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STIFTELSEN BERGTEKNISK FORSKNING
ROCK ENGINEERING RESEARCH FOUNDATION

APPLICATION OF DRILLING MONITORING PARAMETERS IN TUNNELLING – WITH FOCUS ON EXCAVATION DAMAGE, ROCK MASS CHARACTERISATION AND ROCK SUPPORT DESIGN

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Användning av borrparameterna i tunneldrivning - med fokus på sprängskador, bergmassakarakterisering och bergförstärkningsdesign

Jeroen van Eldert

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PREFACE

The project "Analysis of damage zones development depending on geology with the application of MWD technology for the prognoses of the extent in the remaining rock mass" was part of a PhD project at the Luleå University of Technology.

The study focussed at the possibility to predict blast damage and rock support requirements based on MWD. It showed the good possibilities to apply MWD in tunnelling, especially for rock support and characterising the rock mass more detailed. This work provides a framework for MWD data and its usage in practice for rock support installation. It takes the first steps towards correlating blast damage and rock mass properties based on MWD data. The work in this project was funded by the Rock Engineering Research Foundation (BeFo) and Swedish Blasting Research Centre (Swebrec).

The work in this project was performed by Jeroen van Eldert (LTU) under supervision of Håkan Schunnesson (LTU), Daniel Johansson (LTU) and David Saiang (LTU). The project was supported by the reference group consisting of: Urban Åkeson (Swedish Transportation Administration), Hans-Åke Mattsson (ÅF), Mats Olsson (EDZ consulting), Robert Sturk (Skanska), Rolf Christiansson (Swedish Nuclear Fuel and Waste Management Co), Johan Jonsson (Epiroc) and Per Tengborg (BeFo).

Stockholm

Patrik Vidstrand

FÖRORD

Projektet ”Analys av skadezonens utbredning beroende på geologi samt tillämpning av MWD- teknik för att prognosticera dess omfattning i kvarvarande berg” utgör en del av ett doktorandprojekt vid Luleå Tekniska Universitet.

Studien har fokuserat på att se möjligheterna med att prognostisera sprängskador och bergförstärkningsbehov genom att tolka borrningsparametrar som MWD. Studien har visat att det finns goda möjligheter för att tillämpa MWD i tunneldrivning, speciellt med avseende på bergförstärkning och möjligheten att karakterisera bergmassan mer detaljerat. Rapporten ger ett ramverk för användning av MWD data i framtiden, speciellt i bergförstärkningsprocessen. Därtill, har arbetet visat första steget för att korrelera sprängskador med MWD. Projektet har finansierats av Stiftelsen Bergteknisk Forskning (BeFo) och Swedish Blasting Research Centre (Swebrec).

Arbetet i projektet var utfört av Jeroen van Eldert (LTU) under handledning av Håkan Schunnesson (LTU), Daniel Johansson (LTU) och David Saiang (LTU). Projektet har haft stöd av en av en referensgrupp med följande deltagare: Urban Åkeson (Trafikverket), Hans-Åke Mattsson (ÅF), Mats Olsson (EDZ consulting), Robert Sturk (Skanska), Rolf Christiansson (SKB), Johan Jonsson (Epiroc) och Per Tengborg (BeFo).

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ABSTRACT

Before underground excavation, a site investigation is carried out. This includes reviewing and analysing existing data, field data collected through outcrop mapping, drill core logging records and geophysical investigations. These data sources are combined and used to characterise, quantify and classify the rock mass in order to design the tunnel and select the excavation method.

Despite the care taken a site investigation cannot reveal the required level of detail. Gaps in information might become significant during the actual construction stage. This can lead to; for example, over-break due to unfavourable geological conditions. In addition, an underestimation of the rock mass properties can lead to unplanned stoppages and tunnel rehabilitation. The excavation method itself, in this case, drill and blast, can also cause severe damage to the rock mass. This can result in over-break and reduction of the strength and quality of the remaining rock mass. Both pose risks for the tunnel during excavation and after project delivery.

Blast damage encompasses over-break and the creation of an Excavation Damage Zone (EDZ). Irreversible changes occur within the remaining rock mass inside the EDZ, physically manifested as blast fractures. This report investigates a number of methods to determine blast damage in two ramp tunnels of the Stockholm bypass. It compares the most common methods of blast damage. It uses the comparison to select the most suitable method for blast damage investigation in tunnelling, based on the environment and the available resources. The study applies Ground Penetrating Radar, core logging (for fractures) and P-wave velocity measurements to determine the extent of the blast damage in the two ramp tunnels, the SKB's TAS04 and the Veidekke access tunnel.

The study of the two tunnels in the Stockholm bypass showed a significant overestimation of the actual rock mass quality during the site investigation. In order to gain a more accurate picture of the rock mass quality, Measurement While Drilling (MWD) technology was applied. The technology was investigated for its ability to predict rock mass quality, quantify the extent of blast damage, and forecast the required rock support. MWD data were collected from both grout and blast holes. These data were used to determine rock quality indices e.g. Fracture Indication and Hardness Indicator, using the MWD parameters. The Fracture Index was then compared with the installed rock support at the measurement location.

Lastly, the study evaluated if the MWD parameters could forecast the extent of the blast damage zone. The study clearly showed the capability of MWD data to predict the rock mass characteristics, e.g. fractures and other zones of weakness. It demonstrated that there is a correlation between the Fracture Index (MWD) and the Q-value, a parameter

widely used to determine the required rock support. It also found a correlation between the extent of the blast damage zone, MWD data, design and excavation parameters (for example, tunnel cross section and charge concentration).

