Schubert, 2008). Some of the current design practices in Sweden are according to Holmberg and Stille (2007) comparable with the framework of the observational method, although additional demands on transparency, contractual relations and documents are introduced by the formal requirements stated the in Eurocode. Holmberg and Stille (2007) also conclude that interpretation and assessment of rock mass conditions have good potential of benefitting from observational design approaches. What these benefits could imply for the handling of engineering geological information in current practise could need some further clarification and exemplification.

1.1 Objectives and scope of work

The general aim of this study has been to summarize and clarify the current use of engineering geological information in the design of underground projects. Attention has been given to how engineering geological information is handled throughout the construction process and a comparison is made between the traditional design approach and the observational design approach. Two specific study areas are to;

(i) Investigate how engineering geological information is handled throughout the construction process.

(ii) Describe the design process, with special emphasis on two commonly discussed design approaches: empirical design and the observational method.

The study has been carried out as a literature review and considers conceptualisation, characterisation and classification of ground conditions, as well as the handling of geological uncertainties and design work carried out throughout the construction process. An overview of the structure and chapters included in this study is given in Table 1.1. Each chapter is summarized with conclusions.

1.2 Limitations

This study has concentrated on ground conditions, guidelines, construction methods and design work normally encountered in underground rock construction in Sweden. The focus is on the construction phase.

Table 1.1 The included chapters and the main focus of the different chapters.

CHAPTER	TITLE AND CONTENTS
Chapter 1	Introduction *Provides background, aim, scope of work and limitations*
Chapter 2	The underground construction process *Presents the framework for design and the handling of geological information*
Chapter 3	Engineering geological information *Reviews common approaches to conceptualisation, characterisation and classification of ground conditions in underground rock construction*
Chapter 4	Risk and uncertainty in rock engineering *Focuses on uncertainty, risk, risk analysis and the decision-making process*
Chapter 5	Design methods *Gives a background to the different design methods and a comparison between traditional design and the observational design*
Chapter 6	The observational method in rock support and grouting *Presents two design issues and how the observational method can be applied in these*
Chapter 7	Conclusions *Summarises some findings and provides suggestions for future work*

2 THE UNDERGROUND CONSTRUCTION PROCESS

This chapter presents some general characteristics of the underground construction process, with definition of the different phases and stakeholders typically involved in an underground rock construction project.

Underground facilities (e.g. transport tunnels, mines, hydro power plants and caverns) go through a construction process before the operational phase begins. The construction process starts with an idea developed from an identified need and finishes when the facility is brought into operation, see Figure 2.1. The activities normally included in the engineering process are as follows (Hudson, 1993; SKB, 2007):

- Feasibility and location studies
- Cost and optimisation studies
- Environmental impact studies
- Site investigations
- Preliminary design work
- Detailed design work
- Procurement
- Production planning
- Production
- Performance monitoring during and after construction

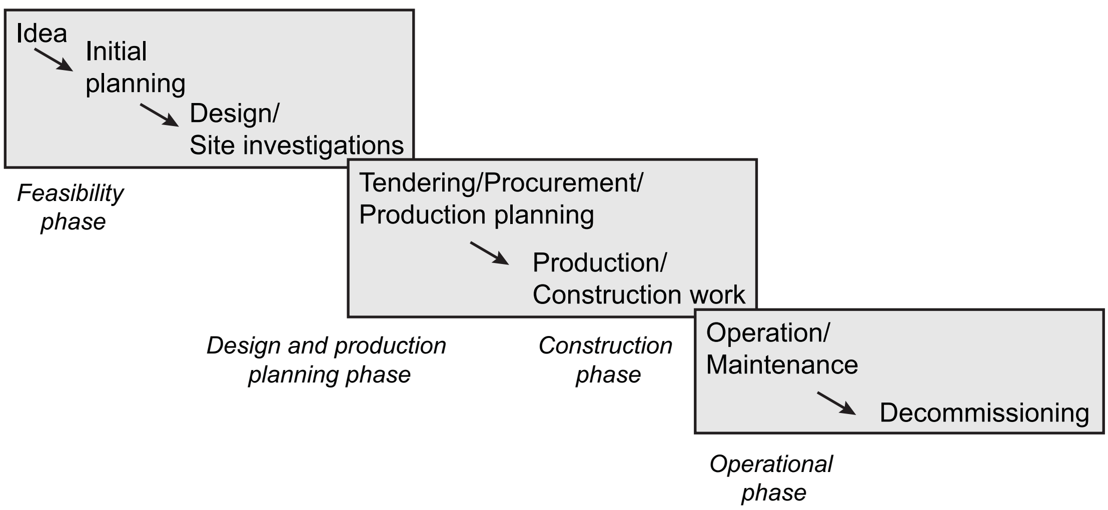

Figure 2.1 Illustration of the process of a construction project, from idea to decommissioning. Modified from Tengborg (1998).

The construction process is often divided into different phases. Sturk (1998) recognises three distinct phases: *the feasibility phase*, *the design and production planning phase* and *the construction phase*. There is no strict definition of what activities should be included in the various phases. The procurement of construction contracts, for instance, could take place either before or after the design and production planning phase. Certain activities could also take place in several phases, which is often the case with site investigations.

2.1 Feasibility phase

The underground construction project starts with the feasibility phase. In this phase, the project idea is analysed with the aim of establishing whether the project should continue. The analysis usually involves assessing a plausible construction method, developing a preliminary layout, identifying environmental impact, assessing time schedules and budgets and deciding on the project organisation (Sturk, 1998). The feasibility phase normally starts with a desk study to determine what is known about the site and how it should be investigated further (Nicholson et al., 1999). The desk study should also identify whether there are any geological conditions that could make the project unfeasible, i.e., recognising stop signs (Sturk, 1998).

The desk study provides important input for the design work required for assessing the suitability of the project (Lindblom, 2010). Often several possible alternatives (corridors) exist and they are examined and compared to each other in terms of the functional, environmental, technical and financial aspects (Sjöberg et al., 2006). The number of alternatives is narrowed down until the most suitable alternative is chosen. The preliminary design work included in these evaluations is often based on professional assessments rather than actual design calculations (Sjöberg et al., 2006). Empirical rock mass classification systems (described in Chapter 5.4) are for instance often used for a preliminary comparison of rock mass quality between different areas (Sturk, 1998).

2.2 Design/production planning phase

If the feasibility study shows that the project should continue, the design and production planning phase begins. Design and production planning rely on good knowledge of the geological and hydrogeological settings and the majority of the required site investigations are normally carried out at the beginning of this phase (Lindblom, 2010). The assessment of geological, hydrogeological and rock mechanical settings, together with project prerequisites (layout and requirements), form the basis for designing appropriate construction measures, such as excavation sequence, rock support and grouting. The design and production planning phase often includes procurement, which involves deciding the contractual arrangements and quality requirements for the project (Sturk, 1998).

Design guidelines for construction of infrastructure tunnels in Sweden are provided by the Swedish Transport Administration (STA). The guidelines list three important documents that need to be prepared during the design phase: the pre-investigation report, the engineering geological prognosis and the construction plan (STA, 2011).

The pre-investigation report compiles the investigation results. The engineering geological prognosis provides a prediction of expected geological conditions at the site that are of technical or financial significance to the project (Holmberg and Stille, 2007). The engineering geological prognosis should also according to STA (2011) include an estimation of the uncertainty in the prognoses, mainly related to the estimates of ground parameters used in design, as well as rock mass quality descriptions according to an established classification system, such as RMR and Q-system (see Chapter 5.4). The construction plan contains drawings and technical descriptions that provide a detailed description of technical solutions for the planned construction concept.

2.3 Construction phase

The production phase takes place when the underground facility is constructed. Investigations may be carried out in parallel or integrated with the construction work. The scope and focus of the investigations depends on the uncertainties that remain after the design and production planning phase (SKB, 2007). Analyses carried out during production typically relate to the final design of construction measures based on ground conditions at the excavation front and identification of features that could disturb the production cycle (Sturk, 1998). Results from monitoring during and after construction are also analysed to ensure the facility satisfies the requirements (SKB, 2007).

2.4 Stakeholders

There are different stakeholders involved in an underground project, with the principal actors being the clients, the contractors and the consultants (Lundman 2011). The client initiates the construction process and often owns the construction after the construction phase has finished. The main client for underground projects in Sweden is the Swedish Transport Administration (STA), although there are other authorities, companies and organisations that construct underground facilities. The contractors produce the construction at the client's request and the consultants carries out design work for the clients or the contractors. The construction process is also affected by legislation, building standards and governmental regulations commissioned by society, thus society is also regarded as an important party in the process (Tengborg, 1998).

Responsibilities and commitments between clients, consultants and contractors are defined by terms stated in contracts. The contracts include measures to allocate risks (Chapter 4.1) and resolves disputes, so the quality and communication of contract specifications play an important role to achieve successful project management (Baynes, 2010). One of the main concerns is to formulate contracts so that deviations from originally assumed ground conditions can be fairly dealt with during the production phase (Tengborg, 1998).

2.5 Concluding remarks

The following concluding remarks on the construction process can be stated:

- The construction process can be divided into the feasibility phase, the design and production planning phase and the construction phase.
- The feasibility phase is mainly based on a desk study of available site information and involves preliminary design work often based on professional assessments and empirical classification systems.
- The main site investigation and majority of the design work takes place during the design and production planning phase. The site investigation results and the design work are compiled in pre-investigation reports, engineering geological prognoses, and in the construction plan.
- Additional site investigations and design work is carried out during the production to aid the production and execute the final design.
- The main actors of the construction process are the client, the contractor and the consultant. The communication of responsibilities and geological information between the different actors are managed with contracts.

3 ENGINEERING GEOLOGICAL INFORMATION

This chapter reviews how engineering geological information is handled within underground construction and considers general aspects of ground models, site investigations and characterisation of ground conditions.

One of the main concerns in underground engineering is the complexity and uncertainty of the ground conditions at the project site. Detailed knowledge of the actual ground conditions cannot be determined until they are revealed at excavation, and the risk of preparing insufficient design alternatives must be balanced against the cost of investigating the ground before construction (Einstein, 1996). The collection and interpretation of relevant data from site investigations is therefore considered an essential part of the construction process. The investigated ground conditions are presented in engineering geological prognoses and these form a basis for decisions throughout the construction process (Sturk, 1998). Traditionally, prognoses are established by collecting and characterising data, creating geological models of the site and grouping material with similar engineering characteristics (classification).

3.1 Conceptualisations and key issues

An underground facility and its interaction with the ground can be seen as a complex system with various problems or key issues that needs to be solved to fulfil engineering requirements and preferences (Stille et al., 2003; Andersson et al. 2000). The analyses of various key issues are handled by models which can be defined as an approximation of reality created for the purpose of solving a problem.

The set of assumptions that describe the model qualitatively and establish the geometric framework is called the conceptual model, whereas application of the conceptual model, with data inserted into a mathematical model, is referred to as model realisation (Gustafson, 2009), see Figure 3.1. The conceptual model is important as it provides structure and identifies key processes that should be translated into a mathematical model. Olsson et al. (1994) developed a structure for presenting and describing models, presented in Table 3.1. This format highlights the essential aspects of each model used and to facilitate comparisons between models.

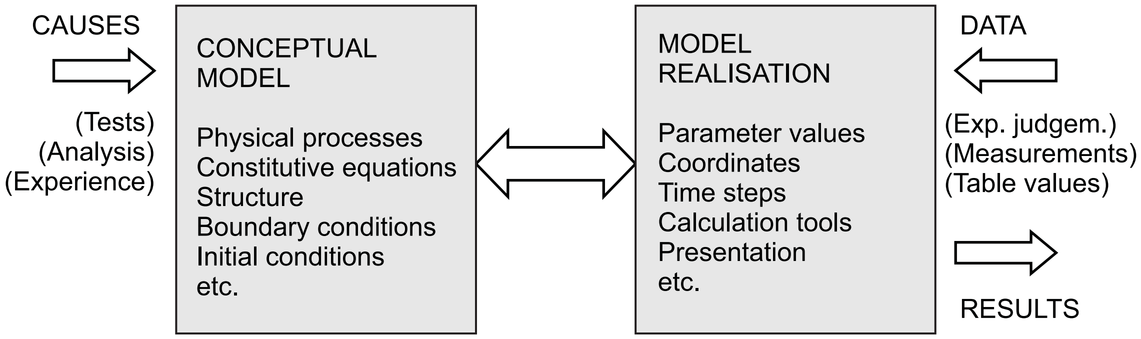

Figure 3.1 A conceptual model and the translation into a quantitative model, modified from Gustafson (2009).

Table 3.1 A structure for conceptual models, from Olsson et al. (1994)

<table>
<tr><td colspan="2">MODEL NAME/DEFINITION</td></tr>
<tr><td colspan="2">Model scope or purpose
Specify the intended use of the model</td></tr>
<tr><td colspan="2">Process description
Specification of the processes accounted for in the model, definition of constitutive equations</td></tr>
<tr><td>CONCEPTS</td><td>DATA</td></tr>
<tr><td colspan="2">Geometric framework and parameters</td></tr>
<tr><td>dimensionality and/or symmetry of model specification of what the geometric (structural) units of the model are and the geometric parameters (the ones fixed implicitly in the model and the variable parameters)</td><td>specify size of modelled volume
specify source of data for geometric parameters (or geometric structure)
specify size of units or resolution</td></tr>
<tr><td colspan="2">Material properties</td></tr>
<tr><td>specification of the material parameters contained in the model (should be possible to derive from the process and structural descriptions)</td><td>specify source of data for material parameters (should normally be derived from output of some other model)</td></tr>
<tr><td colspan="2">Spatial assignment method</td></tr>
<tr><td>specification of the principles for how material (and if applicable geometric) parameters are assigned throughout the modelled volume</td><td>specify source of data for model, material and geometric parameters as well as stochastic parameters</td></tr>
<tr><td colspan="2">Boundary conditions</td></tr>
<tr><td>specifications of (type of) boundary conditions for the modelled volume</td><td>specify source of data on boundary and initial conditions</td></tr>
<tr><td colspan="2">Numerical tool
Computer code used</td></tr>
<tr><td colspan="2">Output parameters
Specify computed parameters and possibly derived parameters of interest</td></tr>
</table>

A subsequent step in the identification of key issues is to analyse how processes and parameters interact with each other and with other design issues in the project (Gustafson, 2009). Hudson and Harrison (1997) present interaction matrices as means to illustrate relationships and influences between different variables. The organisation of an issue in a matrix provides a structure for subdividing a complex problem into smaller, manageable problems. It also facilitates the communication of what is known about the problem. Examples and further details on interaction matrices can be found in Gustafson (2009) and Hudson and Harrison (1997).

