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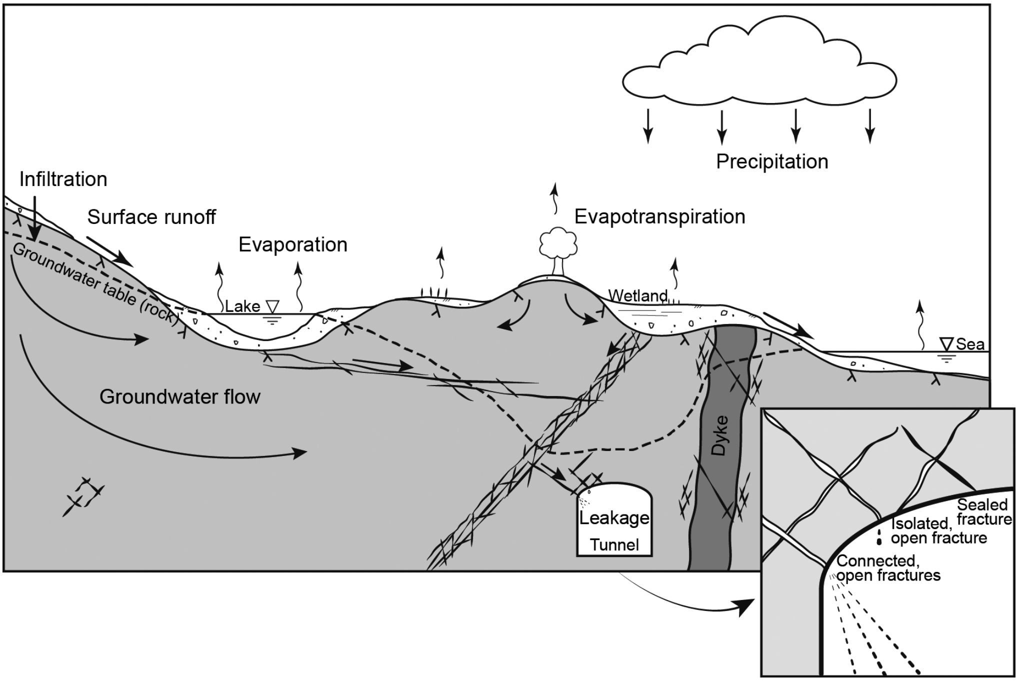

ON THE USE OF ENGINEERING GEOLOGICAL INFORMATION IN ROCK GROUTING DESIGN

Sara Kvartsberg

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Användning av ingenjörsgeologisk information i injekteringsdesign

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FÖRORD

I undermarksprojekt behöver vi kunna hantera inläckande vatten, speciellt i trafiktunnlar där isbildning vintertid kan orsaka stora problem. För att undvika förseningar, överskridna byggkostnader, säkerhetsproblem och stora underhållskostnader behöver tunneltätning utföras på ett fullgott sätt.

En förutsättning för att uppnå en fullgod tunneltätning genom injektering är att kunskapen om bergets och dess hydrauliska egenskaper är tillräcklig och anpassad till de ingenjörsmässiga behov som projektet har. För att lyckas med det behövs en strukturerad metodik för att tolka, beskriva och kommunicera geologisk och hydrogeologisk information såväl tidigt i processen som i själva byggskedet.

Detta forskningsarbete har behandlat några olika områden med målet att tydliggöra tillvägagångssätt och hjälpmedel för att ta fram och underhålla en injekteringsdesign genom projektet. Det rör hanteringen av information för att göra ingenjörsgeologiska prognoser och en utveckling av geohydrauliska domäner. Vidare att hantera och ansätta effektivitet av tätningsåtgärder och karaktärisera ”hydrauliska ledare” genom att använda transmissivitetsfördelning för sprickor. Ett annat område med koppling till de föregående är att använda så kallade injekteringsklasser (eng. grouting design classes) och en anpassning av dessa till en föränderlig geologi.

Föreliggande licentiatarbete utfördes av Sara Kvartsberg vid Institutionen för bygg- och miljöteknik på Chalmers Tekniska Högskola i Göteborg med Åsa Fransson som huvudhandledare. En referensgrupp har följt projektet och bidragit med värdefulla råd och diskussioner. Referensgruppen bestod av Carl-Henric Wahlgren från SGU, Robert Sturk från Skanska, Assen Simeonov från SKB och Per Tengborg från BeFo. Projektet har finansierats av BeFo och Formas som ett resultat av en gemensam utlysning.

Stockholm i oktober 2013

Per Tengborg

PREFACE

In underground projects the water leaking into a facility needs to be handled, especially in traffic tunnels where build-up of ice may cause large problems. To avoid delays, cost overruns, safety problems and large maintenance costs the tunnel sealing need to be performed adequately.

A prerequisite adequate sealing of tunnels by grouting is sufficient knowledge of the rock, its hydraulic properties as well as engineering demands for the project. To succeed with this, a structured methodology to interpret, describe and communicate geological and hydrogeological information is needed. This information needs to be available early in the process as well as during the actual construction works.

This research has dealt with some areas aiming at describing the approach and tools to make and maintain a grout design throughout a project. It covers handling of information to make engineering geological prognoses and development of geohydrological domains. Further, using distribution of fracture transmissivity this work is to handle and assess efficiency of sealing and characterization of hydraulic conductors. Related to this, so called grouting design classes were proposed for different types of geology.

The present licentiate thesis were performed by Sara Kvartsberg and supervised by Åsa Fransson, both at the Department of Civil and Environmental Engineering, Chalmers University of Technology in Gothenburg, Sweden. A reference group followed the project and contributed with valuable advice and discussions. This group consisted of Carl-Henric Wahlgren at Geological Survey of Sweden (SGU), Robert Sturk at Skanska, Assen Simeonov at Swedish Nuclear Fuel and Waste Management Co (SKB) and Per Tengborg at Rock Engineering Research Foundation (BeFo). The research project was financed by BeFo and The Swedish Research Council for Environment, Agricultural Sciences and Spatial Planning (Formas).

Stockholm in October 2013

Per Tengborg

SAMMANFATTNING

Effektivt bergbyggande är beroende av lämpliga bedömningar av platsspecifika geologiska förhållanden. Relevant geologisk information måste därför samlas in, tolkas och kommuniceras, vilket görs med hjälp av ingenjörsgeologiska prognoser. Ingenjörsgeologiska prognoser blir generellt mer användbara om de anpassas efter informationsbehovet hos konstruktionsåtgärderna som dimensioneras för den specifika berganläggningen. Arbetet som presenteras i denna rapport har som mål att förtydliga vad denna anpassning kan innebära för ingenjörsgeologiska prognoser som används vid projektering av tätningsåtgärder vid tunnelbyggnation.

En viktig del av arbetet har varit att identifiera geologisk information som kan öka förståelsen för bergmassans hydrauliska egenskaper. Detta gjordes genom att sammanställa relevanta geologiska parametrar samt introducera och demonstrera användningen av hydrauliska domäner för att definiera förväntade variationer i bergmassans hydrauliska beteende. De hydrauliska domänerna motsvarar geologiska typmiljöer med olika hydrogeologiska egenskaper och kan exempelvis definieras utifrån olika typer av bergarter och deformationszoner.

Arbetet med att beskriva bergets hydrauliska egenskaper för injektering har även omfattat att förutsäga bergmassans behov av injektering samt utvärdera tätningseffekten av olika åtgärder. För att göra detta användes bland annat en modell som simulerar transmissivitetsfördelningar för olika sektionslängder i tunnlar. Fördelningsmodellen var även användbar för att identifiera flödesegenskaper hos olika strukturer i bergmassan, men den uppvisade samtidigt begränsningar eftersom den enbart gav rimliga inflödesskattningar för en viss typ av vattenförande strukturer.

Nuvarande användning av geologisk information vid projektering av injekteringsåtgärder i svenska tunnelprojekt har även studerats. Studien visade att injekteringsklasser ofta används under byggfasen för att definiera olika injekteringsutföranden. Klasserna är i stor utsträckning definierade för att anpassas till varierande krav som ställs på tunneln, exempelvis täthetskrav, men i mindre utsträckning till olika förväntade bergbeteenden. Anpassning till olika förväntande bergbeteenden längs tunnelsträckningen är dock något som

förutsätts när design utförs enligt observationsmetoden. I detta arbete föreslås att hydrauliska domäner representerar de olika förväntade bergbeteendena, både fördelaktiga och ofördelaktiga, och att domänerna används tillsammans med täthetskrav för att definiera injekteringsklasser. Domänindelning ger en tydlig struktur för hanteringen av geologisk information och underlättar en identifiering av relevanta geologiska förutsättningar, vilket minskar risken för att träffa på oförutsedda bergförhållanden under byggfasen. Den del av den ingenjörsgeologiska prognosen som ska ligga till grund för dimensionering av tätningsåtgärder föreslås därför att baseras på en indelning i hydrauliska domäner redan i tidiga projektfaser.

Nyckelord: Sprickigt berg, Tunnlar, Injektering, Designklass, Geologisk prognos, Hydraulisk domän, Transmissivitetsfördelning, Observationsmetoden

SUMMARY

Underground rock construction is dependent on appropriate assessments of the geological settings at the project site. Relevant geological information must therefore be collected, interpreted and communicated clearly with the aid of engineering geological prognoses. The general quality of the geological prognoses can be improved if the information relates to the engineering application and the project requirements. This work aims to clarify ways in which this can be taken into account in grouting design throughout the construction process.

Attention has been given to geological information that can increase understanding of the hydraulic properties of the rock mass. This was done by compiling relevant geological parameters and introducing hydraulic domains to define the various expected forms of hydraulic behaviour at a project site. A fracture transmissivity distribution model was presented for evaluating the sealing efficiency of grouting measures. The distribution model was also found useful for identifying differing flow configurations in the rock mass and indicating limitations in analytical models, which could be valuable for grouting design and inflow predictions.

A study was also made of the current use of geological information in grouting design in Swedish tunnel projects. The study indicated that pre-defined grouting design classes are generally adapted to differing requirements but are less clearly modified in order to suit the variation in geological settings. Hydraulic domains are suggested to be used together with stated requirements to establish grouting classes adapted to both favourable and unfavourable scenarios, which is in accordance with the observational method. It is suggested that engineering geological prognoses based on hydraulic domains are made in the early phases of a project to facilitate identification of grouting design prerequisites and reduce the risk of encountering unforeseen ground conditions in later phases.

Keywords: Fractured rock, Tunnels, Grouting, Design class, Geological prognosis, Hydraulic domain, Transmissivity distribution, Observational method

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LIST OF NOTATIONS

Roman letters

A	[m^2]	Cross section area
b	[m]	Hydraulic aperture
d	[m]	Depth below reference surface
g	[m/s^2]	Gravitational acceleration
K	[m/s]	Hydraulic conductivity
Q	[m^3/s]	Flow
T	[m^2/s]	Transmissivity

Greek letters

μ	[Pa·s]	Fluid viscosity
ρ	[kg/m^3]	Fluid density

Mathematical expressions

dh	[m]	Hydraulic head
dh/dl	[m/m]	Hydraulic gradient
Q/dh	[m^2/s]	Specific capacity

Abbreviations

GIS	Geographical Information System
GSI	Geological Strength Index
HCD	Hydraulic Conductor Domain
HPF	High Permeability Feature
HRD	Hydraulic Rock Domain
ISRM	International Society for Rock Mechanics
NRC	National Research Council
RMi	Rock Mass index
RMR	Rock Mass Rating
SKB	Svensk Kärnbränslehantering AB
STA	Swedish Transport Administration, Trafikverket
TBM	Tunnel Boring Machine
VOIA	Value of Information Analysis
WPT	Water Pressure Test
Äspö HRL	Äspö Hard Rock Laboratory

1 INTRODUCTION

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 (Dalmalm, 2004).

Grouting design involves making an appropriate choice of grouting material and grouting technique based on knowledge of the hydraulic properties of the fractured rock mass (Gustafson, 2012). Relevant geological information should therefore be collected, interpreted and communicated clearly with the aid of engineering geological prognoses. The prognoses are useful in the early project phases to identify critical design applications and reduce the risk of encountering unforeseen ground conditions in later project phases, which could lead to cost or time overruns, see e.g. Sturk (1998), Baynes et al. (2005), and Lundman (2011). Continued development of established prognoses throughout the project is also important in order to develop optimised designs that are implemented appropriately during construction. However, uncertainties and inadequacies in geological prognoses could lead to delays and raised budgets (Lundman, 2011). Unreliable grouting prognoses can be caused, for instance, by the use of empirical rock mass classification systems that were originally developed for rock support design (Palmström and Broch, 2006). Another issue is the scarcity of project-specific data, which could lead to grouting designs being copied between tunnel projects without acknowledging that each project has its own key issues (Palmström and Stille, 2010).

The quality of the geological prognoses is generally improved if the geological information relates to the engineering application and to the project requirements (Gustafson 2012; Baynes et al., 2005). What this would mean in practice for handling information as part of the grouting design process is not stated clearly and it would be useful to convert general suggestions into practical advice. It would seem useful, therefore, to clarify and summarise how engineering geological information is handled within the grouting design process and to identify areas where improvements could be made.

1.1 Objectives

The general aim the study presented in this report has been to clarify the handling of engineering geological information in grouting design for underground rock construction projects. Attention has therefore been given to conceptualisation, characterisation and classification of geological settings throughout the construction process. In addition to the overall aim of the project, there are a number of specific objectives.

- Analyse the current use of geological information in grouting design and list the geological parameters and processes that are considered important.
- Present an approach to constructing engineering geological prognoses for grouting design purposes in the early phases of a project.
- Demonstrate how fracture transmissivity distribution models can be of use for predicting and assessing the sealing efficiency of grouting measures but also for identifying differing flow configurations within the rock mass.
- Suggest improvements in grouting design classification by dealing with the way geological settings are defined, described and quantified.