Keywords: Blast damage, Excavation Damage Zone, EDZ, Measurement While Drilling, MWD, Rock support, Rock mass characterisation, Tunnelling

SAMMANFATTING

Innan byggandet av berganläggningar behöver en förstudie genomföras. En sådan förstudie inkluderar analys av tidigare data, fältdata som till exempel bergmassakartering, kärnborrning och geofysikaliska mätningar. Analyserna används för karakterisering, kvantifiering och klassificering av bergmassan där till exempel en tunnel ska drivas. Till detta tillkommer valet av tunneldrivningsmetod.

Förstudier idag har dock inte alltför stor detaljnivå. Informationsluckor som kan bli signifikanta under själva tunneldrivningen, som till exempel resulterar i överberg på grund av dåliga bergförhållanden. Underskattning av bergförhållanden kan leda till oplanerade avbrott, extra arbetsinsatser och ökade kostnader. Drivningsmetoden, som i detta fall var borrning och sprängning kan orsaka allvarliga skador (sprängskador) i kvarstående berg vilket resulterar i överberg, minskad hållfastighet och sämre kvalitet av kvarstående bergmassa. Både dåliga bergförhållanden och sprängskador utgör risker under tunneldrivning och efter det att tunneln färdigställts.

Denna rapport undersöker ett antal metoder för att fastställa sprängskador i två ramptunnlar i projektet Förbifart Stockholm. I studien har de mest vanliga och mest lämpliga metoderna använts för att fastställa sprängskador. I denna studie användes markradar, kärnkartering (för sprickor) och P-vågs mätningar för att fastställa omfattningen av sprängskador i den två ramptunnlarna, SKBs TAS04 tunnel och Veidekkes tillgångstunnel.

Studien har visat en signifikant överskattning av bergmassas kvalitet i förstudier från den två ramptunnlarna i projektet Förbifart Stockholm. För att få en mer detaljerad bild av bergmassan, har mätning av borrparameter (Measurement While Drilling, MWD) utvärderats. Studien har visat att MWD har potential att förutspå omfattningen av sprängskadezonen. Arbetet har även visat att MWD-tekniken kan förutse bergmassklassificering, till exempel sprickor och svaghetszoner. Ett samband har även observerats mellan sprickindex (genererat från MWD-data) och Q-värde, som används att bestämma behovet av bergförstärkning längs efter tunneln.

Nyckelord: Sprängskador, Sprängskada zon, Borrparameter tolkning, Bergförstärkning, Bergkarakterisering, Tunneldrivning

CONTENTS

GLOSSARY

AMA	Allmän Material- och Arbetsbeskrivning
BeFo	Stiftelsen Bergteknisk Forskning (Swedish Rock Engineering Research Foundation)
CMS	Cavity Monitoring System
DC	Drill Core
DxM	Dynomex (Dynamite)
EDZ	Excavation Damage Zone
ESR	Excavation Support Ratio
HCF	Half Cast Factor
HRL	Äspö Hard Rock Laboratory
Ja	Joint Alteration Number
Jn	Joint Set Number
Jr	Joint Roughness Number
Jw	Joint Water Parameter
MPES	Mine Planning and Equipment Selection
MRGIS	Mine Roof Geological Information system
MWD	Measurement While Drilling
DC	Diamond Coring
GPR	Ground Penetrating Radar
GSI	Geological Strenght Index (Hoek and Brown, 1997)
MLR	Multiple Linear Regression
RMR	Rock Mass Rating (Bieniawski, 1973)

RPM	Rotations per Minute
RQ	Research Question
RQD	Rock Quality Designation (Deer, 1964)
PCA	Principle Component Analysis
PPV	Peak Particle Velocity
Q	Rock Mass Quality (Barton et al., 1974)
UM	Epiroc (former Atlas Copco) Underground Manager
S	Selective bolting
SGU	Sveriges Geologiska Undersökning (Swedish Geological survey)
SC	Sprayed Concrete
SKB	Svensk Kärnbränslehantering (Swedish Nuclear Fuel and Waste Management Co.)
SRF	Stress Reduction Factor
Swebrec	Swedish Blasting Research Centre
UCS	Uni-axial Compressive Strength
WTC	World Tunnelling Congress

1 INTRODUCTION

Several large tunnelling projects are being initiated or are under construction in Sweden, e.g. SKB's Spent Fuel Repository in Forsmark and infrastructure development (e.g. subway extensions, Gothenburg's western link, Stockholm bypass). These tunnelling projects require significant investments, so the projects must be well-prepared. Despite the best efforts to thoroughly characterise the excavation sites, the projects often encounter challenging ground conditions quite unexpectedly during construction (Wahlström, 1964; U.S. National Committee on Tunneling Technology, 1984; Kovári and Fechtig, 2000; Kjellström, 2015). These challenges arise from the fact that it is not feasible to provide complete and highly accurate information about the ground conditions. Hence, in Sweden, or Scandinavia in general, making continuous efforts to forecast the ground condition ahead of a tunnel during construction is often a requirement set by clients, e.g. Swedish Transport Administration and municipalities. This is implemented, for example, through continuous geotechnical mapping of tunnel walls, probe and core drilling and drill data acquisition and analysis. The latter is commonly referred to as Measurement While Drilling (MWD).