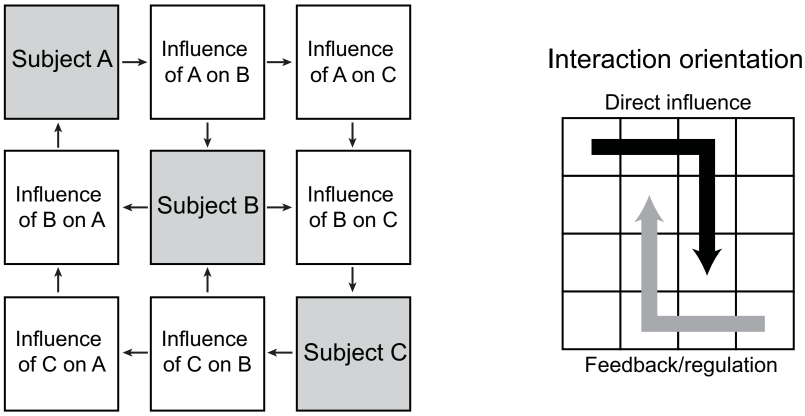

Figure 3.2 Left: Example of a 3x3 interaction matrix after Hudson and Harrison (1997). Right: Interaction orientation, from Gustafson (2009).

3.2 Ground models

Models of geological conditions at construction sites are created to simplify the surface or subsurface conditions for analysis of different engineering applications (Fookes, 1997; Harding, 2004). The models may take the form of tabulated data, written descriptions and annotated 2D or 3D diagrams. Advances in the use of Geographical Information Systems (GIS) and 3D modelling software mean that a 3D model (diagram) of a site can be created and attributed with a wide range of physical, chemical or hydrogeological parameters (Royse et al., 2009).

These site models should, however, be seen as comprehensive site descriptions that serve as input for separate models that describe more specific problems (Andersson et al., 2000; Gustafson, 2009). The starting point for the engineering geological model is the application and not the geology. The same geological settings will interact differently with different engineering applications and will thus require different questions to be asked. Tunnel inflow and interaction between support and ground are two examples of design problems that are expected to have different requirements in terms of input data, boundary conditions and calculation tools, and should be addressed using two separate models.

The knowledge of the ground conditions are preferably structured into disciplines relating to different processes to ensure that important aspects are not overlooked. Examples of such disciplines are geology, thermal properties, hydrogeology, rock mechanics, chemistry and transport properties (Andersson et al., 2000).

Ground models should preferably be introduced in the early phases of the project where they can influence decisions regarding the types of data to be collected (Fookes 1997). The 'total geology' model approach presented by Fookes et al. (2000) and Baynes et al. (2005) emphasises the benefit of preliminary engineering geological models that can guide site investigation planning. The preliminary model is developed during the early project phases, based on an understanding of the geological and geomorphological history of the site. The model is then updated and made more detailed through investigations and subsequent construction. The use of models and an iterative working mode is a natural way of structuring the underground engineering work (Gustafson, 2009).

3.3 Site investigations

Site investigations involve a range of studies and investigations undertaken to describe the subsurface soil, rock and groundwater conditions of a site for engineering applications (Fookes, 1997). Investigations can be separated into direct, intrusive techniques (boreholes and the associated soil/rock sampling and testing), and indirect, non-intrusive techniques (observations and surface/borehole geophysics).

The objectives and level of detail in the investigation programme change throughout the construction process. Financing is generally limited in the early stages and initial investigations aim to generate low-cost information from desk studies and field visits (Baynes et al., 2005). A traditional desk study is based on pre-existing material, such as geological and topographical data (e.g. maps), data from airborne geophysical measurements, aerial photographs, well logs and engineering reports from previous construction activity in the area. Field visits usually comprise visual inspections, geological mapping of the surface, photographing and sampling (Bergman and Carlsson, 1986).

The detailed site investigation carried out during the design and production planning phase should bring information to a level where understanding of the ground conditions is as complete as possible (Fookes, 1997). Site investigations normally conducted include (Gustafson, 2009);

- Geophysics (seismics, resistivity, ground penetrating radar, borehole radar)
- Detailed field mapping
- Exploration by drilling (core drilling, percussion drilling)
- Geotechnical testing (e.g, sounding, standard penetration test)
- Rock mechanical testing (stress measurements, laboratory tests)
- Hydrogeological testing (e.g. pressure build-up tests, investigations of wells)

Supplementary investigations or investigations carried out in parallel with excavation are typically performed to confirm anticipated ground conditions or to yield additional information about critical areas or critical geological features. Examples of critical design situations are shallow tunnels in complex ground conditions, intersecting tunnels, and underground stations with large spans (Sjöberg et al., 2006). Critical geological features could include geological boundaries, fracture zones, weathering, areas with problematic *in situ* stress, water bearing structures and high water pressures (Sturk, 1998).

Important sources of information during excavation are mapping of exposed tunnel faces and probing in advance of the tunnel excavation, e.g. providing lithology of drill cuttings and rate of drill penetration. This information enables the contractor to predict rock quality and positions of water inflows ahead of the tunnel face (Almén and Stenberg, 2005). One shortcoming with investigations integrated within the tunnelling is that they normally are carried out under time constraints between each drill-and-blast cycle. If the fracturing is extensive, as in fracture zones, this may lead to simplified characterisation. In areas with poorer rock the exposed rock may not be stable and the mapping must be executed from a safe distance, meaning that no details can be characterised (Almén and Stenberg, 2005).

The investigations needed for each project are site-specific and typically depend on the requirements of the project, the complexity of the geology and the level of investigation carried out previously in the area (Harding, 2004; Sturk, 1998). The site investigation programme is also cost-constrained, and if the cost of any additional investigation exceeds the value of the expected information, the investigation is not worth performing (Zetterlund, 2009). It is therefore useful to optimise the investigation programme with decision analyses, such as cost-benefit analyses and value of information analyses.

High quality site investigations are considered an important part of a successful tunnel project, both in terms of financial and technical success. Likewise, inadequate investigations are significant contributors to cost and time overruns (Harding, 2004; Riedmüller and Schubert, 2001). Site investigations are ideally planned based on geological models of the site provided by the desk study (Harding, 2004). However, site models are traditionally created after or during site investigation work rather than before and subsurface investigations may be planned in a routine manner with limited focus on the site geology or project-specific issues (Riedmüller and Schubert, 2001; Baynes et al., 2005). Extensive investigations with focus on collecting large amounts of data also have disadvantages, such as cumbersome evaluations of the data. Harrison and Hadjigeorgiou (2012) advocate staged site investigations where collected data influence the subsequent data collection strategy, which differs from the customary 'collect, characterise, design' procedure.

3.4 Characterisation of ground conditions

The procedure of interpreting and quantifying ground conditions for engineering purposes is generally referred to as site characterisation. The characterisation should reflect the material properties without considering any design loadings, such as stress conditions with regard to tunnel direction (Stille and Palmström, 2003). The resulting description, however, is used as input for design tools that take into account design loadings, such as empirical rock mass classification systems and numerical modelling tools, see Figure 3.3.

Several studies emphasise the importance of understanding the difference between systematised characterisation (classification) and the use of empirical rock mass classification systems. Characterisation is the procedure of measuring and/or describing features or parameters of relevance to a project, whereas the empirical rock mass classification is a subsequent step that is part of the design process (Stille and Palmström, 2003).

Characterisation can be simplified by placing the various properties into different pre-defined and generally accepted categories. This grouping of material properties into representative classes can lead to improved understanding of a phenomenon or a set of data (Stille and Palmström, 2003). ISRM (2007) presents a number of methods for systematic description of rock mass parameters, e.g. stages of weathering (fresh to disintegrated), apertures (very tight to cavernous), joint waviness (planar to undulating) and roughness profiles (slickensided to very rough). The structure division of the Geological Strength Index, GSI (Hoek et al., 1998), illustrated in Figure 3.4, is an example of systematic characterisation of rock mass composition developed for rock mass strength.

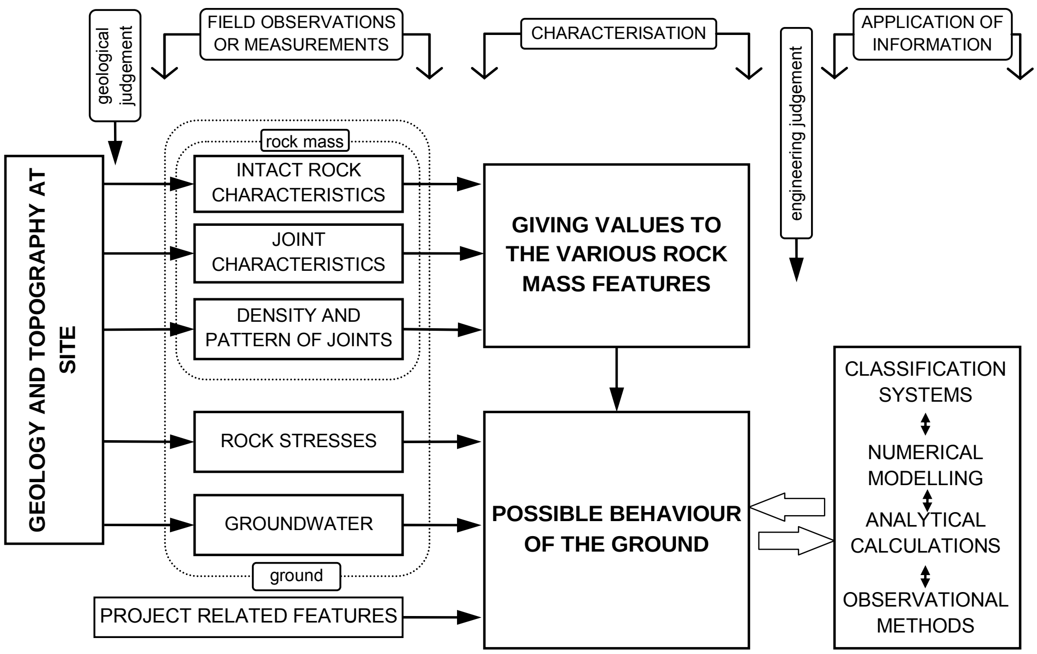

Figure 3.3 Rock mass characterisation and classification (Stille and Palmström, 2003).

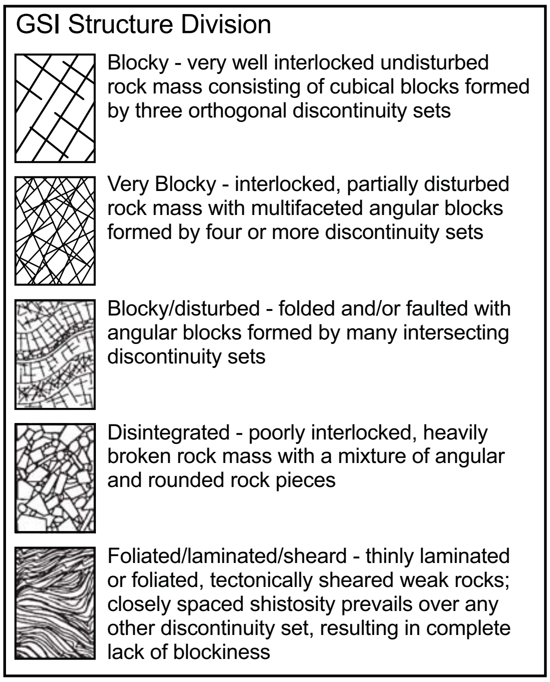

Figure 3.4 Characterisation of rock mass structures in the Geological Strength Index (GSI). Modified from Hoek et al. (1998).

3.5 Categorisation of ground conditions

The subsequent step of assessing relevant rock mass parameters is to determine the type of behaviour expected from the ground surrounding the opening. To facilitate this evaluation, Stille and Palmström (2008) recommend a separation of the geological input of ground conditions between 'rock mass features' and 'forces acting on the rock mass'. The forces that the rock mass is subjected to is mainly rock stresses and groundwater.

The rock mass features can be further divided into two main groups; (i) the host rock and (ii) the deformation zones. Deformation zone is a general term that refers to an essentially two-dimensional structure in a rock mass in which brittle, ductile or combined brittle and ductile deformation has been concentrated (Munier et al., 2003). The deformation behaviour can range between highly localised, brittle deformation and uniformly distributed ductile strain (shearing). Typically, the structure style depends on the rock type, the magnitude of the slip and the physical conditions during deformation, e.g., pressure, temperature and strain rate (Cosgrove et al., 2006). The term fracture zone generally denotes a brittle deformation zone made up of numerous short fractures that together make up a longer, planar zone of weakness in the bedrock (Cosgrove et al., 2006). Fracture zones that display shear movements are commonly referred to as fault zones.