1.2 Scope of work

The work has included literature reviews, studies of grouting designs for some selected Swedish tunnel projects and two case studies. Chapter 2 to Chapter 5 comprise the literature review covering the construction process, engineering geological prognoses, hydraulic properties of fractured rock and grouting design. Chapter 5 also includes a study of the use of grouting design classes in Swedish tunnel projects. Chapter 6 summarises the results of the two case studies. Discussions and conclusions are presented in Chapter 7 and Chapter 8, respectively.

The work has concentrated on ground conditions, guidelines and construction methods normally encountered in underground construction in Sweden. The design application in focus is water-mitigating measures, mainly as pre-excavation grouting. A complete account of the factors needed to develop appropriate engineering geological prognoses for all engineering applications in a tunnel project falls outside the scope of this study.

Consideration has been given to the input information, the scale of interest and the design focus in different phases during construction. Figure 1.1 shows an overview of the conducted work and how the case studies fit into this framework.

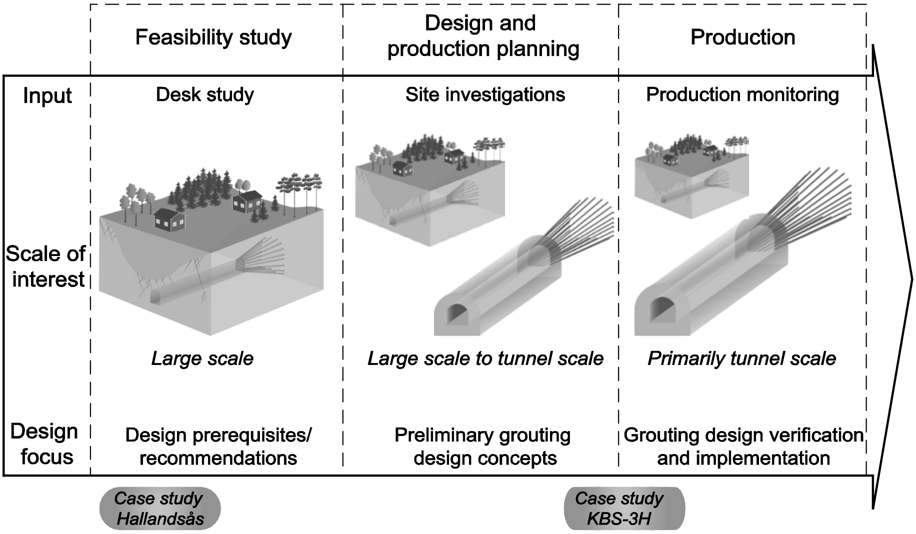


Figure 1.1 A schematic description of the framework for this study.

2 DESIGN AND CONSTRUCTION OF UNDERGROUND FACILITIES

2.1 The construction process

The construction process for an underground facility starts with an idea developed from an identified need and finishes when the facility is brought into operation. 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

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.

2.1.1 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. 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 ground conditions that could make the project unfeasible, i.e., recognising stop signs (Sturk, 1998).

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.

2.1.2 Design and 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 conditions 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 conditions, 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 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 two important documents that need to be prepared during the design phase: the engineering geological prognosis and the construction plan (STA, 2011b). The engineering geological prognosis provides a prediction of expected ground conditions at the site that are of technical or financial significance to the project (Holmberg and Stille, 2007). The construction plan contains drawings and technical descriptions that provide a detailed description of technical solutions for the planned construction concept.

2.1.3 Production 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).

2.2 Design methods

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, 2012).

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 and Palmström, 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.

A basic structure of the preliminary design process is described by Goricki et al. (2004), see Figure 2.1. 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

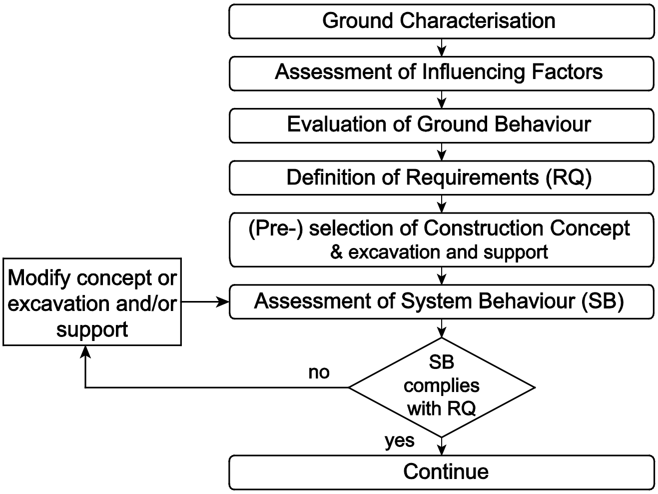


Figure 2.1 A simplified version of the basic structure of the preliminary design process, modified from Schubert (2010b).

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).

The preliminary design process emphasises that design should be based on actual ground behaviour and project-specific issues rather than using standardised design solutions. 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.

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

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 3.3), 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), see principles in Figure 2.2. The observational method was introduced formally 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.

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

a) 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.

b) 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). 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.

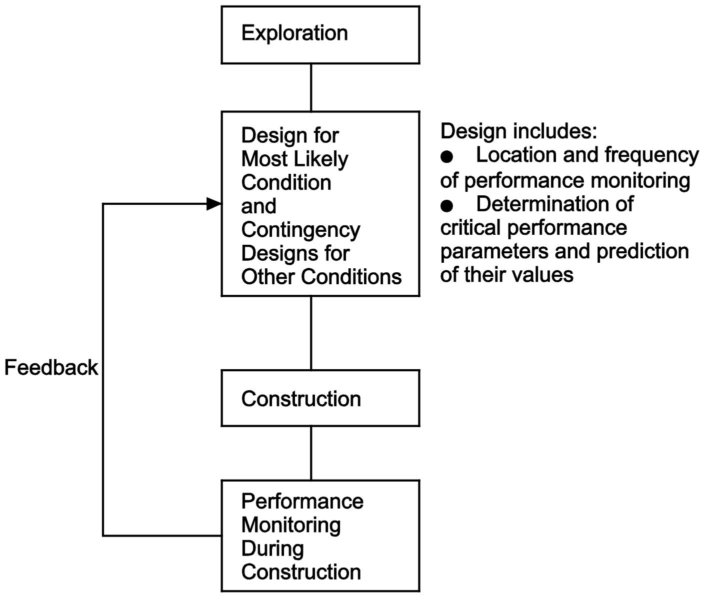


Figure 2.2 The principle of the observational method, in which design parameters are updated continuously through monitoring and feedback (Einstein and Baecher, 1982).

3 ENGINEERING GEOLOGICAL PROGNOSES

The variability and complex behaviour of geological materials imply that it is not possible to obtain complete knowledge of actual ground conditions before construction (Hammah and Curran, 2009; Sturk, 1998). Site investigation techniques can only explore a very small volume of rock mass involved in the project and information gathered may be difficult to extrapolate to other areas (Hudson, 1993). The cost of investigating the ground must be balanced against the benefit of the information obtained, which is linked to the risk of having a design that fails to satisfy the requirements (Einstein, 1996). The risk of choosing an insufficient design alternative can be reduced by increasing the knowledge of ground conditions to a level where unforeseen ground conditions are unlikely to be encountered (Baynes et al., 2005). The collection and interpretation of relevant geological data is therefore an essential part of the construction process.

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). The quality of geological prognoses is generally improved if the information is related to engineering applications, if uncertainties are quantified and if the information is communicated effectively (Sturk, 1998). Some aspects of establishing geological prognoses will be described in this chapter although for detailed information and Swedish guidelines the reader is referred to Bergman and Carlsson (1986) and STA (2011b).

3.1 Engineering geological models

A model can be defined as "*an approximation of reality created for the purpose of solving a problem*" (Griffiths, 2012). The set of assumptions that describe the key processes qualitatively and establish the geometric framework for the model is called the conceptual model, whereas application of the conceptual model, with data inserted into calculation tools, is referred to as model realisation (Gustafson and Olsson, 1993; Gustafson, 2012), see Figure 3.1. The conceptual model is important as it provides structure and identifies relevant aspects of the model.

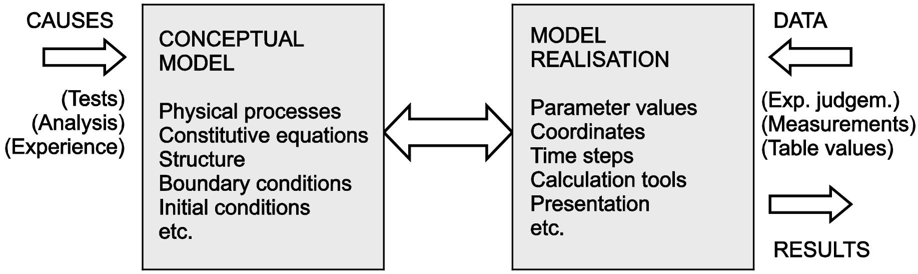


Figure 3.1 The relationship between the conceptual model and its realisation, from Gustafson and Olsson (1993) and Gustafson (2012).

Models of ground 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, 2012). The starting point for the engineering geological model is the application and not the geology. The same geological setting 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 data, boundary conditions and calculation tools, and should be addressed using two separate models.

Geological 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 (NRC, 1996). The 'total geology' model approach presented by Fookes et al. (2000) 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.

3.2 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 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, VOIA, see e.g. Bedford and Cooke (2001), Danielsen (2010) and Zetterlund (2009).

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.

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). 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.

Important sources of information during excavation are mapping of exposed tunnel faces and probing in advance of the tunnel excavation.

Ideally, investigations are 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. Moreover, 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). 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.

3.3 Characterisation and classification

Data collected during site investigations need to be processed and converted into information to be of importance to the project (Harrison and Hadjigeorgiou, 2012). 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. 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 3.2, is an example of systematic characterisation of rock mass composition developed for rock mass strength.

3.3.1 Empirical rock mass classification 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). 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).

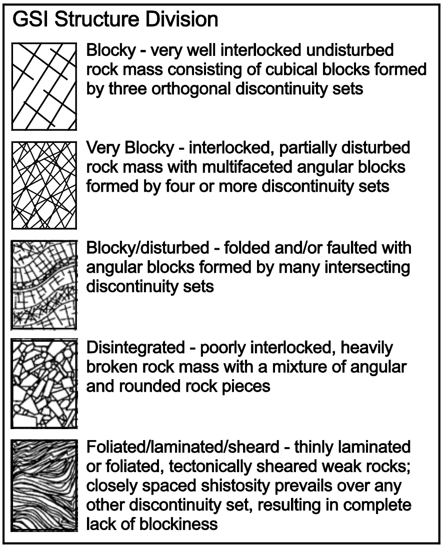


Figure 3.2 Characterisation of rock mass structures in the Geological Strength Index (GSI). Modified from Hoek et al. (1998).

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). The quantified parameters often incorporate design loadings, which imply that descriptions of these cannot be seen strictly as characterisation. However, the parameters may be part of the characterisation process if they exclude factors related to the design, such as fracture orientations and stress conditions in relation to the tunnel orientation.

There are quite a number of publications that discuss the shortcomings of empirical rock mass classification systems, e.g. Riedmüller and Schubert (1999), Palmström and Broch (2006) and Milne and Hadjigeorgiou (2000). Commonly discussed shortcomings include the loss of valuable information about the rock mass structure when various rock mass properties are combined into a single index. This leads to homogenisation of the rock mass, which does not take account of differing failure modes or anisotropic and time-dependent behaviours of the rock mass (Riedmüller and Schubert, 1999). 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. 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 (Stille and Palmström, 2003).

3.3.2 Rock classes and construction 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 (Holmberg and Stille, 2007). Examples of possible classes are design of rock support (Figure 3.3), grouting and excavation. 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, 2012), see schematic representation in Figure 3.4. These can vary, for instance, according to different layouts, varying sensitivity of the surroundings, or the existence of nearby constructions. Design classes are also adapted to various expected ground conditions and ground behaviours at the site, i.e. the geological settings (Gustafson, 2012). 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 represents 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

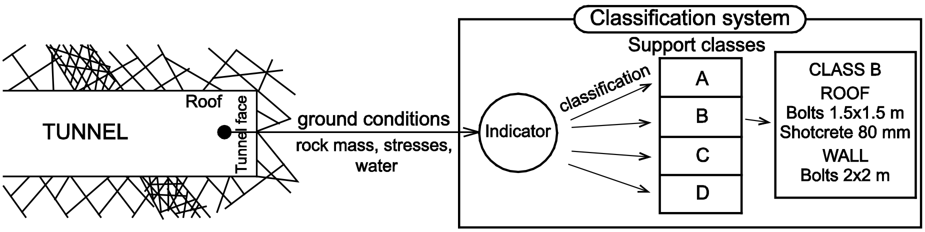


Figure 3.3 An example of a classification scheme for support classes (modified from Stille and Palmström, 2003).

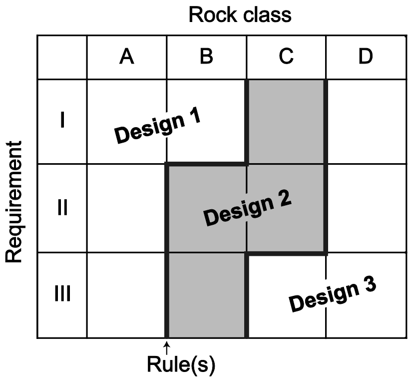


Figure 3.4 Matrix displaying the relationship between the geological settings (rock class), the project requirements and the stepped, pre-defined designs (design classes) (Gustafson, 2012). The rules are the selection criteria that are observed to be able to decide which construction class apply for the subsequent excavation stage. For the same geological setting, different designs may need to be implemented due to differing requirements.

together in classes with similar engineering characteristics, often labelled 'rock classes', see Figure 3.4 (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) they should depict the range of conditions that can reasonably be anticipated or foreseen at the site. Baynes et al. (2005) label the groups used to communicate geological settings as 'reference conditions' and list some important functions of the geological classification:

- Formally define and describe the components of the geological models
- Simplify the geology by grouping together geological materials with similar engineering characteristics and thus facilitate communication between geologists and engineers
- Document conditions that can be reasonably foreseen for contract purposes
- Reduce the number of investigations – representative tests for each reference condition are required rather than for every material encountered
- Allow the incorporation of knowledge from similar geological units outside the project area that can then be correlated with the reference conditions
- Predict design performances and choose construction classes.