The selected construction method affects the near-field rock mass around the tunnel. In hard rock mass conditions as in Scandinavia, the preferred excavation method is drill and blast. Besides being a cost effective method, it also provides a high level of flexibility. A major side effect of drilling and blasting is that it introduces excavation damage outside the intended tunnel perimeter. The key components of this damage, as illustrated in Figure 1.1, are over-break and an Excavation Damage Zone (EDZ). Over-break results in an irregular tunnel contour and additional material haulage with additional costs. Similarly, the presence of a damage zone affects the long term stability of the tunnel, along with requirements for appropriate ground support. In Sweden the AMA anläggning 17 (allmän material- och arbetsbeskrivning för anläggningsarbeten) (Svensk Byggtjänst, 2017) sets the theoretical limits for the extent of the blast damage zone, as shown in Table 1.1. The criteria are based on the relationship between the amount of Dynomex (DxM or dynamite) per metre and the expected blast damage. In practise, it is difficult to relate these criteria for blasting in varying rock mass conditions.

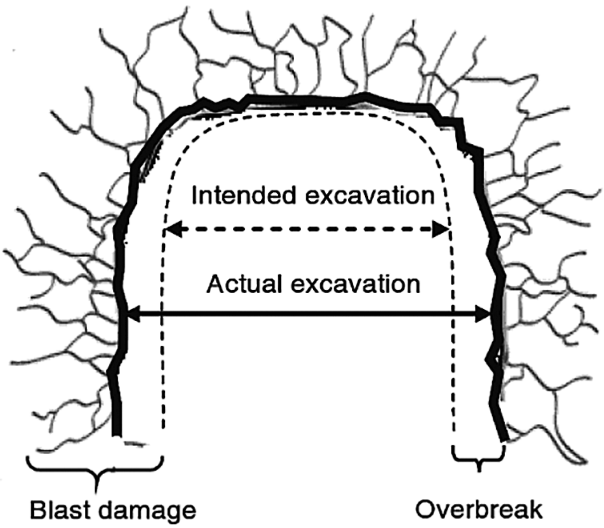


Figure 1.1 Blasting induced damage and over-break outside the intended tunnel perimeter (Warneke et al., 2007).

Table 1.1 Theoretical damage zone in relation to the charge concentration in AMA17 (after AMA17 anlägning, Svensk Byggtjänst, 2017).

Theoretical damage zone	Charge concentration DxM (kg/m)
0.2	0.10
0.3	0.15
0.4	0.20
0.5	0.25
0.6	0.30
0.8	0.40
1.0	0.55
1.1	0.70
1.2	0.75
1.4	1.00
1.6	1.20
1.8	1.40
2.0	1.60
2.2	1.80
2.4	2.00

To comply with the required theoretical limits of the extent of the damage zone while achieving a smoother tunnel contour (i.e. reduce over-break) high-tech drilling and charging technologies are used in Scandinavian tunnelling projects. This type of equipment has the possibility to record and optimise operational performance, e.g.

explosive charge per hole (charging equipment), drill hole deviation and rock characterisation with Measurement While Drilling (MWD) (drill rig). The data acquired from the drill rig via the MWD database can be used to calculate the Fracture Index, Hardness Index and Water Index of the rock mass. These calculations are based on the measured operational data during drilling and their variations along the hole (Schunnesson, 1996; Schunnesson, 1998; Schunnesson et al., 2011; Epiroc, 2018b). These indices have been used to validate and re-characterise the rock mass in several tunnel projects (Humstad et al., 2012; Bever Control, 2015).

In tunnelling, encountering bad ground conditions, which are often coupled with extensive blast damage, leads to construction delays and ultimately to cost overruns. Extensive grouting (injection of cement into drill holes to seal the surrounding rock mass) is necessary in bad ground conditions, as is increased rock support. The consequences have been discussed in earlier research, including Panthi and Nilsen (2007), Kim and Bruland (2009), and Saiang and Nordlund (2009). Accurate predictions are imperative for optimisation in tunnelling.

1.1 Problem Statement

Today in Sweden, the regulation on the extent of blast damage is solely theoretical; it does not incorporate existing rock mass conditions into the assessment. The applied theory is based on Holmberg’s research (1978). Thus, the actual blast damage is not measured and therefore is unknown and in practice not verified. Moreover, in most tunnelling projects, there is a limited knowledge of the actual rock mass conditions ahead of the face. Therefore the rock support design procedure is often sub-optimal. Ultimately, the lack of knowledge on the rock mass conditions ahead of the face can cause delays and lead to increased excavation costs (Wahlström, 1964; U.S. National Committee on Tunneling Technology, 1984; Kovári and Fechtig, 2000; Kjellström, 2015).

1.2 Purpose and Objectives

This report investigates a number of methods for quantifying the extent of blast damage, focusing on the usage of drill monitoring data to assess rock mass conditions ahead of the face. In the best case scenario, the acquired knowledge on the rock mass conditions may be employed to optimise the rock support design and to predict the extent of the blast damage in the remaining rock.

1.3 Research Questions

To fulfill the purpose of the report, the following research questions (RQs) were formulated:

RQ1 How can the extent of excavation damage be measured?

RQ2 How can drill monitoring data be used for rock mass quality assessment?

RQ3 How can rock mass characterisation based on drill monitoring be used to improve the rock support design process?

RQ4 To what extent can excavation damage be predicted by using rock mass characterisation based on drill monitoring?