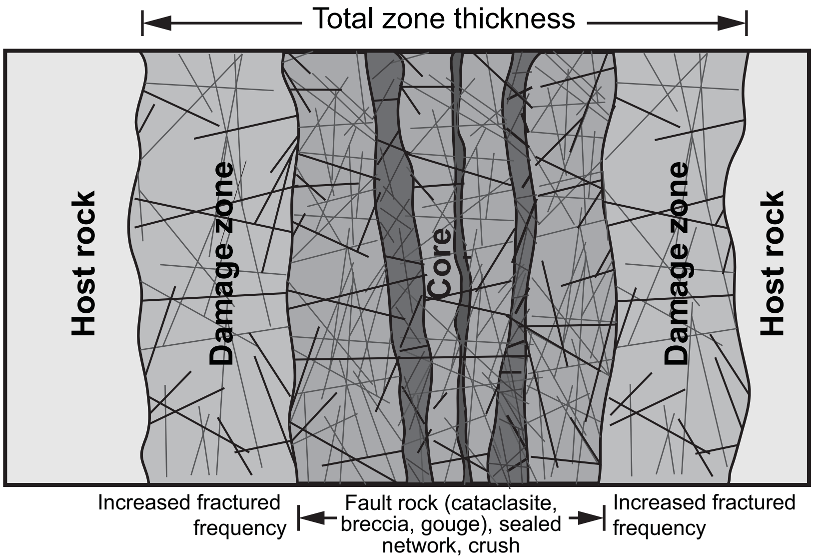

Figure 3.5 Structure of a brittle deformation zone (Munier et al., 2003). The core may be clay-altered and act as a low-permeability barrier against flow and the more intensely fractured damage zone is likely to act as a conduit for the less permeable host rock.

The character of deformation zones can differ significantly from the surrounding rock mass in terms of composition, geometrical constraint and long-term behaviour. The host rock and the deformation zones should consequently be treated separately during investigations, evaluations, and design (Stille and Palmström, 2008). Deformation zones introduce engineering problems, and can provide severe stability problems in connection to excavation (Goodman, 1993). The zones can also offer highly conductive flow pathways within the rock mass although their heterogeneous composition generally results in internally heterogeneous hydraulic properties (Caine et al., 1996). Brittle deformation zones are composed of two main structures of importance for rock engineering aspects; a high-strain core, where the main displacement has occurred, and a low-strain damage zone, see Figure 3.5.

The host rock is mainly described in terms of rock type and the occurrence and conditions of fractures (Palmström and Stille, 2008). The description of fractures can be rather complex, but their impact on the rock mass characteristics are crucial. Figure 3.6 illustrate fracture properties generally considered importance for rock engineering (Harrison and Hudson, 1997).

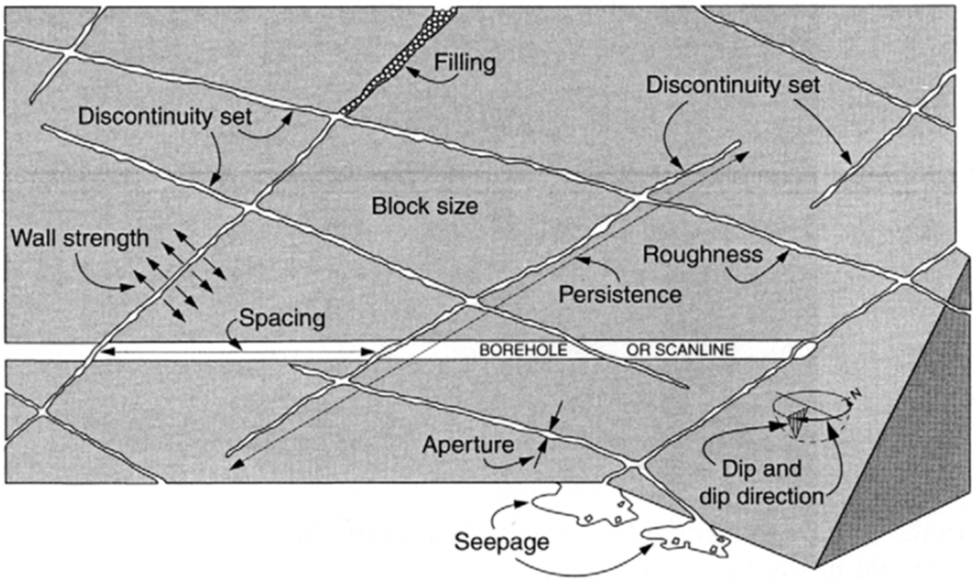

Figure 3.6 Fracture properties of importance for rock engineering, from Harrison and Hudson (1997).

3.6 Concluding remarks

The following concluding remarks on engineering geological information can be made:

- The use of conceptual models and identification of key issues is beneficial for rock engineering design.
- Ground models should be introduced in the early project phases to benefit the site investigation planning. They should be based on an understanding of the geological and geomorphological history of the site and is updated and made more detailed through subsequent investigations.
- Comprehensive ground models can serve as input for separate models that describe relevant geological settings for specific design problems. This since different design problems are expected to have different requirements in terms of input data, boundary conditions and calculation tools.
- The investigations needed for each project are site-specific and the objectives and level of detail in the investigation programme also change throughout the construction process. The collected data should influence the subsequent data collection strategy.
- Characterisation refers to the procedure of interpreting and quantifying ground conditions of relevance to a project. Systematised characterisation reflects material properties without considering any design loadings, and should not be confused with the output from empirical rock mass classification systems.

- Rock mass descriptions should preferably separate between deformation zones and host rock since the character of deformation zones can differ significantly from the surrounding rock mass in terms of composition, geometrical constraint and long-term behaviour.
- The relevant parameters to investigate may differ for various rock engineering issues as they are expected to have different requirements in terms of input data, boundary conditions and calculation tools.

4 RISK AND UNCERTAINTY IN ROCK ENGINEERING

This chapter provides a brief overview of the concept of uncertainty and risk in geotechnical engineering. Managing uncertainty is also linked to the decision process, which is also considered.

Construction of underground facilities involves uncertainties that give rise to geotechnical risks in the decision-making process, i.e. situations that possibly involves a loss, disaster or some other undesirable outcome (Hammah and Curran, 2009). An important part of decision-making in underground construction is to balance the risk-taking and evaluate the consequences of the decisions taken, which is often made using qualitative, subjective design strategies, such as engineering judgement. Risk analysis and other reliability based approaches are useful complements to the qualitative strategies since they provide a systematic approach for decision-making that could lead to better decisions (Einstein, 1996; Einstein and Baecher, 1982; Sturk, 1998).

4.1 Geotechnical risks

A decision can be defined as the choice between different alternatives. Risk analyses are carried out to estimate risks associated with various alternatives (Nilsen and Aven, 2003). The terminology within risk analysis varies amongst different scientific disciplines and it is important to describe clearly how the terms are used in a specific application. According to Nilsen and Aven (2003) and Ang and Tang (2007) is the aim of the *risk analysis* to quantify to what extent a potential undesirable consequence threatens the performance of a given activity. The *risk* is the combined effect of the probability of an undesirable event and the consequences that may follow. The event or situation that has the potential to cause harm is also known as *hazard*.

Various types of geotechnical risks have been discussed in literature on the subject. Baynes (2010) adopts a framework that separates 'technical risks' derived from uncertainties associated with the geological model, design properties and engineering analyses from risks that arise from managing and communicating risks, i.e., 'project management risks' and 'contractual risks'.

Project management risks

Project management involves managing geotechnical risks and establishing risk registers. Project management risks often develop during early project phases if risk-mitigating measures are not implemented (Baynes, 2010).

Technical risks

Technical risks derived from uncertainties in the geological model may be a result of variability in the behaviour of geological materials and their spatial distribution and resolution (scale) (Harrison and Hadjigeorgiou, 2012). Geological uncertainties may also evolve from a lack of knowledge or understanding of the geological conditions, which is a main reason for geotechnical design problems (Baecher and Christian, 2003). Variability due to naturally variable phenomena in time or space can be termed 'aleatory', whereas uncertainty due to a lack of knowledge or understanding is referred to as 'epistemic' (Baecher and Christian, 2003).

Epistemic uncertainties include uncertainty as to whether the engineering analyses and mathematical models accurately represent reality (referred to as analytical or model uncertainty). Inappropriate choices may result from a lack of understanding of what is important and insufficient knowledge of weaknesses or assumptions in the models (Hammah and Curran, 2009). Errors associated with data collection, characterisation and parameter estimations (property or data uncertainty) are also common sources of epistemic uncertainty (Bedi and Harrison, 2012).

Contractual risks

Poor acquisition, understanding and/or communication of geological information can cause contractually 'unforeseen' ground conditions, which often lead to geotechnical risk claims (Baynes, 2010). The communication and interpretation of data from site investigation reports are therefore associated with contractual risks. Contractual risks can arise, for instance, as a result of insufficient communication between site investigation personnel and designers (Hadjigeorgiou, 2012). This could lead to a lack of critical data in later phases because the design needs were not understood and communicated to the site investigation personnel.

4.2 Handling of uncertainties

Uncertainty is an inevitable part of rock engineering. However, traditional design and construction approaches often considers uncertainty in an unsatisfactorily way, especially when encountering difficult rock conditions (Einstein and Baecher, 1982; Stille and Palmström, 2008). The handling of geotechnical uncertainties is a central issue for any underground project and a certain degree of flexibility and sensitivity needs to be included in the construction process to avoid costly consequences of unforeseeable conditions (Goodman, 1993). Baynes (2010) relates certain established techniques to handle geotechnical uncertainties to idealised phases in a project, which is illustrated in Figure 4.1. The techniques mentioned include the following:

- Use of risk registers (documentation of perceived risks) for overall management of geotechnical risks
- Adequate and comprehensive site investigations, preferably staged (parallel with design) and carried out by multidisciplinary teams
- Well-defined reports of investigation results

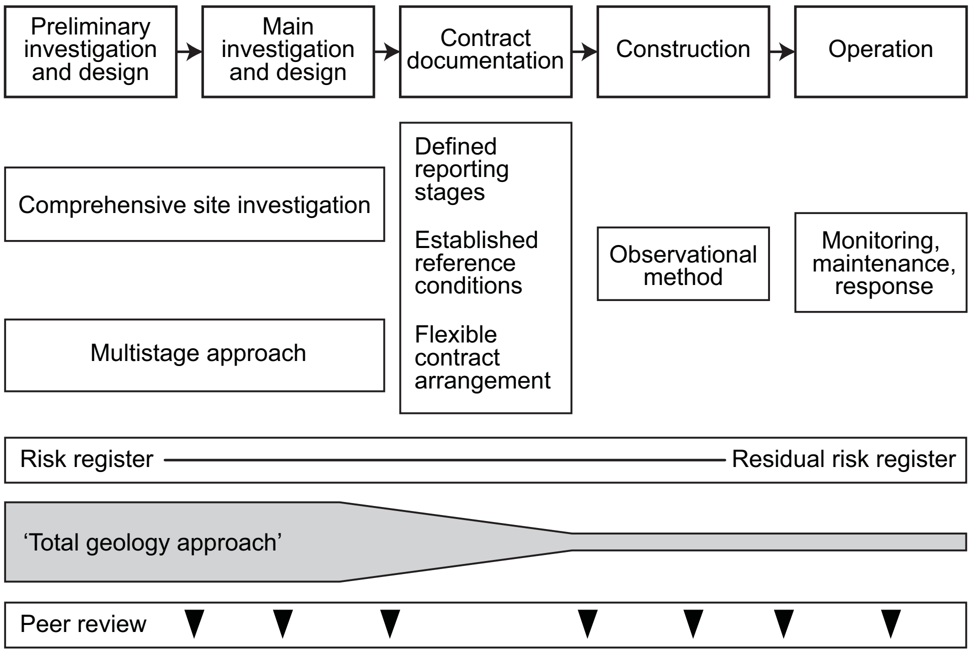

Figure 4.1 Techniques for managing geotechnical risks in various project phases. Modified from Baynes (2010).

- Peer reviews of critical milestones and tollgates in the project
- Implementation of the observational method
- Efforts to understand the geology and define reference conditions during the early phases ('total geology approach').

Adequate and comprehensive site investigations are generally considered to be important for reducing geological uncertainties (Harrison and Hadjigeorgiou, 2012), and a study presented by Lundman (2011) shows that there is a correlation between the quality of site investigations and the accuracy of the geological prognoses. The quality of the geological prognoses, for instance, is influenced negatively by poorly performed work and data registration, outcrop observations not being representative of the situation underground, limitations in core drilling and core logging, incorrect observations and use of inappropriate investigation methods for the geology in question (Palmström and Stille, 2010). It is difficult, however, to provide general recommendations for appropriate methods and a suitable number of investigations. The required types and number depend on the character and complexity of the project (Palmström and Stille, 2010). Analysis of the value of the new information (e.g. using VOIA) can provide a strategy for the rational design of a field investigation programme.

Other techniques for reducing uncertainties mentioned in the literature include paying attention to problem identification, i.e., the key issues to be considered, the data to be collected and the reason for collection (Palmström and Stille, 2010; Gustafson, 2009). Furthermore, information in geological prognoses should preferably be communicated in a form that can be understood by the parties involved in the project. Engineering standards and codes of practice, such as those described by ISRM (2007), are considered useful for reducing linguistic uncertainties. Visual representation of data (e.g. drawings, maps and photographs) are beneficial as they enables those with little or no understanding of the descriptive terms in geological prognoses to gain an idea of the geological settings (Hammah and Curran, 2009; Baynes et al., 2005).

4.3 Quantification of uncertainties

Uncertainty can be analysed subjectively without statistical tools or models. However, a risk analysis prerequisite is that the uncertainty can be quantified in terms of 'probability'. Probability can be expressed as the relative frequency with which an event is expected to occur (Christian, 2004). Probability can also be expressed as the degree of belief justified by evidence, which is a view associated with 'Bayesian' statistics. The Bayesian statistics holds probability as a subjective measure of uncertainty, either meaning that there is a lack of knowledge about a process or that purely personal degrees of belief are involved. The probability can then be seen as a measure of confidence in an uncertain outcome (Burgman, 2005; Christian, 2004). In Bayesian statistics it is possible to combine expert judgements with measurements in a mathematical formal manner (Burgman, 2005).