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.

3.4 Uncertainties in engineering geological information

Construction of underground facilities involves uncertainties that give rise to geotechnical risks in the decision-making process, i.e. a situation that possibly involves a loss, disaster or other undesirable outcome (Hammah and Curran, 2009). 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, choice of design properties and engineering analyses from risks that arise from managing and communicating risks, i.e., 'project management risks' and 'contractual 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 and contractual risks are explained further in the following sections.

3.4.1 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 ground 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'.

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).

3.4.2 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.

3.4.3 Handling of geotechnical risks and uncertainties

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 (Fookes, 1997). There are, however, techniques to handle different geotechnical uncertainties. Baynes (2010) relates certain established techniques to idealised phases in a project, which is illustrated in Figure 3.5.

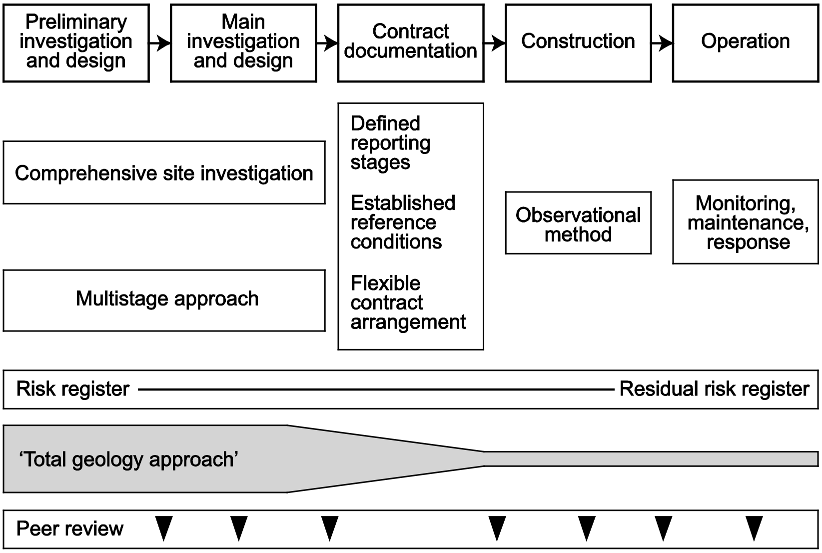


Figure 3.5 Techniques for managing geotechnical risks in various project phases (modified from Baynes, 2010).

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
- 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).

The choice of investigation method is influenced by the experience of the personnel and what they consider to be of value to their investigation. Alm et al. (2013) present a study of attitudes towards site investigation methods in Sweden and it was found that direct *in situ* methods, such as core drilling, test pumping and soil-rock sounding, are considered to be the most valuable. It is difficult, however, to provide general recommendations for appropriate methods and the 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. This has been studied by, for example, Zetterlund (2009) and Danielsen (2010).

Uncertainty in parameters derived from inherent random (aleatory) variability cannot be reduced by conducting additional investigations. Uncertainty 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.

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, 2012). 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 HYDRAULIC PROPERTIES OF FRACTURED ROCK

An important difference between groundwater flow issues and stability issues is that stability is a local problem whereas groundwater flow also involves the groundwater balance on a larger scale (Gustafson and Wallman, 1995). Groundwater can flow considerable distances through the rock mass and there can be substantial temporal variations in the groundwater balance during the year and between years. The groundwater flows through complex fracture geometries and, coupled with the effects of geological, hydrological and chemical processes, this causes hydraulic properties to be highly variable and heterogeneous (NRC, 1996; Olofsson et al., 2001).

The hydrogeological conditions in the rock mass are influenced by a combination of independent and interrelated factors on the local as well as regional scale (see Figure 4.1). Local factors mainly include the characteristics of the fractures and the fracture system, whereas regional to sub-regional factors include topography, groundwater recharge, rock type distribution and regional stresses (Gustafson, 2012; Olofsson et al., 2001).

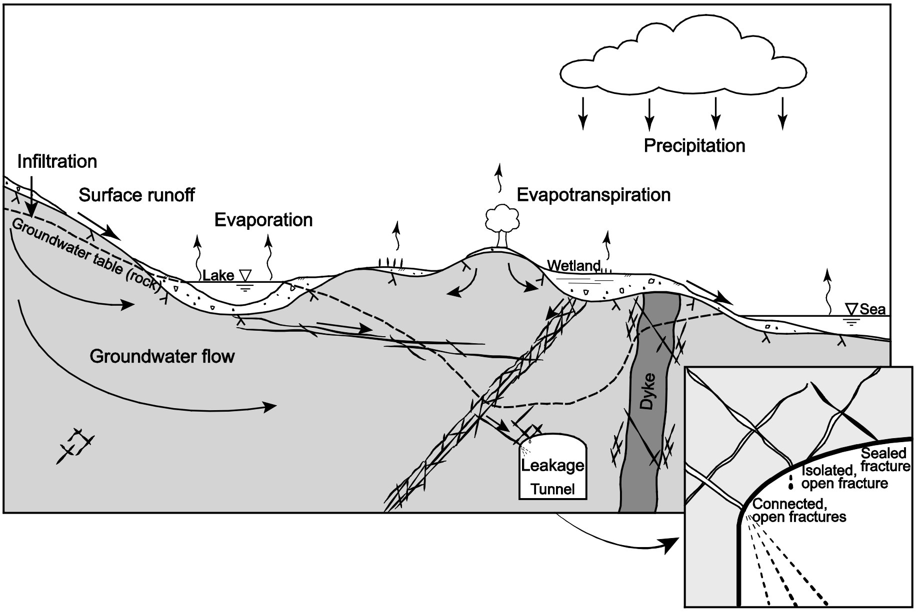


Figure 4.1 Conceptual model of groundwater flow in an area in which a rock tunnel is being excavated.

4.1 Fractures and fracture systems

An understanding of the characteristics of the fracture system is essential to describe water transport in fractured rock. However, not all fractures actively participate in the groundwater flow (see e.g. NRC, 1996). Hydrogeological prognoses should therefore focus on identification and localisation of water-conductive structures and determine the extent to which they conduct water.

A fracture is a mechanical break in the rock that can be visualised as two rough surfaces that are more or less in contact with each other (NRC, 1996). A conceptualisation of a water-conductive fracture is shown in Figure 4.2. Voids occur in areas where there is some distance between the surfaces and these voids can form interconnected networks of flow paths within the fracture. The distance between the surfaces is called aperture and the flow in a fracture depends mainly on the aperture, the amount of contact area and the spatial distribution of the contact area (Zimmerman and Bodvarsson, 1996). Fracture flow is also affected by the state of stress in the rock and the distribution of fracture fillings (NRC, 1996). The amount of contact area increases with the confining stress and the introduction of infillings in fractures will usually decrease the aperture.

Studies of flow in natural fractures indicate that channelling and highly preferential flow paths exist in individual fractures and in fracture networks (Tsang and Tsang, 1989). The formation of channelled flow paths is influenced by the aperture variability within fracture planes and among fractures as well

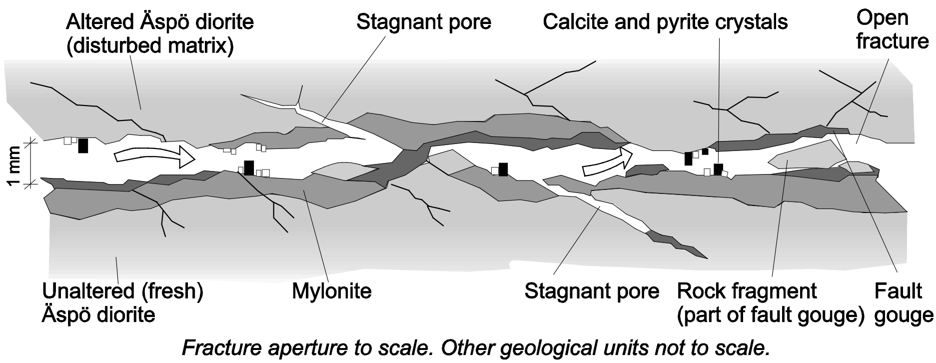


Figure 4.2 A conceptually visualised fracture from field tests at Äspö HRL (modified from Winberg et al., 2000).

as the geometrical connectivity of fractures (Margolin et al., 1998). The fracture connectivity describes the relative number of interconnections between fractures within a fracture network and a higher level of fracture interconnectivity results in higher overall rock mass permeability (NRC, 1996). The connectivity is linked directly to other geometrical factors, such as fracture density, fracture lengths and fracture orientation, but in itself it is a significant factor since a densely fractured rock mass can exhibit low connectivity and permeability if the fractures have similar orientations and few intersections (Berkowitz, 2002).

Different types of flow dimensions occur within the rock mass and these can reflect the geometry of the conductive flow paths within fractures and fracture networks (Doe and Geier, 1990). The flow dimensions are illustrated in Figure 4.3. Constrictions within a fracture force the flow to become channelled ('1D flow') whereas large fracture apertures enable a radial spread ('2D flow'). In the case of a well-connected fracture network, such as heavily fractured deformation zones, the flow can be interpreted as spherical ('3D flow').

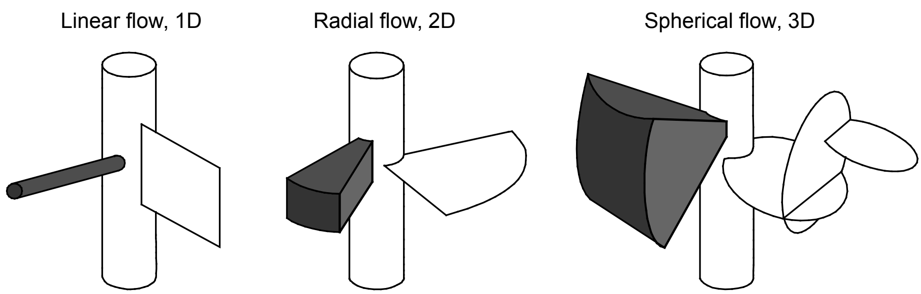


Figure 4.3 Flow dimensions within porous media (to the left) and fractured media (to the right). Modified from Doe and Geier (1990).

4.2 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). 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. Brittle deformation zones are composed of two main structures of importance for hydraulic flow: a high-strain core, where the main displacement has occurred, and a low-strain damage zone (Caine et al., 1996; Faulkner et al., 2010), see Figure 4.4. The transition between the damage zone and the undisturbed host rock is typically marked by a decrease in fracture frequency to a background fracture level (Faulkner et al., 2010).

Deformation zones can offer highly conductive flow pathways within the rock mass although their heterogeneous composition generally results in internally heterogeneous hydraulic properties (Caine et al., 1996). The core may be clay-

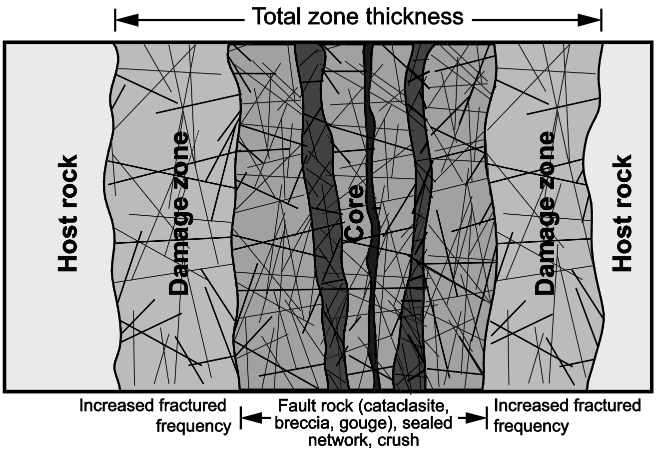


Figure 4.4 Typical structure of a brittle deformation zone (Munier et al., 2003).

altered and act as a low-permeability barrier against flow (Babiker and Gudmundsson, 2004). In contrast, the more intensely fractured damage zone is likely to act as a conduit and a positive boundary for the less permeable host rock. The composition and structure of the deformation zone determines whether the zone acts as a conduit that transports groundwater or as a barrier to groundwater flow (Babiker and Gudmundsson, 2004). The contrasting hydraulic properties of the deformation zones in relation to the surrounding host rock generally imply that zones have a significant influence on the subsurface flow pattern (NRC, 1996). The influence, however, is concentrated on local flow patterns, not regional patterns (Ericsson and Holmén, 2010).

Fracture zones that display shear movements are commonly referred to as fault zones. A conceptual scheme for classifying permeability structures in fault zones is presented by Caine et al. (1996), see Figure 4.5. The scheme describes the barrier-conduit structure of the fault zone based on the development of the fault core and the damage zone. Four distinctive styles have been defined: (i) localised conduits (e.g. localised shear zone, absent to small damage zone), (ii) distributed conduit (e.g. fracture zone with interlinked fractures); (iii) localised barrier, and (iv) combined conduit-barrier (e.g. deformation zone with a well-developed core and damage zone).

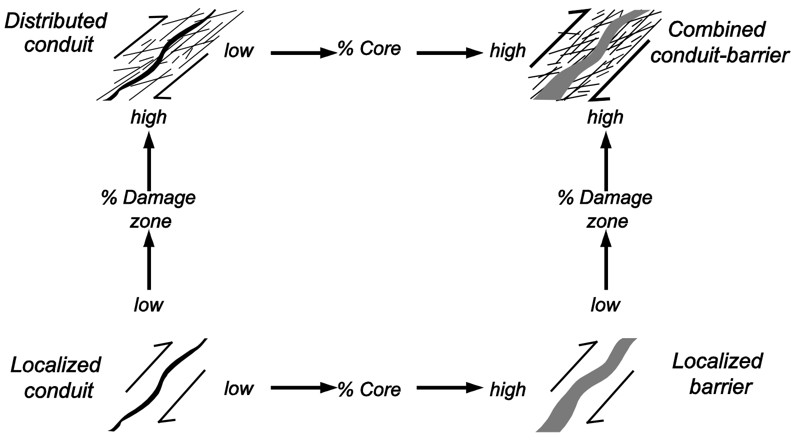


Figure 4.5 Conceptual scheme for permeability structures in fault zones (modified based on Caine et al., 1996).