Additional to this report Van Eldert (2017) discusses methods employed in blast damage investigation. Selected investigation methods (Ground Penetrating Radar, core drilling and P-wave velocity measurements) were applied to investigate the extent of the EDZ in Van Eldert et al. (2016) and Van Eldert et al. (2018b). Van Eldert et al. (2017) gives a historical over-view of MWD technology and its applications today. Van Eldert et al. (2018a) investigates the differences and similarities between grout and blast hole MWD. Van Eldert et al. (2018a) also presents a case study of the application of MWD technology for validation and re-characterisation of the rock mass in a tunnelling project. Van Eldert et al. (2016) investigates the usage of MWD data to predict blast damage at one site. Van Eldert et al. (2018b) extends the findings in Van Eldert et al. (2016) and correlates MWD parameters with the measured EDZ at two additional sites. In addition, Van Eldert et al. (2016) and Van Eldert et al. (2018b) address the influence of the rock mass on MWD parameters and the excavation damage.

2 METHODOLOGY

The study focused on the application of production data (i.e., MWD data) to characterise and predict blasting induced damage and ultimately support predictions based on the rock quality assessment, as visualised in Figure 2.1. First, a literature review was conducted of studies on blast damage investigation and MWD technology; this was by a limited practical study. Based on the findings, the extent of blast damage in the tunnels was investigated with Ground Penetrating Radar (GPR), core drilling and P-wave velocity measurements. The findings of these investigations were statistically (Multiple Linear Regression) compared with the collected MWD data and excavation data (charge concentration, rock cover and tunnel cross section). Lastly, the Fracture Index was analysed to see if it could predict the Q-value and rock support requirements.

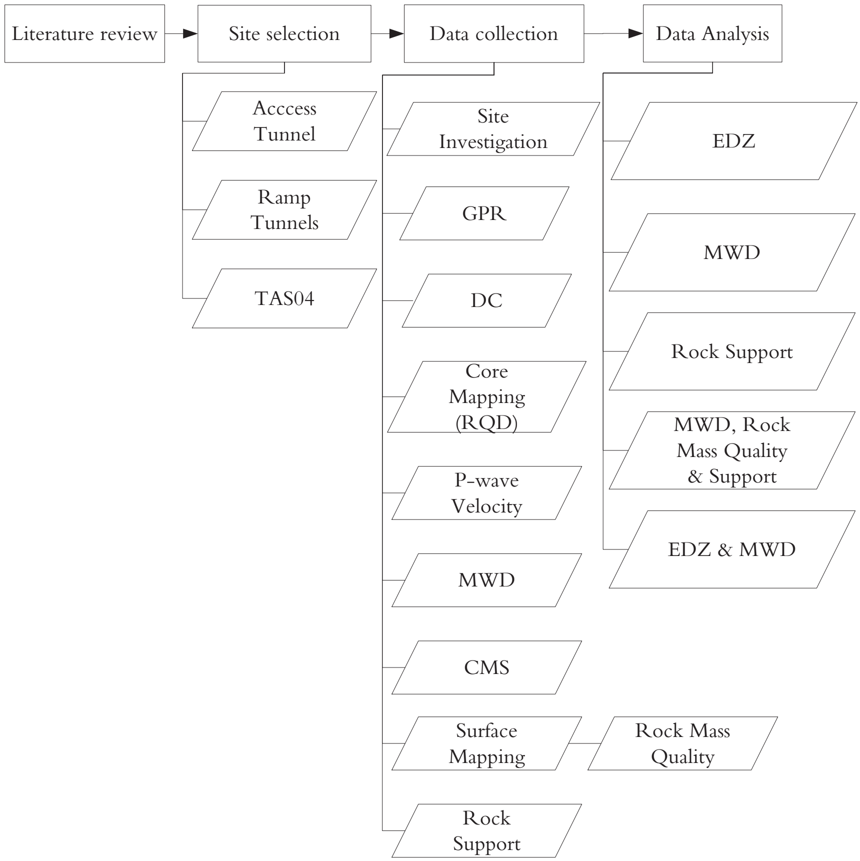


Figure 2.1 Methodology to study excavation damage in tunnelling.

2.1 Literature Review

In the literature review, an extensive search was conducted in conference proceedings, MSc and PhD theses, peer-reviewed journals, technical manuals and company brochures, looking for definitions of blast damage, its formation and measurement methods, both for over-break and the EDZ. A second part of the literature review focused on work on MWD technology, especially percussive drilling and its ability to characterise the rock mass (Van Eldert, 2018).

2.2 Field Studies

The sites selected for investigation were based on the type of data collected during the construction work and the willingness of the contractors and the client to share these data. The site requirements included geological knowledge determined in the site investigation, fracture mapping during the excavation, the possibility of conducting blast damage investigation measurements and, most importantly, the ability to collect MWD data from grout and blast hole drilling.

2.3 Data Collection

The geotechnical site investigation reports were reviewed and analysed. These included the initial Q-values and the prognosis of the rock condition of the test sites. The reports were supplied by Swedish Transport Administration (Trafikverket) (Arghe, 2013; Arghe, 2016) for the two ramp tunnels of the Stockholm bypass and by WSP (Karlsson, 2014) for the Veidekke access tunnel.

During the tunnel excavation, MWD data were collected at 2cm intervals from both the grout and blast holes. The grout hole data were used to determine the ground conditions ahead of the face and decide if additional grout holes were needed (Zetterlund et al., 2017). In addition, Cavity Monitoring System (CMS) scans were routinely performed by the contractors for excavation quality control. These scans provided accurate information on the volume of material extracted. Tunnel surface mapping data on the rock type, weathering, fracturing and the calculated Q-values or RMR values from both excavation sites were supplied by the geotechnical consultants (Karlsson, 2015; ÅF, 2016). These data were used to recommend a certain ground support design (Karlsson, 2015; ÅF, 2016). This data were reviewed by the author. Later, the surface mapping was used to differentiate between natural and blast induced fractures.