There are a large number of probabilistic methods available for describing and handling uncertainties and guiding decision-making. Most of them can be placed in one of two categories; event-oriented (logical trees) and quantity-oriented (also referred to as direct reliability analyses) (Christian, 2004; Nilsen and Aven, 2003). The event-oriented models, such as event trees (Figure 4.2) or fault trees describe the causes and consequences of events and are composed of conditions and logical terms.

The quantity-oriented approach describes a specific quantity by a physical function (Nilsen and Aven, 2003). The uncertainty of this specific quantity can be expressed by incorporating probability distributions into the input variables, resulting in a probability distribution of the outcome quantity. Several methods can be used for determining the resulting probability distributions, such as point-estimate methods and Monte Carlo simulations. The Monte Carlo simulation (Figure 4.2) is a numerical technique which uses random variables to sample the values from the model input probability distributions (Ang and Tang, 2007). The process is performed iteratively to produce an entire probability distribution that represents the outcome quantity.

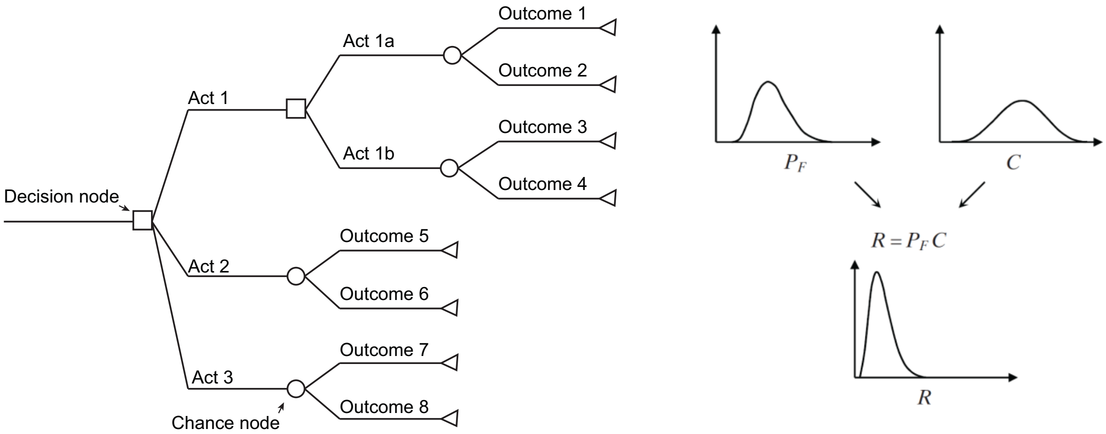

Figure 4.2 Left: example of an event tree. Right: illustration of the principles of a Monte Carlo simulation (from Lindhe, 2010)

Aleatory and epistemic uncertainties must be treated differently in the risk analysis (Christian, 2004). Uncertainty in parameters derived from inherent random (aleatory) variability cannot be reduced by conducting additional investigations; it can only be quantified and handled using stochastic models and probability theory. Conversely, it is possible to reduce epistemic uncertainties through additional information but it is not possible to define them using statistical distributions (Bedi and Harrison, 2012). However, Bedi and Harrison (2012) showed that epistemic uncertainties can be quantified using uncertainty models, such as interval analysis. With these attempts, it may become justified to characterise the uncertainty as aleatory variability and use probabilistic analysis.

4.4 The decision process

Einstein (1996) presents a general decision-making process as a cycle including the following steps:

- Collecting information, including determination of uncertainty
- Deterministic and probabilistic performance modelling (analysis)
- Decision-making
- Updating (collect additional information, modify parameters)

To facilitate the decision process, several decision analysis methods have been developed, e.g. cost/benefit calculations and expected utility optimisations. Aven and Kørte (2003) emphasize that the decision-making process should be separated from these decision analysis methods. They argue that decision analysis methods are decision aids rather than methods for recommendation of choice. Moreover, they argue that it is more useful to put attention to each consequence and its associated uncertainty separately, and make the weighting of consequences and risk as part of a less formal review and judgement process, see Figure 4.3. In this approach, the final decision process is influenced by the results from the various analyses, but it also considers other relevant factors not included in the models, such as preferences by the stakeholder.

A simple decision analysis may be qualitative and entirely based on subjective assessments, i.e. a deterministic approach. A more advanced and formalized decision analysis is probability-based and involves risk analyses and quantification of uncertainties and consequences. The principles underlying the formal risk analysis forces the decision maker to structure the problems and analysing the risks, which may be more important than calculating the mere numbers (Einstein and Baecher, 1982).

Most of the decision analysis models available make use of probabilistic methods. Besides effectively dealing with uncertainty, the probability methods may also specify the relative contribution of different parameters to the uncertainty of the result, thus indicate where further investigations will be most fruitful (Christian, 2004). If there are substantial uncertainties in the estimations, it might be preferable to make sensitivity analyses to investigate if the decision is likely to change due to changes in the input (Stille et al., 2003). However, there are shortcomings in the use of probability methods, mainly due to problems in the estimations of parameters and representation of uncertainty as probabilities due to inadequate knowledge (Christian, 2004).

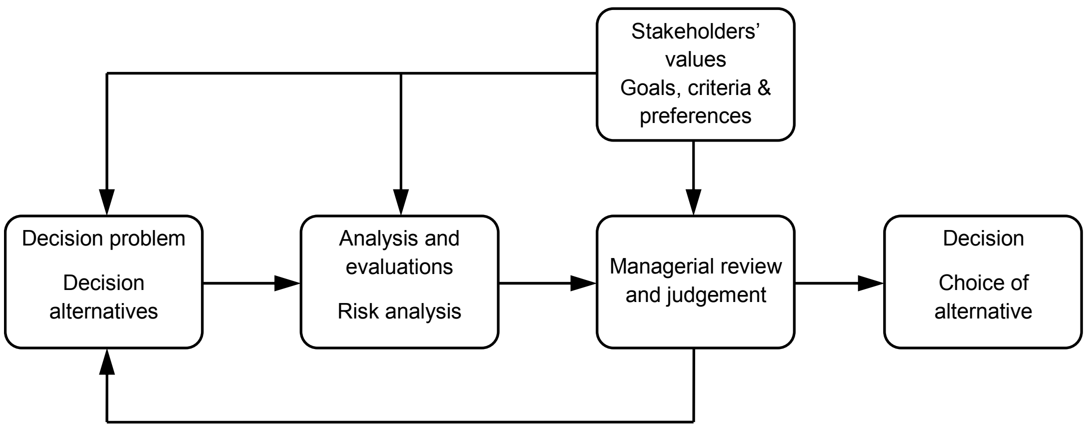

Figure 4.3 Basic structure of the decision-making process (modified from Aven and Kørte, 2003)

4.5 Concluding remarks

The following can be concluded from the studied literature on uncertainties, risks and the decision process.

- Geotechnical risks can be divided into those associated with project management and those related to contractual and technical matters. Technical risks are derived from uncertainties in geological models, engineering analyses and data collection. The uncertainties may be divided into aleatory and epistemic uncertainties, which are treated differently in the risk analysis.
- There are a number of established techniques for reducing geotechnical uncertainties, e.g. risk registers, adequate site investigations, peer reviews, the observational method and use of engineering standards. Probabilistic methods, such as logical trees and the point-estimate method are used to express uncertainties quantitatively, which is needed for implementation of risk analyses.
- The decision process is preferably seen as a process separated from decision analysis methods (e.g. cost-benefit analyses). Weightings and final decisions should be part of a less formal process. However, decision analysis methods and risk analyses are useful since they provide structure and can indicate areas that need further investigation.

5 DESIGN METHODS

This chapter explains various methods for implementing underground rock construction design, including the separation between preliminary and final design. Focus is on reviewing empirical design approaches and the observational method.

Design work for an underground facility generally involves determining the alignment and layout of the facility as well as the engineering work needed to fulfil project requirements (SKB, 2007). In addition to requirements on constructability, working environment, durability and environmental impact may also contractual, financial and political aspects be considered (Andersson et al., 2000; Gustafson, 2009).

A characteristic feature of underground construction is that the actual properties and behaviour of the ground will not be known in detail until excavation is completed (Stille et al., 2003). The design and construction of the underground structure are therefore part of an iterative process, normally separated into a preliminary design prepared before construction and a final design prepared during construction (Palmström and Stille, 2007). Some design decisions, such as alignment, are finalised in the early phases when information is generally limited, whereas the final design of certain construction measures can be postponed until excavation information becomes available. Preliminary design is, however, needed to make reasonably accurate time and cost estimates (Stille et al., 2003).

5.1 The preliminary design process

A basic structure of the preliminary design process adopted by the Austrian guidelines for the geotechnical design is described by Goricki et al. (2004), see Figure 5.1. The preliminary design is divided into two main parts: the first part refers to ground conditions and ground behaviour while the second part deals with system behaviour. Basically, the assessment of expected ground conditions is the result of ground characterisation. Anticipated ground behaviour is based on ground conditions and the influence of excavation without taking into account the effect of support or other construction measures. System behaviour involves the design of appropriate construction measures (excavation sequence, support and grouting) and is the result of the interaction between the measures taken and ground behaviour. The results from the system behaviour analyses are compared to the design requirements to ensure these are fulfilled before the design concept can be established (Goricki et al., 2004).

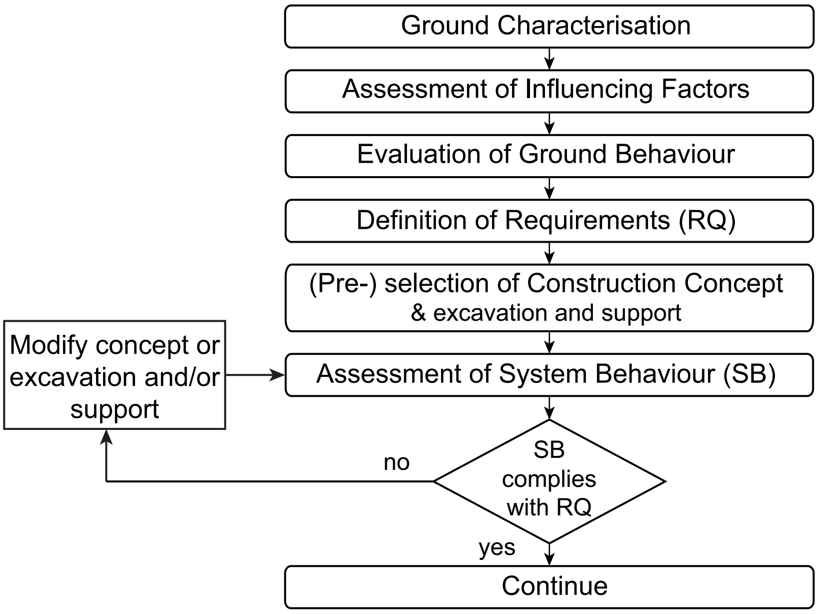

Figure 5.1 A simplified version of the basic structure of the preliminary design process, described by the Austrian guideline (modified from Schubert, 2010b)

The preliminary design process emphasises that design should be based on actual ground behaviour and project-specific issues rather than using standardised design solutions (Palmström and Stille, 2007). Furthermore, it emphasises that variations in requirements and boundary conditions may lead to different construction measures in areas that are considered to have the same ground behaviour (Goricki et al., 2004). However, an issue that is raised by Schubert (2010b) is that the anticipated behaviour is difficult to define and ground behaviour and system behaviour must be described much more precisely than is currently the case in practice.

5.2 Design approaches in the Eurocode 7

The main product of the design work is an assessment of the ground conditions expected in the project area and a description which presents layout proposals and the premises for the different rock works to be carried out, presented in design- and technical specifications (Sjöberg et al., 2006).

The European standard for geotechnical designs, Eurocode 7 (EN-1997-1), presents four approaches to designing geotechnical structures;

1. Design by calculations
2. Design by prescriptive measures
3. Load tests and tests on experimental models
4. The observational method

Design by calculations is a common design approach for underground construction and includes the partial coefficient method and probability-based calculation methods. With these methods, the final design is determined in advance of construction, based on conservative ground parameters that take account of uncertainties inherent in natural ground conditions. Monitoring during construction is carried out to verify assumptions regarding ground conditions and to confirm that system behaviour is within acceptable limits (Schubert, 2010a; SKB, 2007).

Design by prescriptive measures provides design solutions to problems based on experience from similar cases. Experience-based systems, such as empirical rock mass classification systems (see Chapter 5.4), are common in rock engineering design although empirical design is heavily disputed (Schubert, 2012). *Load tests and tests on experimental models* are not really applicable to tunnelling (Schubert, 2010a).

The observational method is presented as a suitable design approach for situations where ground properties and geotechnical behaviour are difficult to predict. The observational design approach uses observations and measurements carried out during construction to actively adapt the final design to suit actual site conditions (Einstein and Baecher, 1982). Further description of the observational method is given in Chapter 5.5.

5.3 Final design/design classes

Technical design solutions presented in the construction plan are normally summarised and described using 'construction classes', also labelled 'design classes'. These correspond to different design options for use when varying ground conditions and requirements are encountered during construction, i.e. to execute the final design (Holmberg and Stille, 2007). Examples of possible classes for design of rock support is seen Figure 5.2. The preparation of pre-defined, stepped solutions is useful since it speeds up decisions during production. They should preferably include just a small number of classes since a large number of classes may complicate communication between users (Stille and Palmström, 2003). Typically, pre-defined design classes are adapted to project-specific requirements (Gustafson, 2009), see schematic representation in Figure 5.3. These can vary, for instance, according to different layouts, varying sensitivity of the surroundings, or the existence of nearby constructions.