4.3 Large-scale influencing factors and processes

The hydraulic properties of the rock mass are affected by factors and processes that are of significance on a larger scale, e.g. rock stresses, groundwater recharge, topography and rock type distributions.

The current stress situation in the rock affects fracture apertures and network connectivity and the permeability of the rock will normally decrease with increased confining stress. Stresses at depth in rock will be governed by regional tectonic stresses and pressures due to the gravitational loading of overlying rocks. Normally, the rock stresses increase with depth and consequently the permeability of the rock will decrease with depth. Stress ratios can also affect flow anisotropy within the rock mass (NRC, 1996). Hydraulic tests conducted in the main tunnel at the Äspö Hard Rock Laboratory (HRL) indicated that the hydraulic conductivity was around 100 times greater in the most conductive direction than in the least conductive (Rhén et al., 1997), see Figure 4.6. These anisotropic conditions, with the largest transmissivity within the WNW to N-S sector, coincide with the main horizontal stress orientation, which is oriented approximately NW at the Äspö site.

The groundwater recharge, i.e. the water flow that reaches the groundwater table, is an important item of hydrological information for evaluating the amount of water that is active in the system. Recharge occurs when the water flow is directed downwards and as the groundwater flow pattern in undisturbed rock is driven mainly by gravity, this occurs when water flows from higher to lower areas in the terrain. The topography therefore has a significant influence on local flow patterns and groundwater levels (Ericsson and Holmén, 2010).

The potential groundwater recharge in rock depends on the maximum amount of precipitation that can be added to the groundwater (net precipitation), whether there are water-saturated soil layers on top of the bedrock surface and whether there are any hydraulic connections to surface water bodies (Olofsson et al., 2001). The recharge also depends on whether the groundwater flow is disturbed or undisturbed by underground construction or pumping of wells. Abstraction of groundwater from the rock increases the groundwater recharge from soil layers to the bedrock considerably (Gustafson, 2012).

The distribution of rock types can be of interest when describing the hydraulic properties of the rock mass. The hydraulic significance of various rock types may not be clear since the spread in hydraulic conductivities within each rock type is typically larger than the difference between different rock types (Gustafson, 1986). However, rock types can respond differently to stress and deformation (Gustafson, 2012). This provides some general differences, e.g. that felsic rock types tend to fracture more easily than mafic rock types.

Holmøy (2008) studied the significance of certain geological parameters to predict water inflows in tunnels and found that high water inflows often concur with rock type boundaries. Dykes are generally regarded as important since they can act either as conductors or barriers to groundwater flow depending on whether they are more fractured than the surrounding rock or less fractured (Babiker and Gudmundsson, 2004). The effect of dykes, however, is mainly of importance for local flow patterns (Ericsson and Holmén, 2010).

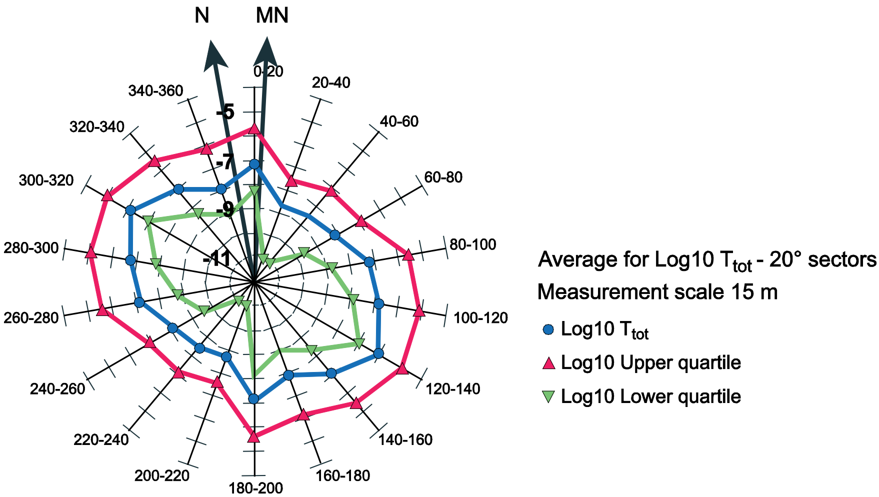


Figure 4.6 The anisotropic hydraulic conditions at the Äspö site, shown with estimated transmissivities (T) from probe holes in the main tunnel. The curves display geometric means and the upper and lower quartiles in 20° sectors. Modified from Rhén et al. (1997).

Knowledge of the geological history of the area, i.e. the origin and formation of the rock, is crucial to understanding how various rock types relate to each other and how they affected each other during formation (Gustafson, 2012). Moreover, the chronology and orientation of various stress fields affect fracture networks over geological time. The orientation and type of brittle fractures that make up a fracture set will be determined by the orientation and magnitude of the causative stress field (Cosgrove et al., 2006). Each stress episode can result in reactivation of existing fractures and the formation of new fracture sets (Gustafson, 2012). New fracture sets interact with older sets during their formation and older sets can become progressively more deflected and less continuous. Information about the creation and reactivation of fractures and deformation zones can provide insight into the probable continuity of the various fracture sets and how they are interconnected. This can be useful for identifying hydraulically active structures (Cosgrove et al., 2006).

4.4 Assessment of hydraulic properties

Many of the previously mentioned characteristics of fractures and fracture networks are difficult to determine *in situ*. Few measurement techniques actually measure fractures directly and data are extrapolated and interpreted differently depending on the model chosen to solve the problem (Berkowitz, 2002). The mathematical model selected to describe the hydrogeological conditions should be appropriate for the purpose for which the analysis is being made as well as the scale of interest (NRC, 1996).

A common approach to the analytical estimation of flow in fractured bedrock is to use a continuum approach. The flow properties of bedrock are then expressed using the hydraulic conductivity, *K*, which is the proportionality constant of a porous medium in Darcy's Law (see e.g. de Marsily, 1986) which is expressed by Equation 4.1:

$$Q = -KA \cdot \left(\frac{dh}{dl} \right) \qquad \text{Eq. 4.1}$$

Darcy's Law states that the flow of water *Q* through a pipe with a cross-sectional area *A* is proportional to the hydraulic gradient (*dh/dl*). The hydraulic gradient expresses the difference in hydraulic head that the flowing water exhibits over a certain distance. The dimension of *K* is that of a velocity and is usually expressed in metres per second.

The continuum approach assumes that the flow is laminar and that the flow in the fractured rock volume can be represented reasonably by flow through a porous medium. This could be the case when the interest is mainly on volumetric flow in larger sample sizes. Estimations of the effective hydraulic conductivity of a larger scale of bedrock, derived from small-scale values, is generally made using standard mathematical averaging, mainly using the arithmetic, geometric (median) or harmonic mean (Gustafson, 2012).

Groundwater flow may also be calculated through studies of flow in individual fractures. The flow in a fracture is described more appropriately using the transmissivity, *T*, which is the product of the hydraulic conductivity and the thickness of the aquifer, i.e. the fracture aperture, *b*:

$$T = K \cdot b \qquad \text{Eq. 4.2}$$

The transmissivity has the unit m^2/s and provides a measure of the ability of a fracture to transmit water, which is a preferred parameter for many hydrogeological purposes. In areas where hydraulic test data is missing, it is common to estimate the transmissivity from specific capacity data. The specific capacity is the standard variable for rating well-productivity in crystalline rock aquifers and is defined as the flow of water *Q* divided by the change in the hydraulic head *dh* during the test (Gustafson, 2012). The transmissivity is linearly proportional to the specific capacity and it has been shown that the specific capacity for short-duration tests can provide a reasonable estimate of the transmissivity (Fransson, 2001):

$$T \approx Q / dh \qquad \text{Eq. 4.3}$$

Specific capacities calculated from well databases are approximately log-normally distributed, which together with the two-dimensional flow situation around the borehole enables a reasonable estimate to be made of the effective transmissivity of the rock mass from the geometric mean of specific capacities. This relationship can be used, for instance, for tunnel inflow analysis (see Gustafson, 2012), although there are factors that will influence the analysis negatively. Wells, for instance, are biased because they are drilled until a sufficient yield is reached. Division of the well yield Q by the total well depth d instead of the drawdown can be done since the test method gives the well capacity at virtually full drawdown (Gustafson, 2012). It does, however, provide a somewhat conservative estimate of the specific capacity.

The flow of water in a fracture can be related to a hydraulic aperture, which represents the thickness of a slot through which the water flows if the fracture has plane, parallel walls. An estimate of the relationship between the transmissivity T and the hydraulic aperture b of the fracture is provided using the cubic law (de Marsily, 1986; Snow, 1968);

$$T = \frac{\rho g b^3}{12\mu} \qquad \text{Eq. 4.4}$$

where ρ is the fluid density, μ is the dynamic viscosity, and g is the acceleration of gravity. The cubic law is based on laminar flow and idealises fractures as equivalent parallel plate openings.

5 GROUTING DESIGN

5.1 An overview of grouting in fractured rock

The inflow of water into an excavated tunnel can be reduced by injecting grout into fractures and blocking the flow paths for the water. The method is termed permeation grouting and it is typically carried out as pre-excavation grouting (Figure 5.1), which means that grout is pumped into boreholes drilled ahead of the excavation, see e.g. Houlsby (1990). 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 for each project facilitates the specification of engineering requirements and preferences (Andersson et al., 2000: Gustafson, 2012). Examples of key issues related to grouting are presented in Table 5.1.

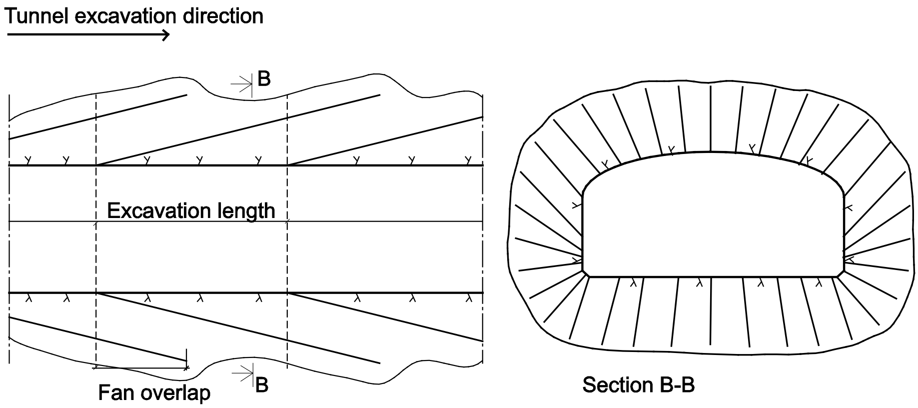


Figure 5.1 A pre-grouting design layout in profile (left) and section (right). The dotted area represents a theoretically grouted zone.

Table 5.1 Example of a list of key questions with a hydrogeological focus for underground construction. Based on Gustafson (2012).

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)

The knowledge of the geological settings can be structured into disciplines relating to different processes. For a deep nuclear waste repository, a distinction is made between geology, thermal properties, hydrogeology, rock mechanics, chemistry and transport properties (Andersson et al., 2000). Some of these disciplines may be less important for grouting but a similar division of geological factors can be made in grouting design to ensure that important matters and aspects are not overlooked.

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, 2012). 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, and facilitates the communication of what is known about the problem. Examples and further details on interaction matrices can be found in Gustafson (2012) and Hudson and Harrison (1997).

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; Funehag, 2007). The grouting technique, however, must be adapted to prevailing conditions at the site. The hydraulic properties of the rock mass are therefore investigated prior to design (Gustafson, 2012).

5.2 Investigation of hydraulic properties

Hydraulic properties of the fractured rock mass are generally evaluated using various types of hydraulic tests. Hydraulic tests measure controlled disturbance of the groundwater flow in the bedrock and can be carried out in a single borehole or in several boreholes simultaneously, i.e. interference tests. Other useful investigations include geophysical methods and geological mapping of boreholes or rock exposures. Detailed descriptions of various hydraulic tests and flow analysis methods can be found, for instance, in Earlougher (1977) and de Marsily (1986). Guidelines on hydraulic investigations are presented, for instance, by Gustafson (1986) and (2012).

Common hydraulic tests are the water inflow measurement, the pressure build-up test and the water pressure test (WPT). The water pressure test is conducted by injecting water into a packer-isolated part of a borehole and is sometimes also referred to as a Lugeon test (Houlsby, 1990). The water loss can be converted into a Lugeon value, which is a unit defined as the water injection rate of one litre per minute into one metre of test length at an injection pressure of 1 MPa. Lugeon values provide averaged flow properties of the rock mass, but not necessarily the properties of the fractures that need to be sealed. Further analyses of the water pressure tests are therefore considered advantageous (Emmelin et al. 2007).

The suitability of different investigation methods depends on the site and the purpose for which data is produced. Analyses of the test commonly use analytical domain solutions corresponding to homogeneous aquifers (de

Marsily, 1986). There are limitations and assumptions associated with these solutions and test evaluations need to consider the initial conditions at the beginning of the test, flow dimensions, boundary conditions and test duration. Another issue is that test results from individual boreholes often correspond to local properties and it may therefore be difficult to interpolate between measurement points (NRC, 1996).

Hydrogeological investigations are conventionally carried out during site investigations to form a basis for the preliminary grouting design, e.g. selection of grout (Fransson et al., 2012). They are also often carried out during construction to confirm that permeability reductions are within acceptable limits and satisfy the stipulated requirements. Information on rock mass properties and grouting response consequently increases as the production cycle proceeds (Dalmalm, 2004). A formal approach to optimising and adjusting the grouting process based on the additional information obtained during construction is offered by the observational method. The investigation parameters, however, must provide relevant information on the grouting behaviour and be measurable during the grouting process. Monitoring the water inflow after grouting is less suitable since the result can only be verified once the tunnel is finished and weirs are installed. Hydraulic tests in probe boreholes drilled before and after grouting can provide measurable quantities, which can be used as proxies to estimate the actual inflow (Gustafson et al., 2010; Hernqvist et al., 2013). The time, flow and pressure recordings is another example of observations made during grouting, and can be used to calculate flow dimensions and grout penetration (Gustafson and Stille, 2005).