Malå GS Ground Penetrating Radar (GPR) was used to measure the tunnel walls in at the three sites. The system was equipped with a 1.6 GHz send-receiver antenna. The GPR measurements were taken every two centimetres based on the drawn-out distance of a wire. This corresponded with the MWD drill settings. A total of 34 GPR measurement lines were recorded in the tunnels and later processed with Malå GroundVision software.

Drill core (DC) extraction was performed with a Hilti DD200 diamond core drill, as seen in Figure 2.2. In these field data collections, a total of 49 drill cores were extracted using a 51mm inner diameter diamond drill. The locations for drill core extractions were selected by analysing the variations in the MWD Hardness and Fracture Indices (Veidekke Access tunnel, Tunnel 213 and 214) or were set in a regular grid (TAS04). The drill cores were logged according to rock type identification and RQD.

Figure 2.2 Collection of core samples with Hilti DD200 diamond core drill.

P-wave velocity measurements were taken diametrically, similar to procedures described by Eitzenberger (2012) at 2cm intervals along the drill cores; see Figure 2.3. The purpose was to obtain the threshold P-velocity of the in-situ rock mass. The threshold was defined by the distance from the tunnel wall where the P-wave velocity was constant.

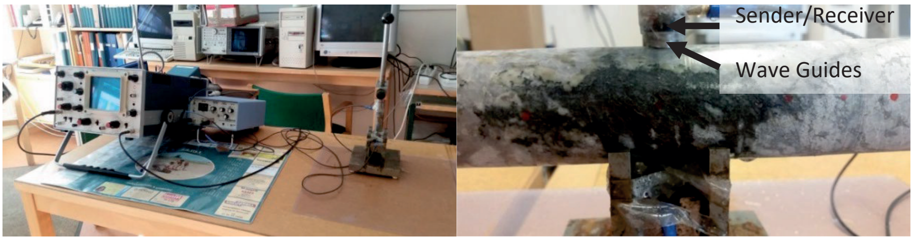


Figure 2.3 Setup for diametrical P-wave measurements on the collected drill cores.

2.4 Data Analysis

The analyses of the measurements on blast damage show the limits and benefits of the investigation methods presented above. Based on these analyses, the most suitable methods were selected and used in further investigation of the blast damage.

The MWD data were processed off-site using the software program of the suppliers of the drill rigs (Sandvik's iSure V7.0 and Atlas Copco's (now Epiroc) Underground Manager (UM) V1.6) and Matlab code. The UM software was used to normalize the MWD data and calculate the Fracture and Hardness Indices (Schunnesson, 1996; Schunnesson, 1998; Epiroc, 2018b). The MWD parameters were filtered based on the distribution of the collected data, whereby extreme values were removed. The purpose of the filtering was to remove unrealistic samples caused by data containing measurement errors or data heavily influenced by the drilling process, e.g. drill hole collaring and drill rod extensions. The MWD data filtering process removed the entire sample point ID when one parameter was outside the accepted interval. The Fracture and Hardness Indices for both the grout and blast holes were statistically scaled and compared. To evaluate the statistical reliability of the normalised Fracture and Hardness Indices, the interpolated Fracture Index was visually compared to the tunnel surface mapping. The software package was used to create a virtual tunnel contour with the MWD data along this contour by "folding out" the data; see Figure 2.4 and Figure 2.5. This presentation was similar to the presentation of fracture mapping data in a tunnel excavation by Karlsson (2015) and ÅF (2016). The folded-out contour was an interpolation of the MWD parameters at the tunnel contour and was compared with the mapped fractures.

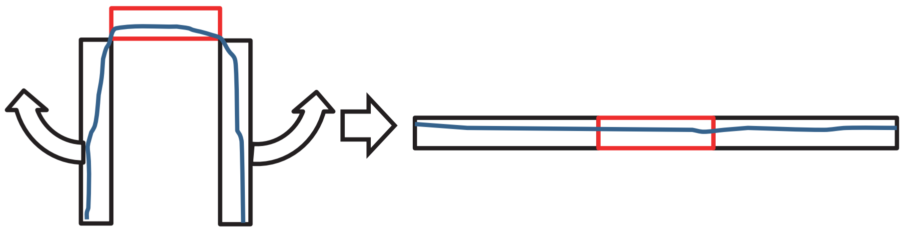
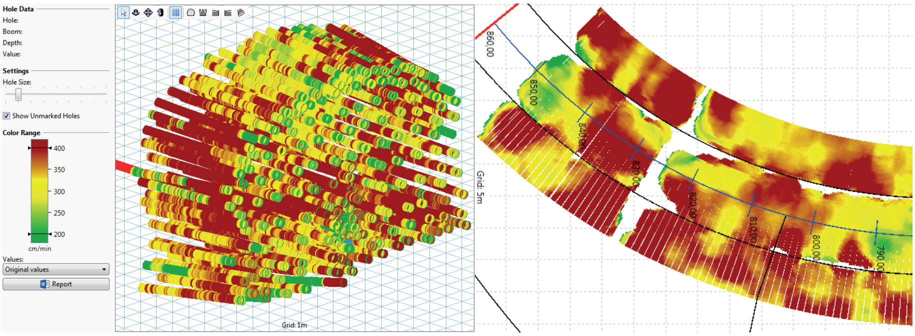


Figure 2.4 Folding out of tunnel contour for visualisation of tunnel mapping and 2D visualisation of tunnel walls and roof



A. B.

Figure 2.5 Penetration rate of one blast round in Underground Manager of section 796 in Tunnel 214 (A) and the interpolated penetration rate on the tunnel contour in the first 65 m of Tunnel 214 (B).