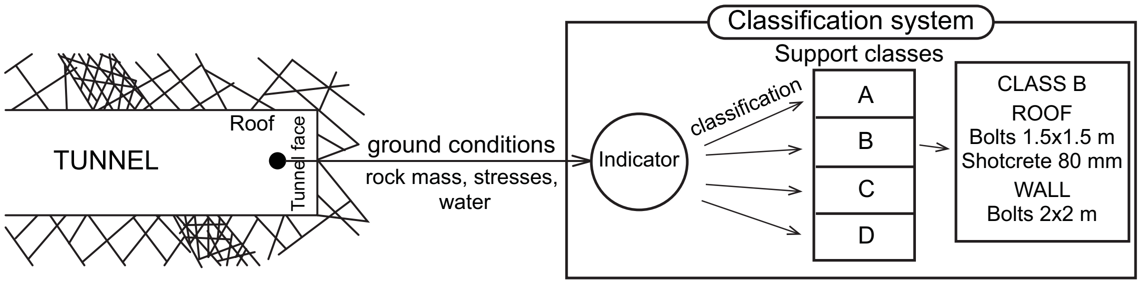

Figure 5.2 An example of a classification scheme for support classes (modified from Stille and Palmström, 2003)

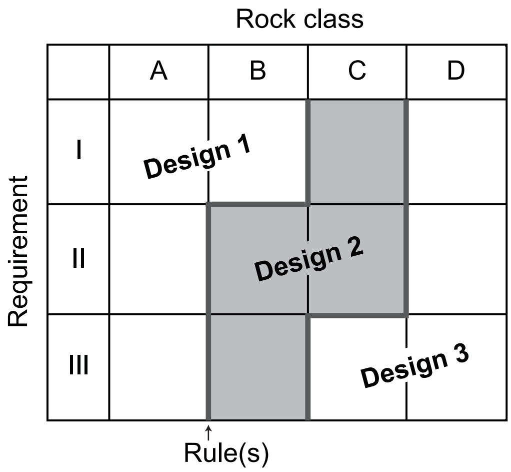

Figure 5.3 ”Rule-matrix” for steering and assigning of design classes (modified from Gustafson, 2009). The rules are the selection criteria that are observed to be able to decide which design class apply for the subsequent excavation stage. For the same geological setting, different designs may need to be implemented due to differing requirements.

Design classes are also adapted to various expected ground conditions and ground behaviours at the site, i.e. the geological settings (Gustafson, 2009). Ground conditions are the product of the geological and geomorphological history at the site, and they can be described by the properties and spatial distributions of geological structures (faults, folds, and fractures), rock types, soils and their boundaries. The ground behaviour is the way the ground acts in response to the ground conditions, the added structure and various processes (e.g., chemistry, rock stresses, and groundwater) influencing the ground at a regional and local scale (Fookes et al., 2000). The geological settings should be described by relevant engineering parameters and the settings are normally grouped together in classes with similar engineering characteristics, often labelled 'rock classes', see Figure 5.3 (Stille and Palmström, 2003).

Rock classes are more or less homogenous with regard to the engineering properties being studied and according to Baynes et al. (2005) should they depict the range of conditions that can reasonably be anticipated or foreseen at the site. Empirical rock mass classification systems can be used as a basis for establishing indicators and intervals for the rock classes. However, there are limitations on their use, which could lead to inadequate geological classification and subsequently an inappropriate design. Stille and Palmström (2003) argue that empirical classification systems provide an averaged value of the site conditions and cannot accurately characterise the conditions that occur at a tunnel location. It is therefore suggested that rock classification should be adapted to site-specific ground conditions and project-specific design considerations.

Decisions on the actual rock class and corresponding design class are associated with uncertainties and geotechnical risks. Misclassification can lead to unwanted consequences, both in terms of unnecessary cost due to a conservative design or failures due to insufficient design (Palmström and Stille, 2010). The uncertainties in the classification and the risk of misclassification should therefore be assessed, although describing and dealing with uncertainty are not straightforward (Chapter 4).

5.4 Empirical rock mass classifications systems

The empirical rock mass classification systems were originally developed to enable rating and ranking of the rock mass and to collate experiences gained at different sites in order to assist in the engineering design (Bieniawski, 1988; Barton and Bieniawski, 2008). The rock mass classification systems are therefore also referred to as empirical design methods (Stille and Palmström, 2003).

The classification systems have sets of parameters that are considered relevant for describing the behaviour of the rock mass, often for design of structural resistance. The parameters are quantified and given ratings that result in a numerical value, which can be used to divide the rock mass formation into separate classes (Stille and Palmström, 2003). Frequently used classification systems for civil engineering applications are the Geomechanics Classification System (RMR) system (Bieniawski, 1988), the Q-system (Barton, 2002), the Geological Strength Index (GSI) (Hoek et al., 1998), and the Rock Mass index (RMi) system (Palmström, 1996).

RMR-system

The Rock Mass Rating (RMR) System, also known as the Geomechanics Classification, was introduced by Bieniawski (1973) and has been modified several times since then.

In the RMR system, the following six parameters (Bieniawski, 1988) are used to classify the rock mass:

1. Uniaxial compressive strength of rock material
2. Rock quality designation (RQD)
3. Spacing of discontinuities
4. Condition of discontinuities
5. Groundwater conditions
6. Orientation of discontinuities

To apply the RMR classification, the rock mass is first divided into a number of structural regions and the classification parameters for each structural region are determined from field- and laboratory measurements. The classification parameters yield ratings which sum up to a total RMR value, belonging to one of five rock mass classes. The output from the RMR classification is the stand-up time and maximum stable rock span for a given rock mass rating (see Figure 5.4) and the rock mass classes give guidelines for selection of rock support in tunnels.

One advantage with the RMR-system is that the parameters are relatively well-defined and the ratings for each parameter can be estimated with an acceptable precision (Stille and Palmström, 2003). However, the RMR system does not consider the rock stresses, and the influence of water on stability is unclear. Using RMR as the only indicator for rock support could therefore be considered unsuitable, especially in areas where rock stresses are important for the ground behaviour.

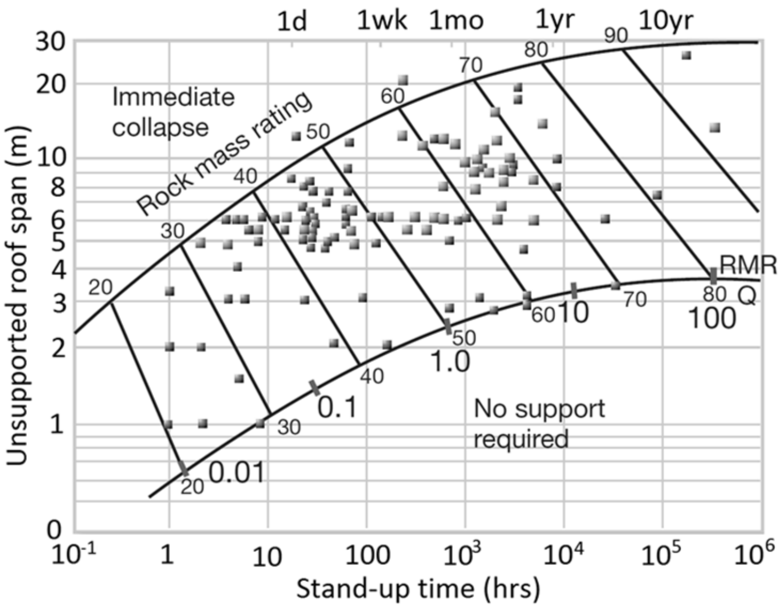

Figure 5.4 Stand-up time data versus RMR, with a conversion of RMR to Q (Barton and Bieniawski, 2008).

Q-system

The Q-system, introduced by Barton et al. (1974), rates the following six parameters (Barton, 2002):

1. Rock quality designation (*RQD*)
2. Number of joint sets, J_n
3. Joint roughness number (of least favourable discontinuity of joint set), J_r
4. Joint alteration number (of least favourable discontinuity or joint set), J_a
5. Joint water reduction factor, J_w
6. Stress reduction factor, *SRF*

The six parameters are given numerical ratings and are combined into a Q-value by the following expression:

$$Q = \frac{RQD}{J_n} \cdot \frac{J_r}{J_a} \cdot \frac{J_w}{SRF} \qquad \textit{Eq. 1}$$

The three pairs of ratios represent the relative block size (RQD/J_n), the minimum inter-block shear strength (J_r/J_a) and active stress (J_w/SRF). The possible Q-values range from approximately 0.001 to 1000, and support recommendations are given when the value is combined with the dimensions of the tunnel or cavern in a Q-support chart (Figure 5.5). Size, continuity, orientation and persistence of fractures are not included in the Q-system. The degree of jointing (block size, density) is characterised with *RQD*, which is for example insensitive for large frequencies and often subjected to sampling bias (Palmström and Broch, 2006).

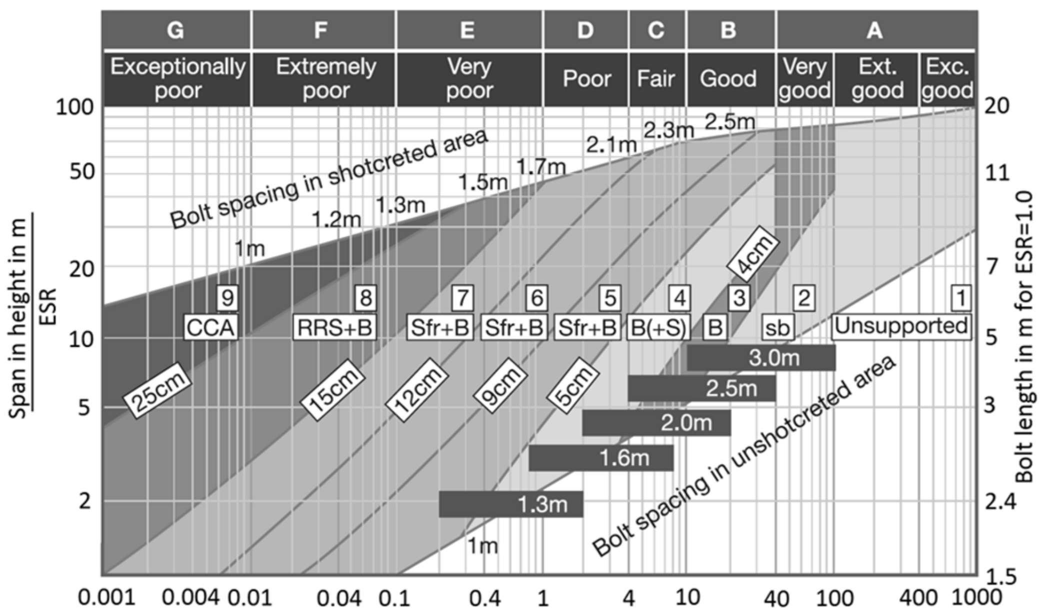

Figure 5.5 The support selection chart for the Q-system (Barton and Bieniawski, 2008)

The Q-system works best in jointed rock masses where instability is caused by block falls. For most other ground behaviour it has limited characterising the conditions in such a way that suitable support can be chosen (Palmström and Broch, 2006).

RMi

The RMi, Rock Mass index, was developed by Palmström (1995) to characterise the strength of rock masses for construction purposes. The RMi value is a volumetric parameter that indicates the reduced rock strength caused by jointing and is expressed as (Palmström, 1996):

$$RMi = \sigma_c \cdot JP \qquad \textit{Eq. 2}$$

where σ_c is the uniaxial compressive strength of intact rock measured on 50-mm samples and JP is the jointing parameter, which is a reduction factor based on block volume, Vb, and the joint condition factor jC. The joint condition is described by the joint roughness, jR, the joint alteration jA and the joint size, jL;

$$jC = \frac{jR}{jA} \cdot jL \qquad \textit{Eq. 3}$$

The joint size factor, jL, is included as a larger joint has a stronger impact on the behaviour of a rock mass than a smaller joint (Palmström 1995). The roughness factor (jR) is similar to the J_r in the Q-system, and can be determined from descriptive charts or from measured values of JRC (the joint roughness coefficient). The relation between the input parameters to the RMi value can be seen in Figure 5.6. The RMi value can be combined with ground factors, such as rock stresses, and the geometrical layout to estimate rock support from support charts. It can also be applied as an input to numerical modelling, the Hoek-Brown failure criterion and to estimate the deformation modulus for rock masses (Palmström and Singh, 2001).

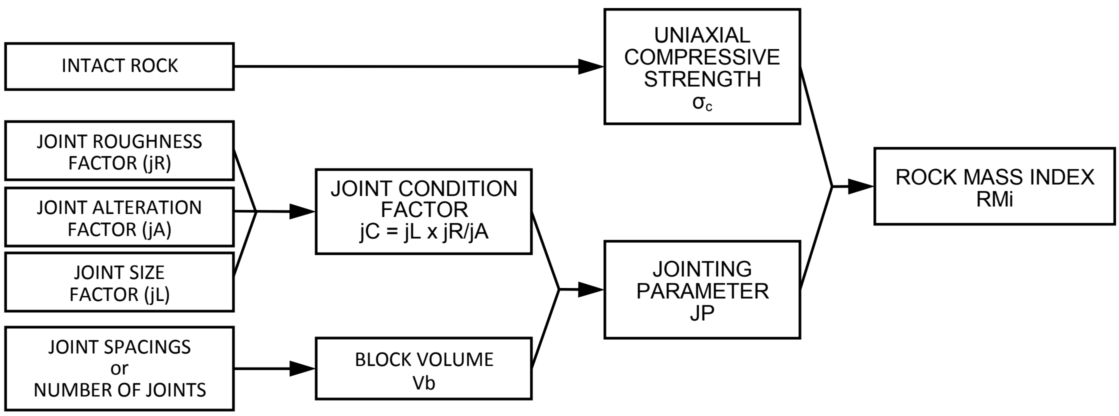

Figure 5.6 Input parameters to the RMi system (modified from Palmström, 1995)

GSI

The geological strength index (GSI) provides a system for estimating the reduction in rock mass strength for different rock mass conditions (Hoek et al., 1998). The conditions are identified by field observations of the rock structure in terms of blockiness, and the surface conditions of fractures indicated by roughness and alteration. The GSI value is determined from the combination of these two parameters according to Figure 5.7, and this combination describes rock structures ranging from tightly interlocked strong rock fragments to heavily crushed rock masses.