5.3 Grouting design in Swedish tunnel projects

Inflow requirements and the calculated hydraulic conductivity of the rock mass commonly form a basis for grouting design in Swedish tunnel projects (Stille, 2012). 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; Stille, 2012). 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 5.2. Based on the distribution of fracture apertures and the required sealing efficiency, it is possible to choose a suitable 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, 2012).

5.3.1 Class division types

According to requirements presented by the Swedish Transport Administration (2011a), water-mitigating measures applied in Swedish tunnel projects should be represented and described using different classes. These classes can express variations in the required hydraulic conductivity of the sealed zone – sealing classes – or different grouting designs to be implemented in response to variations in the hydraulic properties during excavation – design classes (Emmelin et al., 2007).

The *sealing class division* arises due to the varying requirements along the tunnel alignment, such as maximal permissible inflows, grout spread restrictions or grout pressure limitations due to nearby underground structures. The sealing

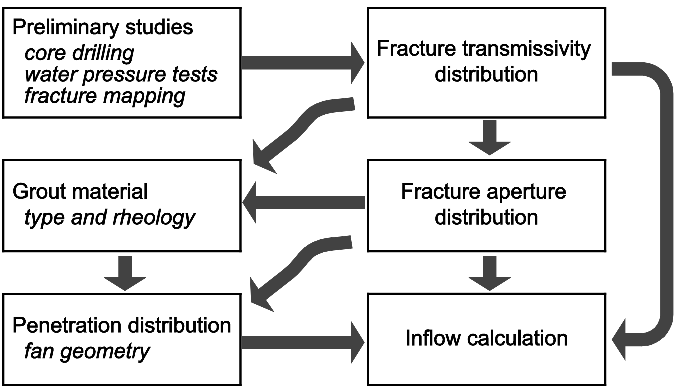


Figure 5.2 Analysis process for preliminary grouting design (Gustafson et al., 2004).

classes often correspond to variations in permissible inflows and pre-defined grouting designs are developed to be appropriate for each class, e.g. by modifying fan layouts, grout types or what remaining inflows to accept after grouting. An example of a Swedish road tunnel project where sealing classes were implemented is the Törnskog Tunnel project in Stockholm, Sweden (Swedish Road Administration, 2003). In the basic design, it was stated that the inflow requirement varied according to two sealing classes. The standard design was the same for both classes although the WPT result that was accepted after grouting varied for the different classes, see Figure 5.3. If the requirement was not fulfilled re-grouting with extra grout holes was to be carried out.

The *design class division* focuses on an adjustment of grouting designs due to varying ground conditions, such as variations in hydraulic conductivity, rock cover and hydraulic head. The grouting design of the Namntall Tunnel, part of the Bothnia Line project in Northern Sweden, was adapted to different

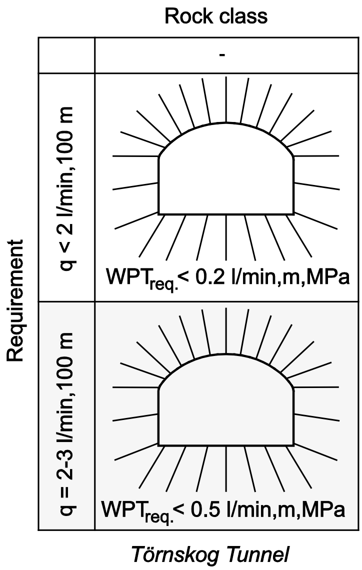


Figure 5.3 Pre-defined grouting strategies – sealing classes – stated in a construction document for the Törnskog Tunnel (Swedish Road Administration, 2003), adapted to the matrix in Figure 3.4. No specific rock classes were stated.

hydrogeological regimes based on Lugeon values from water pressure tests in probe and grout holes (Stille and Andersson, 2008). The largest water pressure test result for each fan determined the grouting execution according to three pre-defined grouting classes, illustrated in Figure 5.4. The permissible inflow and the accepted Lugeon value in control boreholes after grouting were constant for the whole tunnel.

Grouting design for the TASS Tunnel at the Äspö HRL was also adapted to hydrogeological conditions, although the focus was on distribution of hydraulic apertures instead of the Lugeon value (Funehag and Emmelin, 2011). The grouting material in the TASS Tunnel was chosen according to the hydraulic aperture calculated from borehole investigations.

Often there is a combination of the two types of class division, i.e. grouting work is affected by variations in requirements and variations in the encountered hydrogeological conditions. The City Line rail project in Stockholm, Sweden, had two grouting designs based on varying requirements. These were expressed as sensitivity classes (Swedish Rail Administration, 2008). The grouting design was also said to be adapted to ground conditions (such as high transmissivity and passages of zones), although a clear definition of the ground conditions, such as for the Namntall Tunnel and TASS Tunnel, was not stated in the grouting strategy document for the City Line project.

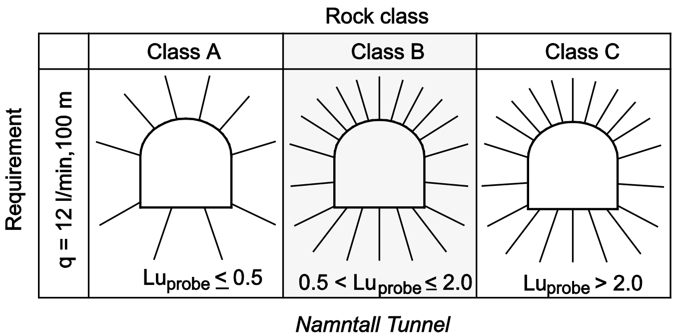


Figure 5.4 The main grouting class division for the Namntall Tunnel (Stille and Andersson, 2008), adapted to the matrix in Figure 3.4. The inflow requirement was constant and the grouting design only varied for it to be suitable for the varying hydrogeological conditions, expressed using Lugeon values.

5.3.2 Implementation and control of grouting

The gradual sealing of a tunnel should be within acceptable limits and control of the sealing efficiency is made by observing quantifiable parameters during and after grouting (Eriksson and Stille, 2005). Common observations to verify requirement fulfilment are water flowing over weirs installed in tunnel sections, water pumped from the drainage system, drips and wet spots on the tunnel periphery and groundwater levels in soil and rock. However, observations may also be useful during the grouting cycle to check if the sealing has been sufficient or if extra grouting rounds are needed to increase sealing efficiency before excavation continues. Decisions regarding extra rounds are mainly based on WPT results in control boreholes drilled after each grouting round (Dalmalm, 2004). In tunnels where sealing requirements vary, this implies that a certain WPT result can be approved in some areas, whereas in other areas the same result leads to re-grouting.

Observations from hydraulic tests can also be useful when choosing and implementing appropriate grouting designs. Hydraulic tests are then carried out in probe holes drilled prior to drilling the grouting fan. Based on the test results, a design concept and fan layout are chosen and executed. A variation is that no specific probe holes are drilled and instead the hydraulic tests are carried out in the grout holes prior to grouting. There are also tunnel projects where standard designs are implemented and no probing is carried out prior to grouting (Dalmalm, 2004). However, the absence of testing before and after each grouted fan implies that no assessment of sealing effect can be made.

The water pressure test is the most commonly used method for investigating hydrogeological conditions before grouting. Some projects also use geological mapping of the excavation front and observations from the drilling to provide information about the position of fracture zones and other significant rock mass deviations ahead of the tunnel front (Emmelin et al, 2007).

5.4 Geological parameters considered in grouting design

An understanding of the characteristics of the rock mass is considered essential for grouting design and a great deal of research has been undertaken to identify geological parameters of significance for grout spread in crystalline, fractured rock, e.g., Fransson (2001), Eriksson (2002), Gustafson and Stille (2005), Hernqvist (2011) and Butrón (2012). Eriksson (2002), for instance, pointed out some important parameters that could identify various rock mass conditions, and showed that grouting design adapted to a specific rock mass theoretically results in higher sealing efficiency and reduced grouting time. Stille and Gustafson (2010) showed that a number of geological factors (zone characteristics, rock types, fracture frequency and *in situ* stress) can explain large-scale distributions of water losses in the rock mass. Descriptions of these geological factors were therefore considered important for preparing the extent of the grouting measures required to fulfil the inflow requirements in different parts of the tunnel.

According to the Swedish Transport Administration (2011a), the design of water-mitigating measures should at least consider the rock types, fracture system, deformation zones, fracture properties, weathering, occurrence of water, hydraulic head and fracture fillings. How these factors could influence the grouting design is presented below, together with some other hydrogeological aspects discussed in Chapter 4.

- The fracture aperture is an important geological parameter since it influences whether the grout can enter the fracture, i.e. penetrability, and the penetration length within the fracture. Hydraulic apertures around 50-100 μm are expected to be groutable using a cement-based grout (Eklund and Stille, 2008; Axelsson, 2009) and an analytical solution to calculate penetration lengths for cementitious grouts is described by Gustafson and Stille (2005). Newtonian grouts, such as silica sol, can penetrate finer apertures and an analytical method to estimate the penetration length for a gelling Newtonian fluid is presented by Funehag (2007). Knowledge of the distribution of hydraulic apertures can be useful to estimate the extent the chosen grout can penetrate and spread in individual fractures, i.e. the number of fractures that are possible to seal with the chosen grouting method (Fransson et al., 2012).

- The geometry of grout holes should have a high probability of intersecting water-conducting fractures. A fan layout should therefore be adapted to the orientation and frequency of the fractures to be sealed (Palmqvist, 1983).

- The grout holes should intersect the conductive parts of the fracture planes. A large contact area within the fracture implies that the flow becomes more channelled and the conductive paths are consequently more difficult to intersect (Butrón, 2012). A small number of wide channels are more easily penetrated than a large number of narrow channels (Gustafson, 2012). The variability of the aperture also affects grout filtration (Axelsson, 2009).

- The interconnection between fractures can influence the grout spread and the extent of the grouted zone. In a low-connectivity network, sealing may occur in fractures intersected directly by the grout holes although there is restricted spread to other fractures (Hernqvist, 2009; Butrón, 2012). The distances between the grout boreholes should preferably be adapted to the length of the fractures that require grouting. Fransson and Gustafsson (2000) found that moderately transmissive fractures at Äspö HRL had correlation lengths of around 4 metres.

- Grout spread in and between fractures may be blocked by clay and other soft fracture fillings that are present, for instance, in fracture zones or other weathered parts of the rock. Soft fillings can move during high-pressure grouting and consequently open and close flow paths (Houlsby, 1990).

- The rock stresses are redistributed after excavation, which may cause the rock mass to move and new conductive flow paths to form in grouted areas. This is of particular concern in rock masses where the *in situ* stress is low and the mobility of blocks is high (Fransson et al., 2010b).

- High hydrostatic pressures require the use of high pressures during grouting. This can cause problems with high water flows from boreholes and high pressures acting on the drill rods (Chang et al., 2005).

- The pressure difference between the excavated tunnel and the water in the surrounding rock creates water flow towards the tunnel. At large depths there are high groundwater pressures and a risk of high hydraulic gradients, which implies that the risk of backflow and grout erosion must be considered in the grouting design (Axelsson, 2009).

- Water inflow into underground constructions can affect the chemical composition of the groundwater in the bedrock and may cause groundwater to become more aggressive towards the construction materials compared to unaffected conditions, see e.g. Mossmark (2010). Lowering of the pH due to acidification and upconing of highly corrosive saline waters are examples of hydrochemical changes that may cause degradation of cement-based grouts. Mossmark (2010) identifies three types of geological settings that are the main contributors to these negative effects: wetlands (and groundwater discharge areas), the presence of sulphide minerals in the bedrock and marine and brine waters.

- Deformation zones can offer highly transmissive flow pathways within fractured rock and there are studies that indicate a substantial increase in injected cement volumes in deformation zones compared to areas with background fracturing (Ganerød et al., 2008). However, the flow in prominent fracture zones could be low due to impermeable clay content, especially in the core of the fault zone (Banks et al., 1992). Permeability differences within the zone generally imply that more grout is injected into the damage zone than in the fault core (Ganerød et al., 2008). A deformation zone may also act as a 'backbone' for groundwater flow, supplying intersecting fractures with water (Hernqvist, 2009). The sealing of such zones may also result in a flow reduction in connected fractures too fine to be grouted.

5.5 A conceptual model for 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 parameters provide information on fractures that need to be sealed and input for choosing grouting design parameters (grout, fan geometry, pressures etc.).

Furthermore, Fransson and Hernqvist (2010) also suggest a conceptual scheme of permeability structures that distinguishes between host rock and deformation zones, see Figure 5.5. The suggested conceptualisation scheme provides a basis for adaption of the grouting design and may also be useful for inflow predictions as it could clarify whether the assumptions of the analytical equation and the input data are reasonable for the water-conductive fracture system (Hernqvist, 2011).

The scheme was introduced by Fransson and Hernqvist (2010) and consists of the model for permeability structures in fault zones presented by Caine et al. (1996) (see Figure 4.5) expanded with sparsely connected and well-connected host rock networks. The sparsely connected fracture network (Type I) displays few connections between hydraulically active fractures and the flow pattern is typically associated with anisotropy and flow restrictions ('bottlenecks'), see e.g. Black et al. (2007). Two-dimensional flow dominates within the sparsely connected network although flow channelling can reduce flow spread to one dimension. The well-connected fracture network (Type II) has a large number of interconnections between water-conductive fractures. Three-dimensional flow usually dominates as flow spreads through a high number of interconnected fractures, although the flow pattern may be anisotropic and become closer to a 2D flow due to differing fracture set properties (Fransson and Hernqvist, 2010).