The data on rock mass quality and the design rock support from one case study were compared with the data collected from the site investigations done before the tunnel excavation. The initial Q-values from the site investigation were compared with the actual or mapped Q-value recorded during the tunnel excavation. The correlation between these data sets was later used to establish MWD's reliability as a predictor of rock mass characterisation.

Lastly, the correlation between the extent of the measured blast damage, the MWD data and operational parameters were investigated with Multiple Linear Regression. The studied explanatory variables were charge concentration, penetration rate, feed pressure, rotation speed, water flow rotation pressure, rock cover (tunnel depth), tunnel area and contour hole spacing. The selection of these MWD parameters was based on their inter-parameter correlations, as discussed by, e.g., Navarro et al. (2018a). In addition, the performances of Epiroc's Hardness and Fracture Indices were tested instead of the raw MWD values. The prediction models for the blast damage were then investigated to determine the most significant parameters. This was performed by the calculation of the *p-value*. The *p-value* for each term tests the null hypothesis; the parameters coefficient is equal to zero and therefore has no influence on the model. A low *p-value* (>5%) indicates the null hypothesis can be rejected, meaning the tested term is likely to be significant for the model.

3 LITERATURE REVIEW

This chapter discusses the relevant literature on tunnelling site investigations (Section 3.1) and Drill and Blast Technology (Section 3.2). It also summarises the history and current status of blast damage investigation (Section 3.3) and its measurement technologies (Section 3.4). Lastly, it addresses Measurement While Drilling technology (Section 3.5) and its current applications.

3.1 Site Investigation

A tunnelling project begins with a site investigation. The investigation determines the rock mass conditions to be expected and predicts their effect on the tunnel and its construction. The site investigation's importance is well-known (Wahlström, 1964; Hoek, 1982; U.S. National Committee on Tunneling Technology, 1984; Hoek and Palmieri, 1998; Nilsen and Ozdemir, 1999; Parker, 2004; Panthi and Nilsen, 2007; Lindfors et al., 2015). In most cases, the site investigation gathers information from existing sources (desktop study), along with data acquired by field mapping, core drilling, geophysical methods, exploratory audits, field tests and laboratory tests (Nilsen and Ozdemir, 1999; Lindfors et al., 2015). The desktop study consists of the collection of available background material, including topographical and geological maps, geological reports, aerial and satellite pictures etc. The gathered data may give further indication of zones of weakness, the degree of fracturing and jointing patterns and directions, soil thickness and degree of weathering. In addition, core drilling might be performed to verify the geological interpretation and obtain new information on the rock type boundaries and degree of weathering. Additional information about the orientation and characteristics of the weakness zones, samples for laboratory analysis, and hydrogeological and geophysical information are often gathered during core drilling (Nilsen and Ozdemir, 1999; Lindfors et al., 2015). Geophysical methods, including seismic refraction, seismic reflection and Ground Penetrating Radar, might be used to determine, e.g., the thickness of the soil or the degree of weathering (Nilsen and Ozdemir, 1999; Lindfors et al., 2015).

Field tests are mostly employed to measure of in-situ rock conditions and stresses, as well as groundwater conditions. In the laboratory tests, the intact rock properties are investigated, including uniaxial and tensile strength, brittleness-value, surface hardness and abrasiveness. These measurements are taken depending on the rock mass conditions. After a thorough analysis of these data, the excavation method is selected (Nilsen and Ozdemir, 1999). The extent of the site investigation depends on the rock mass conditions and the location of the tunnel construction. In general, the U.S. National Committee on Tunneling Technology (1984) recommends a site investigation budget of 3% of the total estimated project costs. However, in hard rock projects, the site investigation costs may be 0.5-1% (Nilsen and Ozdemir, 1999). The degree of detail of site investigation is decided by the project owner depending on potential problems and the degree of expected difficulties (U.S. National Committee on Tunneling Technology, 1984; Nilsen and Ozdemir, 1999; Parker, 2004). The site investigation is required for the tendering process in a tunnelling project (Lindfors et al., 2015). The findings of the investigation are then applied to determine excavation parameters and rock support requirements.

The lack of geological data at the planning stage makes an accurate and reliable rock mass quality assessment difficult. Discrepancies were found in the Citybanan project in Stockholm (Kjellström, 2015) and the Harold D. Roberts tunnel in Colorado, USA (Wahlström, 1964) and were noted in a report by the U.S. National Committee on Tunneling Technology (1984). This is not a new phenomenon: it was noted during the construction of the Simplon tunnel in 1853 (Kovári and Fechtig, 2000). The lack of a reliable assessment may cause conflicts between the client (owner) and contractor.

3.2 Drill and Blast Excavation

The drill and blast excavation consists mainly of the following cycle (also demonstrated in Figure 3.1):

1. Face scaling to prevent rock fall at the face and problems during drilling;
2. Blast hole drilling with fully mechanized drill rigs;
3. Charging of blast holes, commonly with bulk emulsions, where the charge concentration is reduced in helper and perimeter holes;
4. Blasting and ventilation, with pyrotechnical and/or electronic detonators;
5. Mucking and cleaning with large front-end loaders in combination with dumpers or trucks;
6. Scaling and rock support with fully mechanised equipment for scaling, shotcrete spraying, and bolting, often in tunnelling with a face drill rig.

In addition, pre-grouting may be performed every third or fourth excavation cycle.