The resulting GSI value is entered into a set of empirically developed equations for estimating mechanical properties of the rock mass, in particular the compressive strength of the rock mass (σ_{cm}) and its deformation modulus (E_m). Besides the GSI value are values for the uniaxial compressive strength σ_{ci} and the material constant m_i needed. These strength parameters are determined from laboratory testing or estimated from published tables (Hoek et al., 1998).

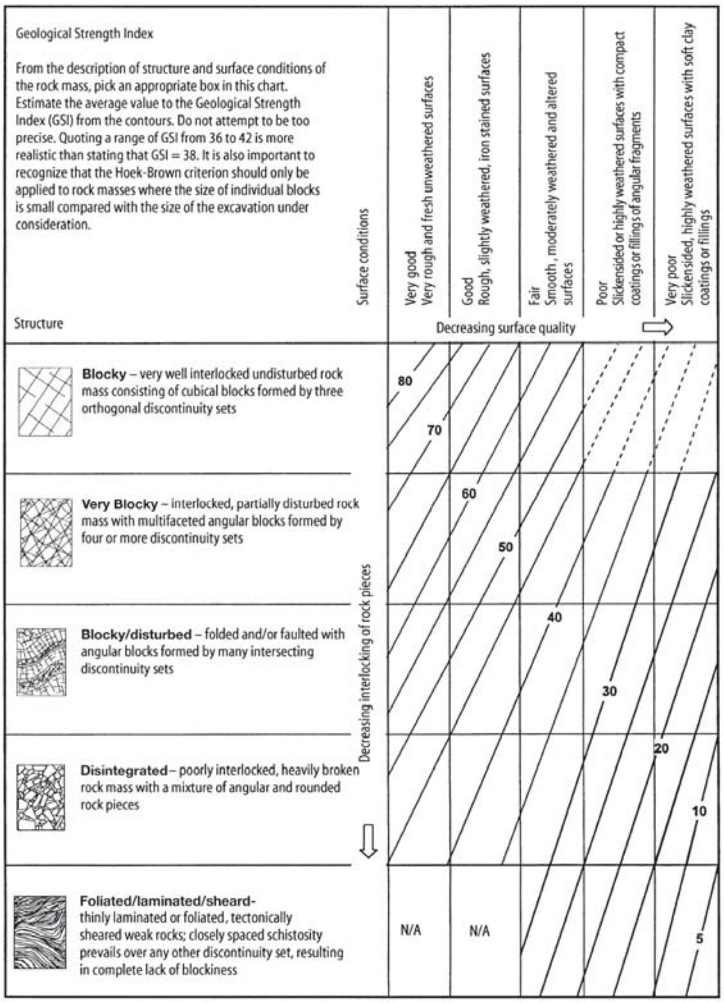

Figure 5.7 The GSI chart for fractured rocks. From Hoek et al. (1998).

Advantages and shortcomings with empirical classification systems

There are quite a number of publications that discuss the advantages and shortcomings of empirical rock mass classification systems, e.g. Riedmüller and Schubert (1999), Palmström and Broch (2006), Barton and Bieniawski, (2008) and Pells (2008). A commonly discussed shortcoming includes the loss of valuable information of the rock mass structure when various rock mass properties are combined into a single index. The rating can be the outcome by various combinations of the classification parameters, thus the same rock classification value may originate from totally different rock mass conditions. This leads to homogenisation of the rock mass, which does not take account of differing failure modes, boundary conditions or anisotropic and time-dependent behaviours of the rock mass (Riedmüeller and Schubert, 1999). The complex interaction between factors such as fracture orientation, degree of fracturing, fracture shear strength and stress conditions are according to Riedmüller and Schubert (1999) not sufficiently considered.

A general problem in all classification systems is the establishment of appropriate fracture characteristics (Palmström and Broch, 2006). Different people may map fractures differently and the parameters may be mischaracterised. In a comparative study of RMR, Q-system and RMi performed by Nilsen et al. (2003) it was shown that some parameters in each method had relatively high variation in rating values among the different observers. A clearer description of how to rate the different parameters, and a possibility of rating values in a range rather than with a single value is suggested to reduce such errors.

Another problem with rock mass classification systems is that their broad level of acceptance tends to make them expand to areas for which they were not originally developed. RMR and Q were developed to estimate support for small-scale civil engineering tunnels in fairly good rock mass conditions where instability is caused by block falls. Their application, however, has been extended to include the design of support for slopes and large mining structures, to specify the need for grouting, to assess modulus of deformation and to predict advance rates for tunnel boring machine (TBM) tunnelling. None of these extended applications are recommended by Palmström and Broch (2006).

Empirical design tools, such as rock mass classification systems, have the advantage of being used frequently and they have simple, practical applications (Riedmüller and Schubert, 1999). They enable ratings of the rock mass quality to be made when little detailed information about the rock mass is available and they can therefore be of considerable benefit for preliminary planning purposes (Palmström and Broch, 2006). The systems may also be used as checklists to ensure that relevant information is gathered and characterised for its intended application. The classification can also ease the communication between different parties involved in a project (Stille and Palmström, 2003).

Several papers (Bieniawski, 1988; Pells, 2008; Palmström and Broch, 2006) conclude that rock mass classifications cannot be taken as a substitute for engineering design. A quantitative rock mass classification, which is merely based on a few universally applied rock mass parameters, simplifies complex problems in underground excavations and can never alone form a basis for a technical and economical optimization of excavation and support in a tunnel. Their best use is as applications during the early phases of a project, e.g. for feasibility and route selection studies or when making a preliminary assessment of the most likely tunnel support requirements.

5.5 The observational method

The observational method was formally introduced by Peck (1969) and was developed in response to the need to avoid highly conservative design assumptions when faced with unavoidable uncertainties in ground conditions. Instead of relying on one single solution that is fully developed before the construction work starts, monitoring and follow up of the actual conditions can be used to modify and optimise the design. The principle of the observational method is outlined in Figure 5.8.

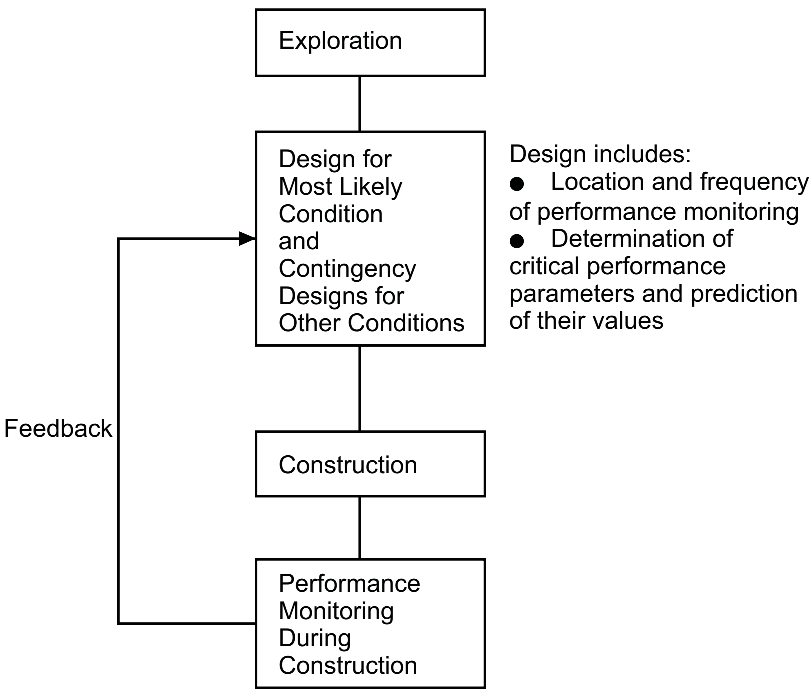

Figure 5.8 The principle of the observational method, in which design parameters are updated continuously through monitoring and feedback (Einstein and Baecher, 1982).

Peck (1969) stated a number of conditions to fulfil the implementation of the observational 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 design to suit actual conditions.*

When applying the observational method in accordance with the description given by Eurocode (EN-1997-1), the following requirements should be met before construction starts:

- *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 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.*

The observational method gives a potential for saving time and money with an assurance of safety. It can even lead to an increased safety since the method requires a focus on good communication, planned procedures, control during construction and a possible implementation of contingency measures (Powderham, 1994).

According to Schubert (2010a), there are two approaches for adopting the observational method, which may benefit engineering design:

(i) The initial design is based on less conservative parameters, such as 'most probable' or 'moderately conservative' conditions. Various contingency measures are prepared before construction commences and are implemented if observed behaviours exceed critical limits.

(ii) The initial design is based on a conservative set of parameters. Observations during construction are used to actively optimise the design.

Both approaches offer potential for cost savings with a reasonable assurance of safety, although starting with a more optimistic design and then changing the design if adverse circumstances occur may create uncomfortably low safety margins (Powderham, 1998).

Peck (1969) mentions that there are conditions under which the observational method cannot or should not be used. One prerequisite is that the design should be possible to modify as the construction progresses. If the character of the project is such that the design cannot be changed during construction, the observational method is not applicable. Moreover, Nicholson et al. (1999) mention that implementation of the observational method requires more resources than a conservative design approach, particularly during construction, when more effort is devoted to monitoring and design evaluations. They argue that the observational method should not be used in situations where a conservative design would imply a lower cost, such as in homogeneous rock conditions where the difference between the most probable condition and the most unfavourable condition is small.

Peck (1969) described that complications in the contractual relations are introduced when the design must be possible to alter during the construction. The main reason for this is that the final design is not fully determined in advance, this making it difficult to estimate the extent and costs of the project. Even though the courses of actions for different behaviours are planned in advance it is not possible to plan the extent of their use. An application of the observational method demands an increased flexibility of collaboration between client, consultant and contractor, and also a more distinct risk-allocation between client and contractor (Kadefors and Bröchner, 2008). It is important that the forms for economic compensation are perceived as fair and reasonable, and that good collaborations are being rewarded.

The observational method in a Swedish context

Kadefors and Bröchner (2008) state that excavation methods and construction designs are normally rather predictable in Swedish tunnelling projects. The largest uncertainties are related to the extent of the different rock qualities and how the rock mass respond to grouting and rock support. The observational method can help the design of these from becoming overdesigned as it provides a more effective use of the resources, adjusted to the actual need. It should though be noted that observations could lead to both a decrease and an increase of resources (Kadefors and Bröchner, 2008).

Further on, Kadefors and Bröchner (2008) identified two aspects being the most important for the implementation of the observational method in Swedish conditions; (i) there must be contingency measures prepared for all rock conditions possibly encountered during the construction and (ii) the organization and everyone involved in the project must be able to manage a continuous modification of the design. A perquisite for this is that those making contractual decisions, as well as those dealing with technical issues, are familiar with the observational method and the advantages it may give in the project in question.

The application of the observational method is similar to a design approach known as 'Active Design', which was introduced by Stille (1986), see Figure 5.9. Both approaches include establishing a preliminary design, determine contingency measures if the actual behaviour deviates from the expected, select relevant parameters to observe during construction, and modify the design to suit the actual conditions. The main difference between them is the strict requirement of preparing all contingency plans in advance when using the observational method, which is not as clearly expressed in Active Design (Holmberg and Stille, 2007).

There are according to Holmberg and Stille (2007) good possibilities for incorporating the observational method into the framework currently used for design practices in Sweden. The observational design strategy is not a substitute for the design process which is normally executed in a tunnel project, and it is not a substitute to empirical knowledge. It provides an addition that allows for a refined and more optimal construction, based on actual site conditions rather than assumed (Stille and Holmberg, 2010).

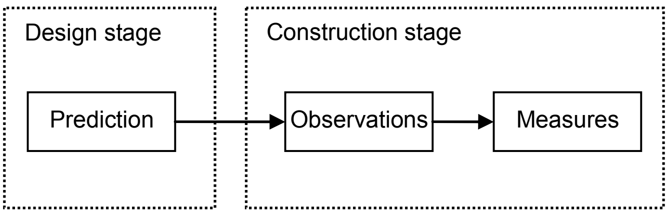

Figure 5.9 The Active Design process (modified from Stille, 1986)

A traditional design usually carries out monitoring and follow-ups to enable a verification of the tunnel design and confirm the tunnel stability and safety. The difference in the observational method is that monitoring and follow-ups are performed to actively update and/or revise the design depending on the outcome of the conducted testing and measuring, see Figure 5.10. For the preliminary design, this mainly implies that the design must be considered flexible and that modifications during constructions are prepared for, both in design documents and in the organizational and contractual framework.

Observation parameters

An important part of the effective implementation of the observational method is the selection of representative observation parameters, parameters which are both possible to predict and monitor. An appropriate observation parameter (or trigger value, design threshold or control parameter) should according to Powderham (1994) be comprehensive, reliable, repeatable and simple. Control parameters should yield relevant answers concerning the acceptable behaviour of the construction and if they are irrelevant and should be stopped monitored (Holmberg and Stille, 2007). The results from the measurements, in terms of assessment and feedback, must be given in time in order to confirm predictions or provide warnings of critical trends and make it possible to implement contingency measures in time (Powderham, 1994).