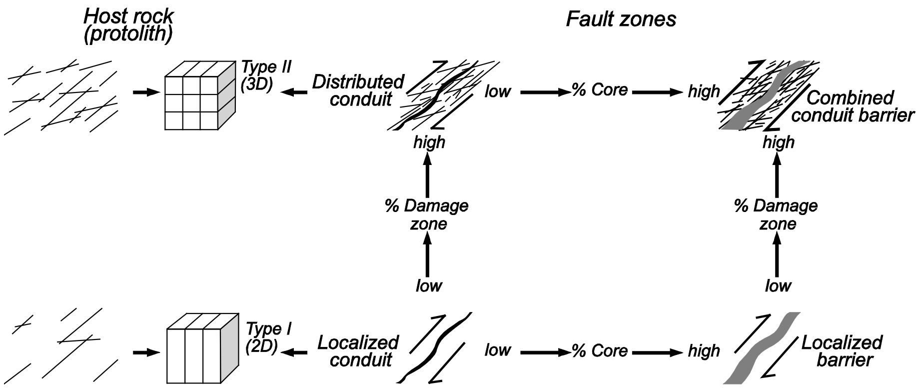


Figure 5.5 Conceptual scheme for permeability structures in host rock (Fransson and Hernqvist, 2010) and in deformation (fault) zones (modified from Caine et al. 1996).

6 RESULTS FROM CASE STUDIES

The previous chapters have concentrated on descriptions on the general use of engineering geological information and some current practises for dealing with grouting design prerequisites in Swedish tunnelling projects. These descriptions have indicated a need for developments in the way geological information is handled within the grouting design process. Some suggestions for improvements have therefore been made with the use of two case studies, whose main results are presented in this chapter.

The two case studies exemplify an approach for handling engineering geological information in grouting design. Both studies involve the classification of geological settings into hydraulic domains, although one (Chapter 6.1) focus on describing and exemplifying the approach whereas the other (Chapter 6.2) uses the approach and concentrates on evaluating executed investigation methods and grouting designs. The first study was applied on the Hallandsås project and focus on information available at early project phases, whereas the second study was carried out using detailed information available from grouting trials carried out at the KBS-3H experimental site at Äspö HRL, cf. Figure 1.1.

6.1 Early grouting prognosis applied to the Hallandsås project

The Hallandsås case study outlines a structured approach to evaluate geological prerequisites for grouting design in the early phases of tunnel projects. The method is exemplified by using low-cost geological information (e.g. geological maps and well logs) available during the project's original feasibility study, and observations from the construction phase were used to show how it is possible to confirm anticipated hydrogeological behaviours. The approach is a development of a method suggested by Fransson et al. (2010a) and a complete account for this case study is presented in Kvartsberg (2013).

6.1.1 Method

The approach adopted includes a division of the engineering geological prognosis into (i) conceptualisation of the hydrogeological behaviour of the rock mass, (subdivision into hydraulic domains), (ii) tunnel inflow estimates and (iii) potential environmental impact on the surroundings. From these parts, it is possible to estimate the need for sealing and the extent of the sealing required and to relate the grouting design to the various hydraulic domains along the tunnel alignment.

The hydraulic domains represent rock units with similar engineering characteristics in terms of flow configuration. The focus on hydraulic properties is considered more useful for the design of water-mitigation measures than a subdivision of the rock mass established from other physical properties. The hydraulic domain division is based on a distinction between host rock (*Hydraulic Rock Domains, HRD*) and deformation zones (*Hydraulic Conductor Domains, HCD*), which is a terminology introduced by Rhén et al. (2003). The hydraulic domains formally define what to expect in terms of hydrogeological settings and they correspond to expected hydraulic behaviours to which water-mitigation measures can be adapted. They can therefore be used as 'rock classes' in the grouting design process (cf. Chapter 7.3).

Identification of the potential environmental impact could indicate the level of permissible tunnel inflow that will be stated in the environmental regulations for the project. Combined with the tunnel inflow estimate, which can be based on data from water wells accessed from national well archives, this gives an initial rough approximation of the level of sealing required. Tunnel inflow estimates can be carried out separately for the various hydraulic domains although separation must be done with caution to ensure each analysed area includes suitable quantities of input data.

6.1.2 Case study – Hallandsås project

The proposed method has been demonstrated with the Hallandsås project, which is a railway tunnel project currently being run in the province of Skåne in southern Sweden. The project involves the construction of two parallel tunnels, 8.7 km in length, through the Hallandsås horst with rock cover of 100-150 metres along most of the tunnel alignment. The study of the Hallandsås project was carried out using geological information available at the time of the original feasibility study, such as published geological descriptions, maps of bedrock and Quaternary deposits, well data and construction records from a nearby tunnel project. Some of the conceptualisations were confirmed by means of observations made during the construction phase of the project.

The tunnels are excavated through the NW-SE-oriented Hallandsås horst, which was formed as a result of recurring movements and deformations along a tectonic zone called the Tornqvist zone. The rock mass is, to a varying degree, heavily fractured with weathered rock masses preserved at considerable depths. The bedrock is composed predominantly of Precambrian gneisses as well as layers and dykes of amphibolite. There are also subordinate occurrences of Permian dolerite dykes. These rock mass settings were conceptualised into five hydraulic domains, presented in Figure 6.1, according to an expected difference in flow properties. Three of the domains described flow configurations in the host rock (HRD) and two in the deformation zones (HCD).

The geological history of the Hallandsås area revealed ground conditions and behaviours of importance for the planning of grouting, such as formation and reactivation of multiple fracture and fault systems, weathering processes and low stress regimes. Several ground behaviours were identified as unfavourable for grouting, including high groundwater pressures at tunnel depth, high-permeability zones, rock deformability and combinations of material-filled and open fractures. The heterogeneous and contrasting ground conditions imply that grouting requirements can vary significantly within quite short tunnel sections, making predictions less reliable.

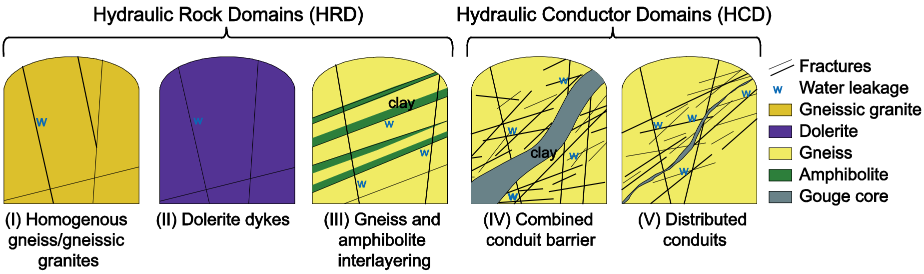


Figure 6.1 Visualisation of the hydraulic domains conceptualised for the Hallandsås site. There are three hydraulic rock domains, corresponding to various forms of hydraulic behaviour in the host rock, and two hydraulic conductor domains, which separate the deformation zones into barrier-conduit structures and conduit structures (cf. Figure 4.5). The host rock is expected to include 'localised conduits', i.e. single fractures that can be highly conductive.

Some of the above mentioned factors can be related to specific hydraulic domains, such as the conduit-barrier effect of dolerite dykes and deformation zones with well-developed gouge cores. The hydraulic domains could thus form a basis for the planning of various grouting designs, although this was not specifically carried out during the case study.

The estimation of tunnel inflows was carried out using flow log data from water wells located within two kilometres of the tunnel alignment. The wells were assigned to one of the two main types of hydraulic domains shown in Figure 6.1; hydraulic rock domain (HRD) or hydraulic conductor domain (HCD). The allocation was based on the positions of the wells in relation to deformation zones identified as lineaments and fracture zones on the structural geological map. Consequently, if a well emplacement coincided with a marked deformation zone, it was assigned to the hydraulic conductor domain. Otherwise it was assigned to the hydraulic rock domain.

The estimated median tunnel inflow based on well data from the Geological Survey of Sweden (SGU) greatly exceeded the levels normally permitted in Swedish rock tunnel projects, both in host rock and deformation zones, thus indicating a need for extensive water-mitigation measures. Some of the most important geological characteristics and key issues for grouting identified for the Hallandsås project are summarised in Table 6.1.

Table 6.1 A general work strategy for analysing engineering geological information for implementation of water-mitigating measures, along with an identification of key issues for the Hallandsås project.

MAIN CATEGORIES	SOURCE OF INFORMATION	IMPORTANT CHARACTERISTICS	KEY ISSUES FOR THE HALLANDSÅS PROJECT
Hydrogeological behaviour of the rock mass (Hydraulic domains)	Bedrock map with description, Engineering reports from previous construction activity, Well archive	Properties of rock types and contacts, Properties of deformation zones (HCD), Fracture network properties (HRD), Rock stresses	Large contrast in rock quality (short correlation lengths), Hydraulic barriers, Highly transmissive zones, Well-connected fracture networks, Open fractures and weathered, material-filled fractures, High groundwater pressures at tunnel depth, Low confining stresses, increasing the deformability of the rock mass (before and after excavation)
Tunnel inflow estimate	Well archive	Estimated median tunnel inflow for host rock and deformation zones	Estimate of the median inflows exceeding the generally permissible inflows
Environmental impact on the surroundings	Quaternary deposit map and hydrogeological map with descriptions, Topographic map	Soil types, Surface hydrology, Land use, Ecological systems	Varying conditions with both poorly sorted and well-sorted soils, Susceptible discharge areas, streams and wetlands, Rural area with private wells for water supply

6.1.3 Conclusions, case study Hallandsås

- The early prognosis of the Hallandsås project identified geological settings that indicated a need for systematic sealing measures and a complex grouting process for the Hallandsås project.
- Central input for the Hallandsås analysis was provided in the form of published material on tectonic development in the region and experiences from a nearby tunnel. This emphasises the importance of considering the geological and geomorphological history and reviewing previous construction activities.
- The early prognosis does not provide sufficient quantitative data for finalising site models or establishing detailed grouting designs. Further investigations are advisable to confirm the hydraulic domains and tunnel inflow estimates. However, the prognosis could outline geological settings that are crucial to the design and identify areas that require further investigation.

- The domain divisions aid the design planning process for favourable and unfavourable conditions, which is in line with the adoption of observational design methods.

6.2 Hydrogeological conceptualisation of the KBS-3H site

The KBS-3H case study exemplifies a hydrogeological characterisation and classification of a rock mass for grouting design. The study focuses on a conceptualisation of the flow properties of a hydraulically conductive fracture system, which is based on the hydraulic domain division presented in the Hallandsås study. The description is used to assess the suitability of various investigation methods and grouting designs and deals with information available from detailed investigations and production monitoring. The resulting conceptualisation forms a basis for discussing limitations and assumptions associated with hydraulic test methods and analytical models. The case study in its entirety is presented in Kvartsberg and Fransson (2013).

6.2.1 Method

The data studied are generated from investigations and grouting activities carried out at the KBS-3H experimental site, located in crystalline rock at the 220 m level at the Äspö HRL. The experimental site was used in 2004-2007 to demonstrate the technical feasibility of horizontal deposition of spent nuclear fuel and included practical trials of post-excavation grouting. The underground openings consist of one niche and two full, face-drilled, horizontal drifts, 1.85 m in diameter and 15 m and 95 m in length respectively. Various geological investigations and hydraulic tests have been carried out in boreholes and drifts during the demonstration phase, including cored borehole investigations, pre-grouting, control borehole tests and post-grouting.

The basis for the conceptual description of the water-conducting fracture system was hydrogeological characterisation based on a method presented by Hernqvist et al. (2012). In addition to the parameter characterisation, a stochastic model was adopted to predict fracture transmissivity distributions. Fracture transmissivity distributions are useful for developing grouting designs as they enable predictions of hydraulic apertures that need to be sealed and they form a basis for selecting suitable grouting materials (Gustafson, 2012). After grouting, they can be used to assess the sealing efficiency of the grouting.

Fracture transmissivity distributions assessed for the KBS-3H site were supplemented using a stochastic transmissivity model in which probability distributions were fitted to fracture counts (Negative Binomial distribution) and water pressure test data (Pareto distribution). This enables simulation of the transmissivity of a requested number of borehole intervals of a selected interval length. A Monte Carlo technique was then used to generate a probable transmissivity distribution for the simulated sets of intervals. An overview of the different steps in generating a simulated transmissivity distribution is given in Figure 6.2.

The simulated distribution could have a useful, predictive function in grouting design. However, in this study the model predicted drift section transmissivity based on borehole section transmissivity and provided input to assess the influence of various hydraulic test methods and test scales on inflow predictions. An underlying concept for its use was that the model's ability to reflect the observed transmissivity of the drift depended on the suitability of model assumptions and input data. If flow characteristics were not represented appropriately by assumptions and/or data input, this would show in the comparison between model and measured outcome.

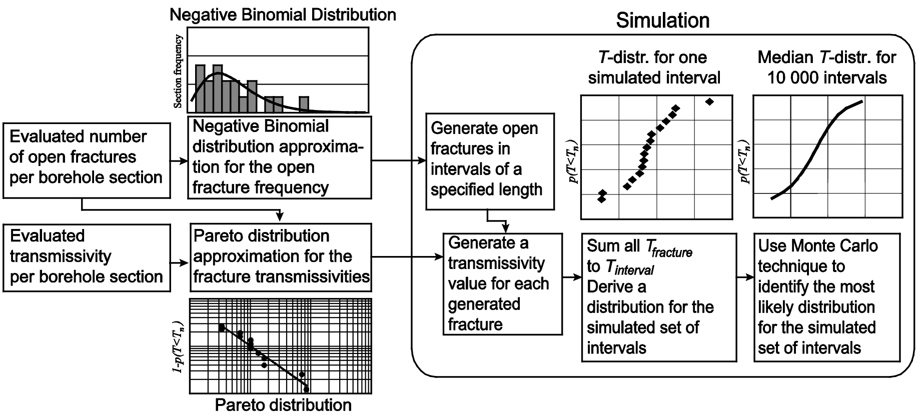


Figure 6.2 The structure of the stochastic model for simulating a fracture transmissivity distribution, here with data from the KBS-3H experimental site, simulated for 18 intervals.

6.2.2 Conceptualisation of the water-conductive fracture system

The hydrogeological characterisation of borehole and drift data at the KBS-3H site resulted in separation between conductive deformation zones (HCD) and the less conductive fracture system in the host rock (HRD), see Figure 6.3. The identification of deformation zones and host rock was achieved mainly through the estimated transmissivity and the flow dimension for the inflow structures.