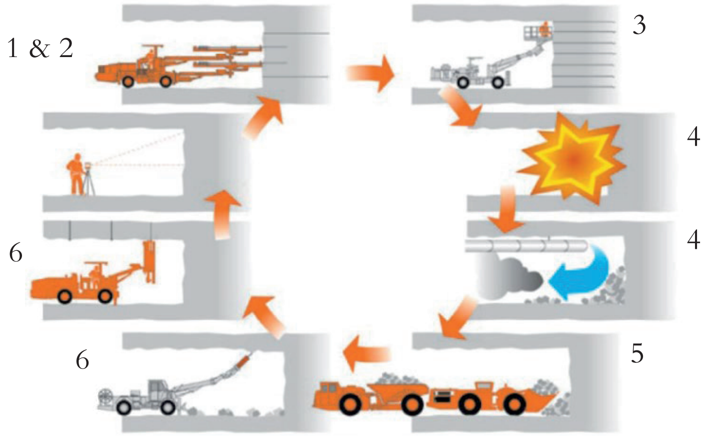


Figure 3.1 Excavation cycle in tunnel excavation (Modified after Tamrock, 1999).

State-of-the-art excavations are now performed with smooth wall blasting techniques (Langefors and Kihlström, 1978; Holmberg and Persson, 1979; Holmberg and Hustrulid, 1981; Olsson and Ouchterlony, 2003) to minimise unwanted damage to the remaining rock mass. This is often done by placing decoupled charges in the contour and helper holes. In smooth wall blasting, the contour holes are initiated simultaneously (electronic detonators) at the end of the blast round. As a result, the remaining rock mass sustains less damage. The most commonly used explosive in Scandinavia is bulk emulsion; it allows varying charge concentrations depending on the excavation requirements.

Today's tunnelling machines are computerized and have the ability to drill (semi-) automated (Epiroc, 2018a; Sandvik, 2018). This optimises excavations and offers an opportunity to acquire excavation data, e.g. activity duration, drilling, charging and mucking logs (Humstad et al., 2012). The drilling performance is highly influenced by the rock mass properties, e.g. compressive rock strength, rock texture, rock mass structure, mineral composition, cavities, weathering, porosity and permeability (Howarth and Rowlands, 1987; Thuro, 1997). Operator skill, rig, hammer and drill bit type also influence the drilling performance (Thuro, 1997).

3.3 Excavation Damage

Extensive efforts to reduce blast damage were initiated in the 1950s (Langefors and Kihlström, 1978). Investigations to quantify the extent of blast damage started in the 1970s with the PPV-approach (Holmberg and Persson, 1979), although the main purpose of this work was to reduce the over-break. Blast damage and its quantification are still of interest today (Fjellborg and Olsson, 1996; Nyberg et al., 2000; Olsson and Ouchterlony, 2003; Ouchterlony et al., 2009; Ericsson et al., 2015; Ittner et al., 2018).

The Excavation Damage Zone (EDZ) is a result of an excavation in a rock masses. It is characterised by irreversible changes in rock mass properties (Martino and Chandler, 2004; Christiansson et al., 2005). The excavation method, design parameters, rock mass properties and in-situ stresses influence the characteristics of the EDZ (Olsson and Ouchterlony, 2003; Christiansson et al., 2005; Ouchterlony et al., 2009). In principle, the EDZ can be divided into subzones (Saiang, 2008; Siren et al., 2015). These are discussed below and displayed in Figure 3.2.

1. Failure Zone or over-break consists of connected fracture networks, causing rock fall-outs beyond the planned tunnel profile.
2. Damage Zone is split into three parts, as shown in Figure 3.3:
    a. Inner Damage Zone (Crush Zone) is located directly around the blast hole and is caused by the shock-wave energy of the detonation.
    b. Transition Zone consists of microfractures connecting and forming macro fractures, both radially and parallel to the tunnel wall.
    c. Progressing Zone extends the existing radial fractures.
3. Stress Damage Zone consists of rock damage caused by the redistribution of stresses.

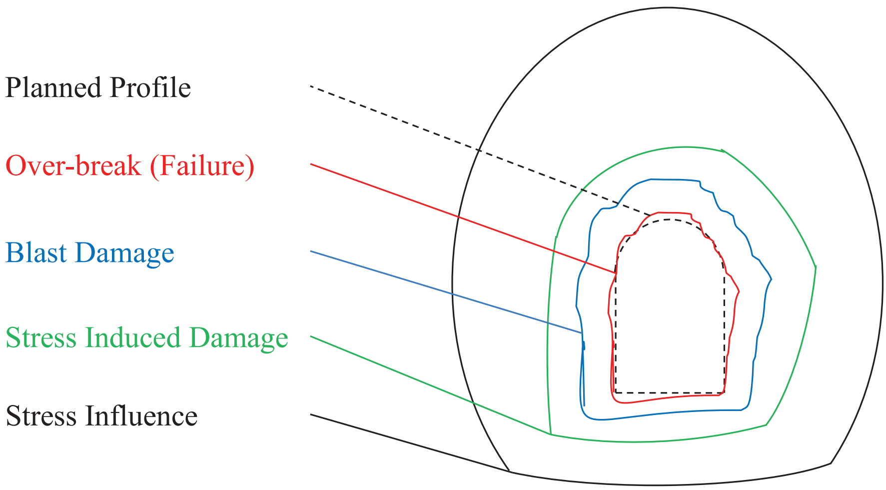


Figure 3.2 Excavation damage divided into subzones: over-break, blast damage, stress induced damage and stress influenced areas.