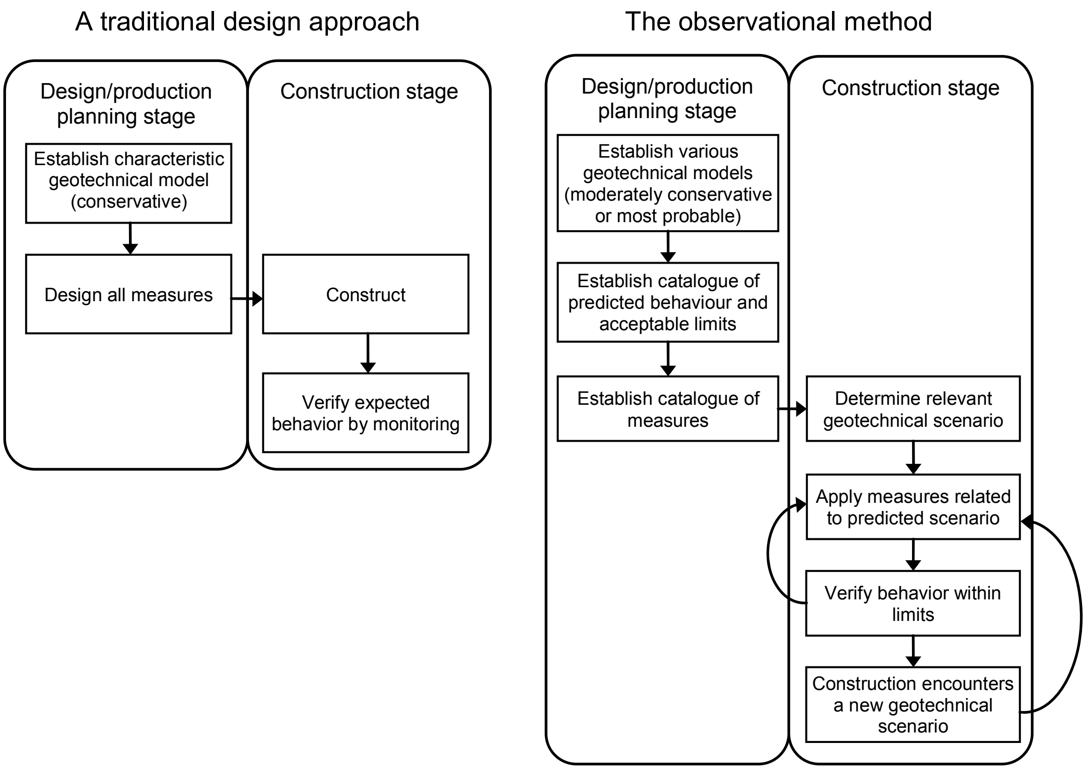

Figure 5.10 The traditional design approach in comparison with the observational method (modified from Schubert, 2010a)

The selection of relevant parameters for observing the behaviour and quantify its limits is not trivial. The parameters which directly observe the sought behaviour may due to technical constraints or other limits be difficult or not possible to measure and quantify (Holmberg and Stille 2007). The groundwater infiltration, the stability and the influence of the tunnel on the environment are all important aspects of tunnelling which normally are not readily available for measurement during the construction phase (Gustafson et al., 2010).

Combined measurements, indirect information and conceptualisation can then be used to provide approximate value of the feature of interest. Proxy parameters are measurable descriptors which stand in for desired (but unobservable) parameters, known as the target parameters. Each proxy parameter is associated with a rule or rules describing how to perform a transformation from proxy to target. The parameter *Q/dh* evaluated in boreholes ahead of the tunnel front is an example of a proxy for water inflow (Gustafson et al., 2010).

Underground excavations are associated with uncertainties and when the design of the unexcavated part is updated based on experience from the excavated parts some of the design uncertainties are reduced, but not all. There are e.g. uncertainties related to the predictive models and the measurements for verifying the behaviour, as well as those inherent in the rock mass properties and in the execution of the construction work (Olsson and Stille, 2002).

If the observational method is linked to probabilistic methods it will be possible to assess probabilities related to these uncertainties, e.g. the variance of design parameters. The updating process can e.g. be analysed using Bayesian statistics, which then provide a formal basis for combining available information with new data or knowledge. Control parameters can be assumed to be stochastic variables that can be statistically described and modelled (Stille and Holmberg, 2010). Some of the probabilistic tools available for these evaluations are briefly described in Chapter 4. Stille and Holmberg (2010) conclude that the use of probability tools within the framework of the observational method needs additional studies before they can be fully practised in underground design.

5.6 Concluding remarks

Some remarks on the literature review on design in rock engineering are:

- Rock engineering design is an iterative process consisting of a preliminary design prepared before construction and a final design executed during production.
- The preliminary design may be structured into estimating ground conditions (characterisation) and ground behaviour without influence of support, followed by

evaluating the system behaviour, i.e. interaction between alternative design measures and the ground behaviour. The various design alternatives are summarized in design classes which are used during the final design to suit varying ground conditions and requirements.

- Eurocode 7 presents four design approaches. The empirical rock mass classification systems belong to the category of design by prescriptive measures, i.e. design based on experience from similar cases. Frequently used classification systems in rock engineering are the RMR-system, the Q-method, GSI, and the RMi-system. However, their use is disputed. Commonly discussed shortcomings are that they simplify complex problems and are used for applications outside their original intent. Their best use is considered to be during the early project phases for construction planning and for providing checklists for collecting data.
- Another design approach often discussed for rock engineering is the observational method, which formalises the use of observations carried out during construction to actively adapt the final design to suit actual site conditions. The approach can facilitate the adaption of design solutions to the ground conditions encountered during construction, although more resources are needed during construction to carry out and evaluate observations.
- The selection of relevant parameters for observations is not trivial. Many ground behaviours, such as stability, groundwater inflow and environmental impact, is not readily measured during construction. Combined information and indirect information in the form of proxy parameters are used as stand ins for the target behaviour. Evaluation of the uncertainties and predictive abilities of proxy parameters are important.

6 THE OBSERVATIONAL METHOD IN ROCK SUPPORT AND GROUTING

The following sections describe the general aspects of two specific engineering issues; rock support and grouting. This chapter shortly describes how they are designed and how an observational approach is suggested to be applied to their design.

6.1 Rock mechanical design in tunnels

The rock mechanical design of an underground opening is carried out to ensure a stable and safe facility in compliance with national norms or building codes. The design should also be cost effective and make use of an optimal amount of rock support. The tunnel size and location is normally given by the requirements related to the function of the facility. Consequently are tunnel shapes, excavation methods and support measures the aspects which are normally possible to optimize, e.g. to suit the current stress situation (Lindblom, 2010).

The rock mechanical design focuses on the structural behaviour of the underground opening, with estimations of the load effects (normally the rock stress) and the load carrying capacity (strength of the rock material). The design situation for the structural capacity in an underground project is similar to other structural design situations, hence it must be established that the bearing capacity is higher that the load effect to a certain degree (Stille et al., 2003). However, the rock surrounding an underground opening is generally a complex building material and the mechanical system in rock can be quite complicated. The behaviour of the load carrying system in rock is a result from an interaction between the rock mass and the supporting elements, thus both are carrying a part of the total loading effect (Holmberg, 2005). For a safe and cost-effective design it is important to define how the bearing capacity can be described and how loads are distributed in the structural system.

The most common methods used for stabilising an underground opening in hard rock are rock bolting and shotcrete lining. There are a number of different design tools available for the design of rock support (empirical methods, analytical methods, numerical modelling, or observational methods). The analysis of the required support in an underground opening should though typically consider (SKB, 2007);

- the rock mass (*in situ* stresses, rock mass strength, possible failure modes);
- the geometry (shape/size of the opening, surrounding structures, rock cover);
- the construction method (damages, loadings).

The *in situ* stress acts as a load upon the rock mass surrounding underground opening and has a major impact on the stability. It is important to consider the magnitude and directions of the *in situ* stresses as these determines the failure processes likely to occur (Martin et al., 2003).

The strength and deformability of a rock mass determines the capacity of the rock mass. It determines if the load changes introduced by excavation will lead to failure and stability problems or not. The rock mass strength and deformability characteristics are determined by the intact rock and the discontinuities within the rock mass. The strength of intact rock mass and the shear strength of discontinuities can be assessed separately with standardised laboratory testing, but the strength of the rock mass is not easily determined. The scale of the rock mass restricts the possibilities of physical testing, and the complex interaction between intact rock and joints makes it difficult to combine the results from small-scale testing of the separated units (Edelbro et al., 2007).

The most common way to describe the rock mass strength is by using empirical methods, which rely on the existing correlation between measurable rock mass properties and the rock mass strength. Examples of such methods are empirical failure criteria or empirical classification systems. Other methods available are back analysis of existing failures large-scale testing, and mathematical modelling.

When the in situ stresses and the rock mass strength are determined the anticipated failure mechanism can be evaluated. Knowledge of the likely failure mechanism is a helpful tool when choosing appropriate strategies for support design and the excavation method (Martin et al., 2003). Martin et al. (2003) identifies two types of instability usually observed around underground openings in hard rock; (i) *structurally controlled gravity-driven processes leading to wedge type falls-of-ground* and (ii) *stress-induced failure or yielding*. Wedges falling or sliding from the roofs and sidewalls of tunnels are common when the confining stress is low, e.g. at shallow depths or at tunnel intersections. The critical parameters for brittle failures are stress and block size. Stress-induced failure occurs when stress magnitudes reach the rock mass strength, and the resulting yielding may create large convergence displacements. Palmström and Stille (2007) added a third category; the groundwater initiated failures, and presented a summary of behaviour types, presented in Table 6.1 and Figure 6.1.

Table 6.1 A summary of behaviour types in underground excavations. Modified from Palmström and Stille (2007).

FAILURE MODE GROUP	BEHAVIOUR TYPE	
Gravity driven	a. Stable	
	b. Block fall(s)	
	– of single blocks	
	– of several blocks	
	c. Cave-in	
	d. Running ground	
Stress induced	e. Buckling	
	f. Rupturing from stresses	
	g. Slabbing	Brittle behaviour
	h. Rock burst	
	i. Plastic behaviour (initial)	
	j. Squeezing	Plastic behaviour
Water influenced	k. Ravelling from slaking	Hydratization
	l. Swelling	
	– of certain rocks	Swelling minerals
	– of certain clay seams or fillings	
	m. Flowing ground	
	n. Water ingress	Flowing water

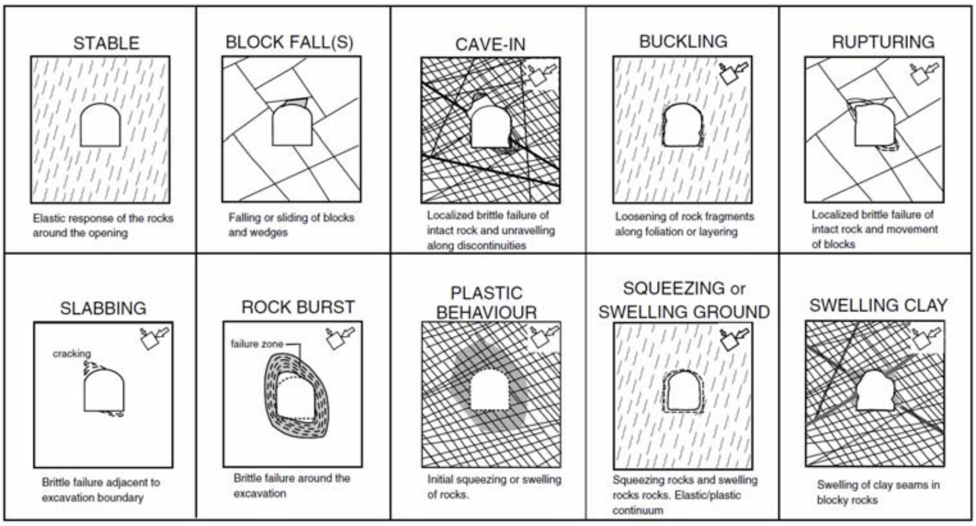

Figure 6.1 Behaviour types in underground openings, from Palmström and Stille (2007)

6.2 The observational method in rock mechanical design

There are a number of uncertainties inherent in rock mechanical design, such as uncertainties in the understanding of the ground support interaction and the lack of knowledge of the true mechanical properties and the true system behaviour (Stille et al., 2003). In addition, a number of simplified models and empirical approaches are used, which add to further uncertainty. The approach of dealing with uncertainties by observing the actual system behaviour has therefore been an important element in structural engineering for many years.

The system behaviour of interest in a rock mechanical design is the structural behaviour of the rock mass and the rock support. Holmberg and Stille (2009) suggests that the observations carried out within the framework of the observational method should "*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 acceptable limits of behaviour*". Hence the chosen control parameters can be related to (i) rock mass qualities, (ii) behaviour of the structural system or (iii) conducted support measures. Further description of what this could imply is presented below and in Table 6.2 (Stille and Holmberg, 2010):

(i) The first example deals with establishing the real conditions (rock classes) in the tunnel by a geological follow-up of indicators describing the rock mass quality. This category also includes the verification of rock support measures (support class) associated with each rock class.

(ii) The second example can relate to the interpretation of results from deformation monitoring, where the structural behaviour of the tunnel is assessed relative to the design criterion.

(iii) The third example deals with assessing the quality of executed support measures, e.g. by measuring the thickness of shotcrete in place.

In many cases the deformation constitutes a robust parameter for assessing the structural behaviour as it can be quantified and monitored during construction (Holmberg and Stille, 2009). The acceptable limit of behaviour can be defined as a deterministic value of deformation, governed by the deformational capacity of the rock support. The range of possible deformational behaviours can be described with probability distributions, and statistical evaluation of the data is made to decide whether there is an acceptable probability that the final deformation will be within acceptable limits. A detailed description of these issues and the way of defining reference parameters and predicting behaviours is given in Holmberg and Stille (2009).

Table 6.2 Interpretation of the terminology in Eurocode and its application to rock mechanical aspects (modified from Stille and Holmberg, 2010)

TERMINOLOGY	EXAMPLE 1A CLASSIFICATION	EXAMPLE 1B EMPIRICAL DESIGN	EXAMPLE 2 TUNNEL BEHAVIOUR	EXAMPLE 3 SHOTCRETE QUALITY
Acceptable limits	Predefined limits of each rock class based on indicators	No damage of the installed support	The monitored deformation should be smaller than a given value (design criterion)	Mean value of thickness of shotcrete for a given test procedure
Possible behaviour	All rock classes	Both damaged and undamaged support	Range of deformations based on evaluating the variability in ground conditions	Variation in thickness for a given shotcrete application
Monitoring program	Observation of indicators	Visual inspection of damage	Deformation monitoring	Measurement of shotcrete thickness
Contingency actions	Alteration within predefined rock classes	Install additional support, modify classification rules or support measure	Install additional support, shotcrete and rock bolts	Spraying more shotcrete

6.3 Grouting design

A number of difficulties in underground rock construction are caused by the occurrence of groundwater. High water pressures and inflow of water may affect the construction and operation of the tunnel and lowering the water table could have a significant impact on the surrounding environment. Design and implementation of water-mitigation measures to reduce the groundwater inflow, such as grouting or lining, are therefore an important part of underground construction.