Three prominent hydraulic conductors (minor deformation zones) were distinguished as being significantly more conductive and with a more continuous flow than the rest of the identified inflow structures. They were assessed as having 2D flow, corresponding to inflows of around 20 l/min, and were identified adequately using short-duration borehole tests. Selective pre-grouting in pilot boreholes was considered useful for sealing the zones.

The remaining inflow positions were considerably less conductive and were interpreted as belonging to a sparsely fractured network associated with flow constrictions. The main indicators for this were flow reductions without any apparent sealing, restricted flow redistribution after selective post-grouting, hydraulic anisotropy and a combination of assessed 2D flow and 1D flow within fractures. Some of these minor inflows were located in sections with high fracture frequency although they displayed dripping and discontinuous inflows that correspond better to a small-scale fracture network rather than the continuous inflows associated with more conductive structures. The flow configuration of less conductive parts of the fracture system was difficult to

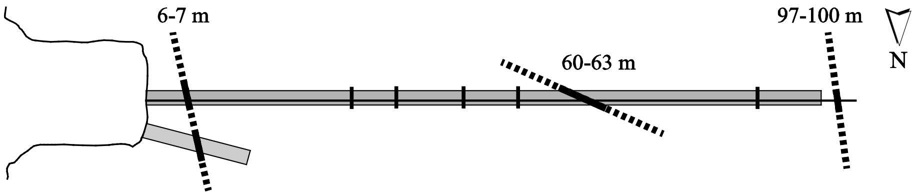


Figure 6.3 Plan view of the KBS-3H experimental site at the 220 m level, which consists of a niche and two drifts. Also shown is a previously drilled, cored pilot borehole. The figure shows the approximate positions of the hydraulically conductive structures along the borehole/drift. The features are separated into prominent hydraulic conductors (minor deformation zones), illustrated by thick black lines intersecting the drift(s), and less conductive structures belonging to the host rock fracturing (thin black lines).

capture using the borehole tests due to measurement limits and the difficulty for the relatively small boreholes to intersect hydraulically active parts of these fractures.

The stochastic model for fracture transmissivity distributions was applied to control borehole data for comparison with data from the excavated drift. The comparison between the modelled distribution and the transmissivity values calculated for the drift showed quite poor agreement, especially for the low transmissivity intervals, where the model clearly overestimated the fracture transmissivities. The simulated distribution showed better agreement for the more conductive features. This implies that the test method and model assumptions of fractures acting as parallel, independent conduits with 2D flow are more representative of the more conductive conduits in the rock mass. This is discussed further in Chapter 7.2.

6.2.3 Conclusions, case study KBS-3H

- The conceptualisation of the hydraulically conductive fracture system resulted in a subdivision between continuously flowing deformation zones (2D flow) and a less conductive network of 2D/1D flow fractures. The former are expected to be identified readily in pilot boreholes and tunnels, whereas the latter form a flow-constricted network that is less likely to be identified due to measurement limitations.
- Model assumptions of parallel, independent conduits with 2D flow were reasonable for larger hydraulic conductors. However, the modelled flow properties of the less conductive rock mass were clearly overestimated, which suggests that model assumptions and measured data failed to represent the flow-constricted fracture system adequately.
- The flow configuration, hydraulic test scales and test methods influence the reliability of analytical methods used for inflow and grouting predictions. Careful consideration must therefore be given to the choice of data acquisition method and the identification of prevailing flow properties at the site. It may be useful, for instance, to combine short-duration tests to obtain local fracture properties for grouting with transient tests in order to predict long-term inflows.
- The characterisation approach presented in this paper provides a useful basis for grouting design although the focus of this study was on the

assessment of grouting results rather than creating a detailed design. This approach can be used to identify the hydraulic character of the fractures that need to be sealed in order to meet specified requirements. It could therefore provide a basis for the selection of grouts and fan layouts.

7 DISCUSSION

Rock grouting design requires engineering geological information and conceptualisations that can differ from other design applications in tunnel construction. For instance, water-conducting fractures, e.g. local conduits, may result in high inflows but no mechanical instability. Conversely, a zone of clay-filled, dry fractures may be of importance for rock support but not for grouting. Grouting design should thus focus on the hydraulic behaviour of the water-conducting fracture system rather than use results from empirical design methods developed for rock support design.

Variations in information requirements for different design applications should be reflected in the classification of geological settings and grouting designs. Stille and Palmström (2003), Gustafson (2012) and others argue that classifications should be adapted to geological settings and requirements that are relevant to the engineering application under consideration in a particular project, i.e. that classification should be site-specific and problem-specific. This implies that engineering geological prognoses are preferably divided in a way so that they represent geological settings central to each specific engineering issue rather than bringing together all the geological information into one description. The prognoses should also be well developed in the early phases of a project when it is still possible to optimise tunnel alignments, construction methods and site investigations.

7.1 Engineering geological prognoses for grouting design

Engineering geological prognoses are considered useful in early project phases to identify critical design issues and reduce the risk of encountering unforeseen ground conditions in later phases, where they cause extra costs and delays, see e.g. Sturk (1998) and Baynes et al. (2005). However, early geological prognoses are not necessarily restricted to predicting difficult conditions. They should describe all the foreseeable conditions, both favourable and unfavourable, and form a basis for the range of designs needed in the project.

An approach to identifying ground conditions of importance for grouting in early project phases is suggested and exemplified in Chapter 6.1. The prognosis presented includes conceptualisation of the hydraulic behaviour of the rock mass at the project site, based on the geological and geomorphological history, the ground conditions, influencing processes and the tunnel layout. The identified behaviours are subdivided into hydraulic domains.

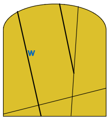
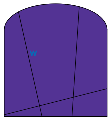
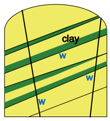
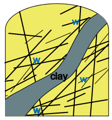
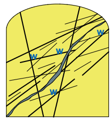
7.1.1 Subdivision of hydraulic domains

The hydraulic domains define formally various flow configurations within the rock mass. These may require different grouting strategies and grouting design can therefore be adapted to the hydraulic domains, which thus correspond to 'rock classes', cf. Chapter 7.3. The hydraulic domains have functions similar to 'reference conditions', presented in Chapter 3.3. The functions could include to establish design recommendations and to incorporate knowledge from similar geological units that occur outside the tunnel alignment. However, reference conditions are formulated for general engineering characteristics and the term 'rock class' is generally linked to rock support. The term 'hydraulic domain' signifies that the focus is on hydraulic properties.

The subdivision into hydraulic domains is based on identification of groups of geological units (e.g. rock types) that have similar hydraulic properties. A further subdivision of geological units is possible, as well as the merging of several geological units into one hydraulic domain if the hydraulic properties do not vary significantly. Ideally, the conceptualisation separates host rock and deformation zones according to the conceptual scheme in Figure 4.5. This is because deformation zones often display a complex structure and geometry that include increased alteration and fracturing compared to the surrounding rock. This can cause deformation zones to have mechanical and hydraulic properties that differ significantly from those of the surrounding rock masses, generally in terms of lower mechanical strength and higher permeability (Andersson et al., 2000).

A hydraulic domain division was exemplified in Chapter 6.1 using the Hallandsås project (summarized in Table 7.1). The division distinguishes between host rock and deformation zones and describes the flow configuration of the expected geological settings at the site. This qualitative conceptualisation was complemented with quantitative inflow estimation and both assessments indicated that extensive sealing would be necessary. This type of early assessment provides a basis for grouting design recommendations and it should be revised and updated when more information becomes available.

Table 7.1 Hydraulic domain division and inflow estimates for the Hallandsås project. It considers most of the parameters mentioned in STA guidelines (2011a): rock types, fracture system, deformation zones, fracture properties, weathering, occurrence of water, hydraulic head and fracture fillings.

	HYDRAULIC ROCK DOMAINS (HRD)			HYDRAULIC CONDUCTOR DOMAINS (HCD)	
INFLOW ESTIMATE (1 MPa)	120 l/min, 100 m			170 l/min, 100 m	
HYDRAULIC DOMAINS	 (I) Homogenous gneiss/gneissic granites	 (II) Dolerite dykes	 (III) Gneiss and amphibolite interlayering	 (IV) Combined conduit-barrier	 (V) Distributed conduits
FLOW CONFIGURATION	3D fracture system, well-connected. Occurrence of localised, highly transmissive conduits	As (I), also increased alteration and fracturing in contacts with surrounding rock, barrier effect	As (I), also weathered amphibolites and clay-filled fractures that create local barrier effects	Complex flow structure: gouge core (barrier) and damage zone (conduit, flow backbone)	2D structure, increased fracture frequency, altered rock, (conduit, flow backbone)
LARGE-SCALE FACTORS	Recurrent contrasts in rock quality, weathered rock mass, conduit-barrier effects, high groundwater pressures (possibly reaching around 1.2 MPa), low confining stresses with increased mobility of rock blocks				

The subdivision cannot be made without considering the concept of 'scale'. Scale can be defined as the size of the domain over which properties are averaged (Palmström and Stille, 2010). The fracture network and the hydraulic properties of the rock mass are scale-dependent, and the scale influences what features to include in the classification of deformation zones. In high-resolution descriptions minor deformation zones (localized conduits in Figure 4.3) may be presented as individual hydraulic conductors, whereas in low-resolution divisions these features would be included in the averaging of host rock properties between major deformation zones. Both divisions may be suitable, depending on the purpose and phase of the conceptualisation.

The need to deal with small-scale features in detailed grouting design is shown in a study of hydraulic conductors provided by Rhén and Forsmark (2000). They analysed data from fractures and deformation zones at the Äspö site that had an observed inflow rate exceeding 100 l/min, defined as High Permeability Features (HPF). They found that less than half of the HPFs were explained by prominent deformation zones and the remainder originated from a single fracture or just a few fractures, i.e., localised conduits. High permeability features thus exist as hydraulic conductors in the sparsely fractured rock mass and can be quite significant from a hydraulic point of view. However, HPFs that originate from single fractures cannot be readily observed before construction, which implies that grouting designs cannot be planned purely on the basis of major, distinct geological units. Observations during grouting are needed to decrease the scale and increase the resolution of the rock mass description.

7.1.2 Geological parameters of relevance to grouting design

The information needed to develop hydraulic domains includes various geological factors that influence the hydrogeological conditions, as described in Chapter 4. These factors consist of parameters that describe the geometrical characteristics of the water-conducting fracture system, as well as geological processes that influence the hydrogeological behaviour, e.g., mechanics, geochemistry and hydrology. The geological and geomorphological history of the area is also considered useful as it may reveal complex and difficult geological settings that are not obvious from investigations and observations. A process understanding of the way in which features are formed also facilitates predictions and extrapolations of geometrical possibilities of, for instance, deformation zones.

A compilation of geological parameters that can be relevant to consider in early engineering geological prognoses is presented in Table 7.2. The parameters are not of equal importance in every project. It may be possible to remove some parameters from further consideration whereas other geological factors may need to be added to the list. Information about certain parameters may also be missing during early phases and will need to be considered in later project phases when more information becomes available. Site investigations for grouting should therefore be planned to provide geological information that is considered relevant and representative of the tunnel depth and the range of expected settings. This means that data collection should not focus purely on areas with expected poor rock quality.

Table 7.2 Geological parameters and comments on their influence on grouting design. These parameters could be important to bear in mind when investigating key issues for grouting.

	GEOLOGICAL PARAMETERS	INFLUENCE ON GROUTING DESIGN AND WATER INFLOW
LARGE-SCALE FACTORS	Rock types, rock contacts, dykes, weathering	Controls the geometry of the fracture systems and fracture properties. Dykes and contacts can be associated with variations in fracturing.
	Topography	Provides a geometric framework for water balance calculations and deformation zone identification.
	Rock stresses	Control the opening of fractures and thus the geometry of the water-conducting fracture system. Influence deformability of fractures during grouting.
	Hydraulic conductivity	Enables assessment of water inflows and indicates grouting difficulty.
	Hydraulic head	Affects water inflows and grouting pressures and is needed to assess hydraulic apertures.
	Hydraulic gradient	Influences the risk of backflow and grout erosion during grouting.
	Groundwater recharge	Needed to assess water balance, which affects water inflow and drawdown recovery.
	Groundwater chemistry	Affects grout degradation.
HOST ROCK (HRD)	Hydraulic aperture/transmissivity	Enables assessment of water inflow, grout penetrability and grout spread, which form the basis for grouting design.
	Other fracture properties: open/sealed fractures, fracture roughness, fracture fillings/alterations, flow dimension	Affect flow paths and thus grout spread within fractures.
	Fracture system properties: fracture frequency, orientation and number of fracture sets (anisotropy), flow dimension	Affect network connectivity and thus grout spread and water inflow.
DEFORMATION ZONES (HCD)	Hydraulic aperture/ transmissivity	Enables assessment of water inflow, grout penetrability and grout spread, which form the basis for grouting design.
	Thickness, orientation	Controls the length of tunnel intersection and thus grout fan length and grout hole orientations.
	Composition: core/damage zone	Affects connectivity and flow anisotropy within and across the zone.

7.2 Detailed characterisation for grouting design

During the detailed design process, it remains important to focus on the geological settings and use new information to develop a more detailed description of the water-conducting fracture system. The case study of the KBS-3H site provides an example of detailed characterisation and conceptualisation of a rock mass volume for grouting design. The focus of the study was evaluation of grouting results although the approach applied could be useful for identifying the hydraulic character of the fractures that need to be sealed. The stochastic model of fracture transmissivity distributions, for instance, is scalable and it is possible to use short-interval tests to determine the likely distribution of interval transmissivities for longer intervals, such as the length of grouting fans.

The stochastic fracture transmissivity distributions of the pre-grouted rock at the KBS-3H site were also analysed in the light of the suitability of the analytical model to simulate fracture transmissivities. The simulated transmissivity distribution (black line in Figure 7.1) is based on fracture counts and section transmissivities calculated from a borehole drilled before excavation (described in Chapter 6.2). The simulation demonstrated its credibility by reproducing the pattern of the transmissivity data from which it was derived (black dots in Figure 7.1). The simulated distribution was also compared to transmissivity distribution derived from measured inflows in the excavated drift (crosses in Figure 7.1).