Pre-excavation grouting in conjunction with construction is considered the most cost-effective means of controlling groundwater inflow into tunnels constructed in fractured hard rock (Gustafson, 2009). Pre-grouting (Figure 6.2) implies that grout is pumped into boreholes drilled ahead of the excavation. The grout spreads into fractures intersecting the boreholes and the excavation can proceed through a zone of sealed rock mass where the water flow is reduced significantly.

The need for water-mitigating measures is founded on specified construction requirements. These requirements may relate to the functionality of the facility, the working environment during construction or the environmental impact on the surroundings (Eriksson and Stille, 2005). The stipulated requirements are also influenced by aspects related to maintenance or the practicality and productivity of the design. Identification of key issues and their underlying processes facilitates the specification of engineering requirements and preferences (Andersson et al., 2000; Gustafson, 2009). Examples of key issues related to grouting are presented in Table 6.3.

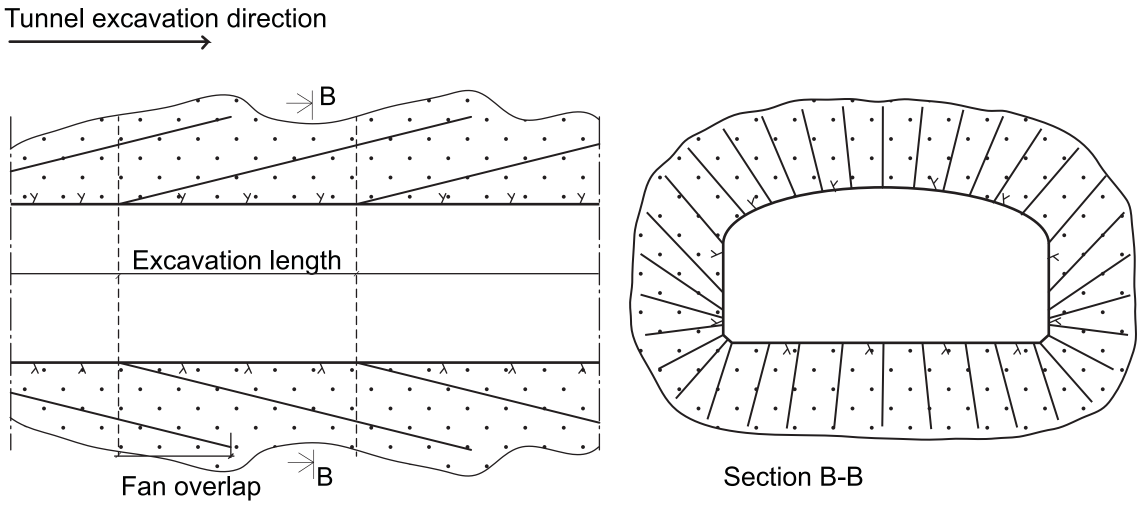

Figure 6.2 A pre-grouting design layout in profile (left) and section (right). The dotted area represents a theoretically grouted zone.

Table 6.3 Example of a list of key questions with a hydrogeological focus for underground construction. Based on Gustafson (2009).

CONSTRUCTABILITY	INTERNAL ENVIRONMENT	EFFECTS ON SURROUNDINGS	DURABILITY
Understanding of the geological and geomorphological history Occurrence and flow of groundwater Rock mass stability Fracture system character Highly fractured/crushed zones Grout properties Observable parameters/ control programme	Working environment and water problems Water-soluble gases Specification regarding dripping and moisture in completed tunnels	Groundwater drawdown around the facility Spread and migration of grouting agents and contaminants Salt water intrusion and other water chemical effects Removal of process water and inflowing groundwater	Durability of sealing agent, shotcrete and bolts Corrosion and groundwater quality Groundwater issues during operation and maintenance (infiltration)

Requirements related to hydrogeology in tunnels are generally summarised into a permissible inflow into the facility. Restrictions on water inflow into tunnels are generally very strict in urban areas since lowering of groundwater pressures can lead to settlement problems in overlying soft soils. The inflow limit is translated into required permeability of the grouted zone, which is a quantity that forms a basis for design parameters, such as grout material, fan geometry, injection pressure and stop criteria, see e.g. Eriksson and Stille (2005). Research in recent decades has led to an increased understanding of mechanisms behind the spread of grout, with the introduction of theoretical analyses to complement personal experiences (Gustafson and Stille, 2005). The grouting technique, however, must be adapted to prevailing conditions at the site.

The majority of the groundwater transport in crystalline rock occurs adjacent to faults and along extensive open fractures (Goodman, 1993). The hydraulic properties of the rock mass are consequently governed by the water-conductive fracture system. As water behaves differently in different fracture systems, the characteristics of the fracture system also have implications for the grouting. Hernqvist et al. (2012) emphasise the importance of creating a conceptual model of the water-conductive fracture system to which the grouting design will be adapted. They identify a set of parameters that are useful for describing a fracture system for grouting-related purposes: hydraulic head, hydraulic aperture, fracture frequency, orientation and number of fracture sets and flow dimension. These are further described in Table 6.4. These parameters provide information on fractures that need to be sealed and input for choosing grouting technique.

The selection of grouting technique involves determining the grouting material, the grouting pressure, the fan geometry (grout-hole lengths, number of holes and spacing), and the stop criterion. Inflow requirements and the calculated hydraulic conductivity of the rock mass commonly form a basis for grouting design in Swedish tunnel projects. These enable an assessment to be made of the required sealing efficiency and the hydraulic conductivity of the grouted zone, which could indicate the complexity and degree of difficulty of grouting work (Eriksson and Stille, 2005). This in turn affects the requirements in design analyses and investigations as well as the number of grouting designs to apply in the project.

Gustafson et al. (2004) describe an analysis process for preliminary grouting design that focuses on individual fractures and the smallest hydraulic aperture that needs to be sealed to fulfil the inflow requirement, see Figure 6.3. Based on the distribution of fracture apertures and the required sealing efficiency, it is possible to choose a suitable

Table 6.4 Parameters of importance for conceptualise the water-bearing fracture system (from Hernqvist, 2011)

PARAMETERS	RELEVANCE FOR GROUTING
Hydraulic head, h	h is necessary for calculating the hydraulic aperture from hydraulic tests, and for deciding the grouting pressure.
Hydraulic aperture, b	The penetrability of a grout is limited by fracture aperture; hence b is important input data for grout choice.
Fracture frequency, P_{10}	A change in P_{10} indicates a change in the rock mass, which should be grouted differently than the surrounding rock mass.
Orientation and number of fracture sets	The number of water-bearing fracture sets determines the connectivity of the fracture system. Borehole geometry should be chosen such that the probability to intersect fractures is high.
Flow dimension, D_q	Flow dimension is important for the behaviour of both water leakage and grout spread. Fractures with 2D flow are possible to seal, while 1D flow channels are very difficult to grout.

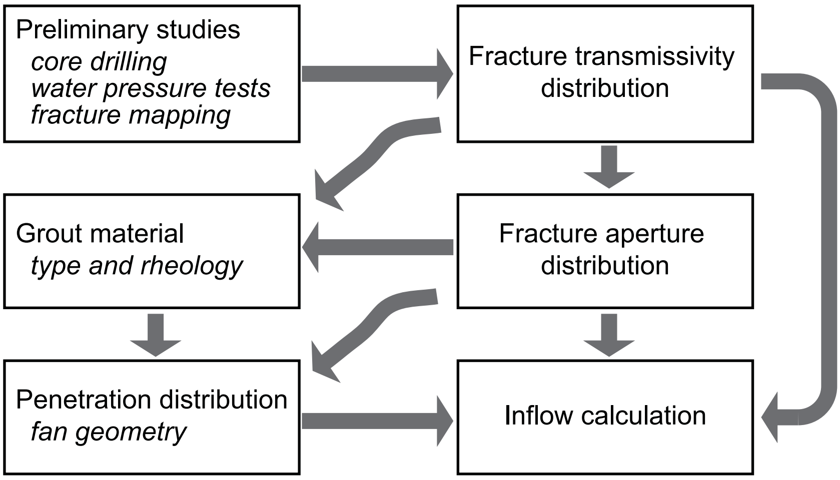

Figure 6.3 Design concept for grouting. Modified from Gustafson et al. (2004)

grouting material and a subsequent fan layout to achieve sufficient grout penetration. The four analysis stages are: fracture transmissivity distribution; fracture aperture distribution; distribution of grout penetration lengths; and calculation of the resulting tunnel inflow for comparison with requirements (Gustafson, 2009).

6.4 The observational method in grouting design

Emmelin et al. (2007) performed a study where the design process presented by Goricki et al. (2004) was adapted to grouting design according to the observational method. The result from the study is presented in Table 6.5. The ground behaviour was presented as the amount of water inflow into the tunnel before grouting and the system behaviour as the amount of water inflow after grouting. The relationship between the behaviour before and after grouting cannot be fully known before construction, and the observational method was considered suitable for reducing the uncertainties.

Table 6.5 The actual behaviour checked against the predicted behaviour, modified after Emmelin et al. (2007)

PHASE	PREDICTION TO BE VERIFIED	REQUIREMENT	OBSERVATION, CRITERIA	ACTION
Before grouting	Ground behaviour: ungrouted rock mass conditions	Current values within indicator value limits for the predicted class	Hydraulic and fracture data collected in probe holes	Assessment or change of grouting class
During grouting	System behaviour: the performance of the grout in the rock fractures	Specification on pressure, flow, volume	Pressure, flow, volume; backflow	Adjust grouting measures within class
After grouting, before excavation	System behaviour: the tightness of the tunnel to be excavated	Tightness in grouted zone	Water loss in control holes	Re-grouting
After excavation	System behaviour: the inflow to the excavated tunnel	Inflow to tunnel section	Inflow in weir	Post-grouting

The design of water-mitigating measures in an observational context starts with assessing possible ground behaviours and dividing the ground conditions into rock classes. Appropriate sealing measures or grouting classes will then be adapted to these, with predictions of their sealing result, i.e. the system behaviour (Emmelin et al., 2007). Examples of different types of sealing measures are 'selective pre-grouting' and 'systematic pre-grouting'. The choice between the different grouting classes is based on observable parameters for the ground behaviour, e.g. hydraulic tests in boreholes.

There must also be verification during construction whether the actual system behaviour is within acceptable limits, which implies that there must be a plan for verifying that the grouting results are satisfying and that the requirements have been met. However, the verification of the grouting results with observations of water inflow after grouting is less suitable as it means that the result can only be verified once the tunnel is finished and weirs are installed (Emmelin et al., 2007). Hernqvist et al. (2013) and Gustafson et al. (2010) suggest grouting decision methods where the inflow to probe-holes during tunnelling and the specific capacity for a borehole, *Q/dh*, can be used as a proxy for estimating the actual inflow. If the evaluation indicates that the requirements have not been met another grouting round or some other measures will be necessary.

6.5 Concluding remarks

Some concluding remarks on the adoption of the observational method within rock mechanical design and grouting design can be made:

- Uncertainty inherent in rock mechanical design includes uncertainties in the understanding of the ground support interaction and the lack of knowledge of the true mechanical properties and the true system behaviour.
- The adoption of the observational design within rock mechanical design could focus on controlling parameters that relate to (i) the rock mass qualities, (ii) the behaviour of the structural system of the tunnel or (iii) the quality of executed support measures. The acceptable limit of behaviour can be defined with deterministic values or probability distributions.
- The relationship between the amount of water inflow into the tunnel before grouting and after grouting is difficult to predict, and the observational method is considered as a suitable method for reducing uncertainties and allow for a more effective use of grouting resources.
- Verification during construction whether the actual system behaviour (e.g. final inflow) is within acceptable limits cannot be made during the excavation. Grouting decision methods using data from hydraulic tests in probe-holes is suggested to provide proxies for estimating the actual inflow.

7 CONCLUSIONS

This literature review has focused on describing the use of engineering geological information in underground rock construction, as well as the consequences of using empirical design methods and the observational method in rock engineering.

It is found that conceptual models are useful to qualitatively establish the significant processes and parameters which affect the underground engineering work. The conceptual models should be continually updated throughout the project and can reduce uncertainties as they increase the understanding of essential aspects and highlights data gaps. Geological parameters considered relevant may differ for different rock engineering issues and it is deemed appropriate to use separate models and different data collection strategies for the different design aspects.

The use of empirical rock mass classification systems is widespread within rock engineering. There are, however, shortcomings in their use that are not always considered. One of the main concerns is that they simplifies and homogeneities complex ground behaviours and are used in areas outside their original intent. However, classification systems can be useful in rock engineering, especially for early estimates and providing good checklists for collecting rock mass data.

The observational method offers a practical approach to reduce risks in the design of tunnels, and it is stated that the approach fits well within the current practises and procedures for design. Some of the key concepts that have to be dealt with are the geotechnical models, prediction of behaviours, contingency measures, determining acceptable limits and verification of behaviours. However, a number of raised issues or questions concerning the observational design approach are:

- The description of expected behaviour may need to be defined more precisely. How to define the range of possible behaviour and how to perform the verification of the actual conditions during construction need more consideration.
- The use of probabilistic and statistical methods as a part of the observational method needs further development. This includes updating and dealing with observational parameters, which should be used during construction to update models and to make quantitative back-analyses of the geotechnical behaviour.

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