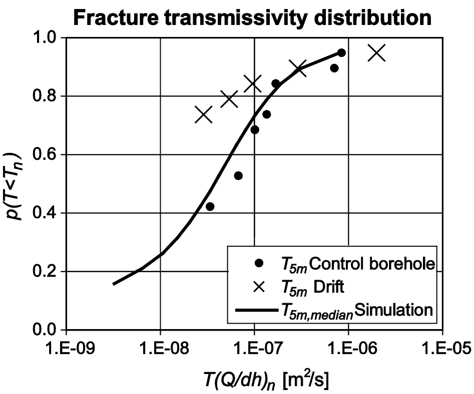


Figure 7.1 Cumulative distributions of fracture transmissivity data evaluated after pre-grouting, all presented for 5 m sections. The black dots represent the specific capacity calculated from the control borehole; the crosses represent specific capacities calculated from the excavated drift. Measurement limits provide censoring of the input data. The black line represents the median T(Q/dh)values for the simulated fracture transmissivity distribution.

The simulation and the outcome in the drift should preferably concur since they are assumed to represent the same rock mass, although one originates from 5 m sections of a borehole and the other from 5 m sections of drift with a diameter of 1.85 m. However, a visual comparison of these three sets of data shows that only the larger inflows (most transmissive sections) were represented reasonably in the simulation. The minor inflows were overestimated.

It is suggested that this divergence between prognosis and outcome is the result of assumptions inherent in the input data and that the mathematical model is not representative in describing the flow properties of fractures in the drift. The input data for the simulation consisted of specific capacities calculated from short-duration water pressure tests and these do not take into account whether or not the open fractures are interconnected with the surrounding network or if hydraulic chokes impede groundwater flow at a further distance from the borehole. Measurement limitations also impeded investigation of the smaller inflows, which made descriptions of small inflow less reliable than large inflows. Moreover, the model is based on assumptions that fractures act as parallel, independent conduits with 2D flow. Consequently, the stochastic model describes local fracture transmissivities that do not necessarily correspond to the effective flow properties of the less transmissive parts of the rock at the KBS-3H site. The simulated distribution showed better agreement for the more hydraulically conductive features. This indicates that investigation methods and model assumptions are more representative for the more conductive structures.

The simulation discrepancy highlights differences between the flow configuration of larger, more continuously flowing conductors and that of the small-scale, flow-constricted host rock. The stochastic model for fracture transmissivity distributions could therefore be used to distinguish between hydraulic conductors (HCD) and host rock (HRD). It is assumed that the conductors are represented by transmissivities in the upper range of the cumulative distribution, whereas water-conducting fractures in the host rock are represented by the lower range of transmissivities. The boundary between the HCD and HRD is probably site-specific; at the KBS-3H site the division between conductors and host rock was shown to be a difference between features with inflows of around 20 l/min, and the less transmissive features.

The Hallandsås project was investigated with less detailed information. Consequently, the subdivision between host rock and deformation zone was based mainly on a qualitative description of conduit and barrier-conduit behaviour. Highly transmissive features in the form of localised conduits are almost certainly included in all host rock domains defined for the Hallandsås project although the limitations of the input data do not enable their deterministic identification. The scale of interest and the measurement limitations are two aspects that influence how the defined division between HRD and HCD can be made.

7.3 Grouting design classes

The pre-defined grouting designs constructed for Swedish tunnel projects are generally adapted to varying requirements along the tunnel alignments, e.g. in terms of permissible inflow, rock cover or underground structures located nearby. There are also certain adjustments of the grouting design to varying ground conditions. This adaption is often expressed in general terms as a modification of the grouting design in tunnel sections with low rock cover, low hydraulic head or highly transmissive features. The terms 'fracture zones' and 'deviating conditions' were stated for some of the investigated tunnels in Chapter 5.3 but were poorly defined in terms of:

- How they are expected to behave hydraulically
- How frequent and for how long stretches they are expected to occur, and
- How the grouting design is modified when encountering them.

Such information gaps could make cost estimates uncertain and consequently increase the risk of misclassification (i.e. choices of inappropriate designs) and unexpected amounts of re-grouting.

Ideally, preparation of designs is based on the range of expected ground conditions, favourable and unfavourable, and their implementation should be based on rules defined using appropriate, observable indicators. Water pressure tests provide easily observable parameters although they may be of limited value if nothing is known about the fracture set-ups that result in the water losses. Conceptualisation of the water-conducting fracture system could increase the value of the water pressure test, e.g. indicate whether one fracture is expected to dominate the grout take or if several smaller fractures need to be sealed. This implies that calculations of the distribution of hydraulic apertures could be more useful than Lugeon values. An example of this is provided in the TASS Tunnel project (Funehag and Emmelin, 2011) where selection of grout type was based on the appropriateness of different grouts for the hydraulic apertures assessed at the site during excavation.

Sometimes, one standard (basic) grouting design is established from the general rock mass conditions at the site. Some of the tunnel projects presented in Chapter 5.3 indicated that the standard design often corresponds to favourable conditions although the design is robust and can disregard smaller, local variations in hydrogeological conditions. Nevertheless, preparations to extend the number of grouting holes are made for less favourable geological settings. This prevents the standard design from becoming overly conservative. However, the word 'standard' can be ambiguous since it indicates that there is a deviating, less frequent grouting design that can be applied besides the standard design. Estimates of their expected frequency should preferably be made, but the terminology can be confusing if the 'deviating' design is equally probable or more probable than the standard. An alternative is to list the designs without any relative linguistic weighting, e.g. 'Grouting design A, B…'.

The expected range of geological settings can be separated using hydraulic domains and, together with the stated requirements, these domains form a suitable basis for establishing grouting design recommendations. The hydraulic domains thus correspond to different rock classes for grouting design. This can be exemplified by the domains developed for the Hallandsås project and the KBS-3H experimental site at Äspö HRL, see Figure 7.2.

The conceptualisation of the geology in the Hallandsås project resulted in several hydraulic conductor domains and hydraulic host rock domains with pre-grouting measures assessed to be necessary for all of them. The most favourable conditions for the Hallandsås project would be represented tentatively by some of the more homogeneously fractured host rock domains, whereas more heterogeneous host rock and barrier-conduit zones could represent unfavourable conditions. Evaluation of the grouting results at the KBS-3H site indicated that differing grouting designs may be required to achieve sufficient sealing, depending on the strictness of the sealing requirements applied to the tunnel, see Figure 7.2.

The use of hydraulic domains can facilitate the preparation of pre-defined grouting classes and contingency measures for both favourable and unfavourable scenarios, which is in accordance with the observational method defined in Eurocode 7. However, the selection of a design class is also a result of requirements and layouts, and different hydraulic domains can correspond to the same grouting design class. Nevertheless, various hydraulic domains are defined because they are expected to act hydraulically different, and the selected design class should therefore be considered suitable for the varying hydraulic behaviours it may be chosen for.

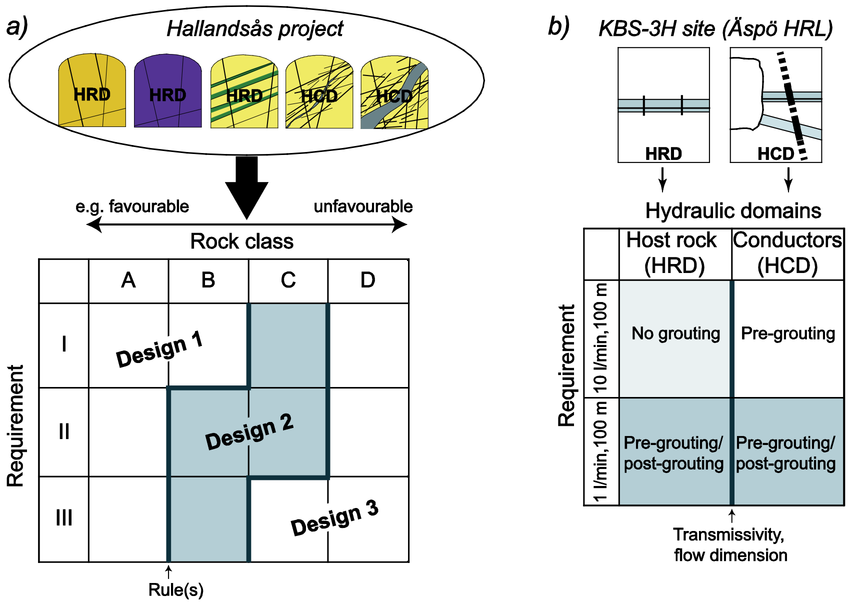


Figure 7.2 a) Hydraulic domains of the Hallandsås project could correspond to various rock classes presented in the design matrix from Figure 3.4. The design does not necessarily need to differ for the various domains; design robustness and requirements influence the design range. The rule corresponds to the indicating parameter(s) for the design class.
b) An example of how the KBS-3H experimental site could be adapted to the matrix. If the requirement is less strict, pre-grouting of the hydraulic conductors is likely to be sufficient for the KBS-3H drifts. On the other hand, if the requirement is strict, post-grouting is likely to be required for both domains. Parameters calculated from hydraulic tests in a pilot borehole are suggested as the indicator for the different domains.

8 CONCLUSIONS

The general aim of this work was to clarify the handling of engineering geological information in rock grouting design and suggesting improvements in the conceptualisation and classification of geological settings. The conclusions of this study are related to three main areas:

- Engineering geological prognoses and hydraulic domains developed specifically to benefit grouting design.
- Assessment of sealing efficiency and characterisation of hydraulic conductors using fracture transmissivity distributions.
- Classification of engineering geological information for grouting design and the use of hydraulic domains to subdivide the expected range of geological settings.

Grouting design has key issues that may differ from other design applications in underground construction. Accordingly, an engineering geological prognosis used for grouting design should focus on geological parameters that can increase understanding of the flow behaviour of the rock mass. Ideally, identification of key issues, such as potential tunnel inflows, should already have been identified in the early project phases based on information available from geological databases and previous construction activities. The early focus on the geological and geomorphological history of the site reduces the risk of encountering unforeseen issues for grouting design in later phases. A suggested approach to early engineering geological prognoses has been applied to the Hallandsås project, exemplifying how information about geological settings can be structured to be useful in the design of water-mitigating measures.

Hydrogeological characterisation and the implementation of a fracture transmissivity distribution model can be used to evaluate the efficiency of grouting measures (Kvartsberg and Fransson, 2013). Moreover, these analyses can identify differing flow configurations in the rock mass, which is useful for grouting design since the flow behaviour of the hydraulic conductors affects how they should be grouted efficiently and their inflows correctly predicted. The analysis of the flow configurations of the rock mass at the KBS-3H site indicated limitations in the chosen analytical model and the data acquisition method. An understanding of the flow configuration within the rock mass is therefore useful for assessing whether the chosen design values and analytical models are suitable.

Classification of pre-defined grouting designs in Swedish tunnel projects could benefit from a better definition of the geological settings than is common practice at present. It would be an advantage if grouting were adapted to the range of expected ground conditions, both favourable and unfavourable. Construction documents should also state how the expected conditions will behave hydraulically, how frequently and how long sections the various conditions are expected to occur, and how the grouting design is modified when certain conditions are encountered. There must also be appropriate, observable parameters that can identify the various conditions during construction, such as specific capacity, hydraulic apertures and flow dimension.

The hydraulic domains formally distinguish geological units with separate flow configurations. They provide a structure for categorising expected geological settings and they can therefore be used for grouting class division. Preparation of grouting designs adapted to the various expected hydraulic behaviours follows the requirements stated in Eurocode 7 for rock construction design according to the observational method. It is suggested therefore that the hydraulic domains facilitate implementation of the observational method. However, it is important to remember that there will always be uncertainties inherent in the ground conditions and a lack of geological understanding and incorrect interpretations could have a negative effect on engineering analyses and calculations. Grouting design should therefore be iterative and conceptualisations and calculation models should be examined carefully throughout the construction process.

8.1 Future research, development and demonstration

The work presented in this report presents opportunities for further development of descriptions of various flow configurations, engineering geological prognoses and classification of grouting designs. The suggested approach to establishing early geological prognoses and hydraulic domains should be applied in ongoing tunnel projects to further investigate its usefulness.

The study of the KBS-3H site showed that it was difficult to predict and measure the flow configuration of the sparsely fractured, flow-constricted host rock. The low-transmissive host rock is not a concern in all underground constructions although small inflows become more important if inflow requirements become stricter. In those cases, there is a need to develop investigation methods and calculation models that represent more accurately the hydraulic properties of the low-transmissive parts of the rock.

The importance of considering the validity of mathematical models and input parameters for inflow analyses have been brought up for discussion. The analysis of the sparsely connected, flow-constricted rock mass at the KBS-3H site indicated limitations in the chosen analytical model and the data acquisition methods. Experiences of limitations in analytical inflow models and investigation methods have also been identified at the well-connected rock mass at the Hallandsås site, which represents the opposite extreme of flow configurations. Identification of hydrogeological conditions that are suitable for applying common analytical inflow models and investigation methods could be therefore be systematised further. The effect of model and parameter choices on geotechnical uncertainties could also be investigated further, e.g. using sensitivity analyses.

It should be possible to define the expected ground conditions for grouting design more clearly. Further investigations are advisable in order to identify suitable statistical analyses to predict the frequency and continuity of various geological settings (i.e. hydraulic domains), which are useful for estimating the number of designs. Quantitative handling of uncertainties facilitates work with a risk-based approach.

Finally, there has been increased concern that anticipated behaviours in rock design should be described more precisely than is currently the case. The use of site-specific and issue-specific domains is suggested to provide a basis for a clear description of expected ground behaviour for various design applications, i.e., also excavation techniques, rock support and environmental impact. Domain division for other design applications needs further consideration. Nevertheless, a subdivision of the geological settings similar to the colour sketches presented for Hallandsås in Table 7.1 could provide a suitable basis in early project phases also for other design problems.

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