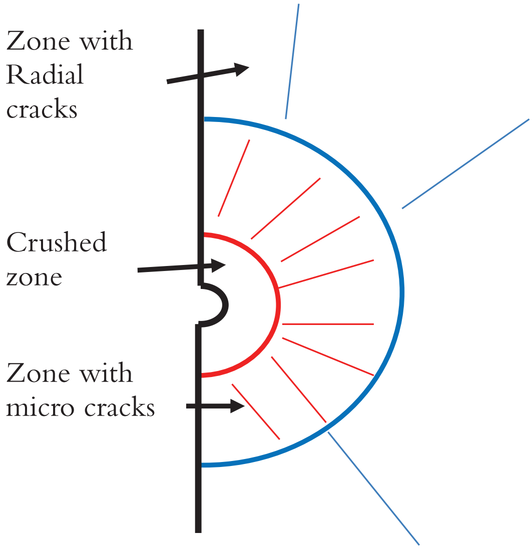


Figure 3.3 Development of Excavation Damage around the blast hole and characteristic zones.

3.4 Blast Damage Measurements

Blast damage can be investigated using a number of methods. The majority have been in use for several decades, but advanced technology has led to new methods. The most common ones are discussed in the sections below.

Core drilling and *rock slicing* are techniques to gain samples for visual inspection of fractures. For this purpose, drill cores (DCs) are extracted perpendicular to the tunnel surfaces. The DC reveals information about the lithology and the condition of the rock mass which has been traditionally measured in terms of the Rock Quality Designation (RQD) parameter (Deere, 1964). Increased fracturing close to the wall is an indication of blast damage. In addition, the physical characteristics of the fractures are used to differentiate between blast fractures and natural fractures. "Fresh" fractures (without weathering, erosion or filling material) are most likely caused by the excavation, e.g. blasting. With rock slicing, slabs of rock are cut out from the excavation walls and floor and visually inspected (Fjellborg and Olsson, 1996; Nyberg et al., 2000). Blast induced damage is distinguished from natural and stress induced fractures by visual interpreting the fractures' location, direction and appearance (Olsson and Ouchterlony., 2003; Ouchterlony et al., 2009; Ericsson et al., 2015; Ittner et al., 2018). This method can be applied to obtain a 3D image of the developed fracture network.

Borehole camera scanning is applied in a similar fashion as core drilling and logging. In this case, the borehole is filmed, and fractures are examined based on the acquired images (Ghosh, 2017; Navarro et al., 2018b). *Scratcher logs* (mechanical tracing of the drill hole wall) and *Pader logs* (imprint of the drill hole wall) are applied in a similar fashion.

Rock surface mapping or *fracture mapping* is generally carried out to estimate the rock mass quality in sections along the tunnel (Edelbro, 2004). The most common are the Q-system (Barton et al., 1974), Rock Mass Rating (RMR) (Bieniawski, 1973) and Geological Strength Index (GSI) (Hoek and Brown, 1997). The latter system includes a rock mass damage factor (Hoek et al., 2002). These classification systems can be applied to investigate the fracture density (number of fractures per given length) as this might indicate the extent of the blast damage.

Half Cast Factor (HCF) is the ratio of half cast or half barrels visible after blasting to the number of contour holes drilled (Lizotte et al., 1996). The HCF is applicable to hard or competent rock masses. A high HCF indicates a stable, competent rock mass with limited blast induced damage and low frequency of natural fractures (Lizotte et al., 1996; Fjellborg and Olsson, 1996; Singh and Narendrula, 2007).

Cavity Monitoring Scanning (CMS) (Mohammadi et al., 2017; Navarro et al., 2018c) or *Profile Scanning* (Van Eldert, 2014) is used to measure the excavated volume. This is done using a point cloud or tunnel profile line. The value gives an indication of over-break compared to the expected volume. *3D-photogrammetry* is used in a similar fashion (Ericsson et al., 2015).

Ground Penetrating Radar (GPR) sends high-frequency waves ranging from hundreds of MHz to several GHz into the rock mass. The wave energy is reflected by micro and macro fractures (MALÅ Geoscience, 2016). The micro fractures create a large number of small reflections, causing a large band of energy loss called dispersion (Silvast and Wiljanen, 2008). Macro fractures reflect the waves, and this reflection can be observed in the GPR results (Silvast and Wiljanen, 2008). The zone of dispersion is seen as the direct extent of the EDZ. Macro fractures might exist prior to the excavation or be caused by blasting. Surface fracture mapping should be used in conjuction with GPR to determine the different fracture types and establish the exact number of blast fracures. The extent of the blast induced fractures determines the depth of the EDZ (Silvast and Wiljanen, 2008; Ericsson et al., 2015).

Hydraulic tests are conducted to measure the flow of fluids in the rock mass. One of these methods consists of injecting water into the rock mass and recording the pressure and flow parameters in adjacent drill holes (Ericsson et al., 2015). The hydraulic transmisivity in the rock mass is calculated from the response time interval. Increased transmisivity corresponds to an increased number of fractures and, thus, blast damage.

P-wave velocity can be measured along the core samples, between drill holes or at the (tunnel) surface. The rock mass texture and mineralogy affect the P-wave velocity (Jern, 2001; Saiang, 2008; Eitzenberger, 2012). Voids and other inclusions in the material reduce the P-wave velocity and wave amplitude (Jern, 2001; Saiang, 2008; Eitzenberger, 2012). These voids can be caused by the blasting microfractures (Jern, 2001). The degree of the reduction of P-wave velocity and thus the number of micro fractures indicate the severity of the blast damage. The extent of the EDZ is determined by the P-wave velocity transition point from damaged rock mass to in-situ rock mass (threshold). At this transition point, the P-wave velocity levels; no changes in the velocity occur at further depth (Jern, 2001; Saiang, 2008; Eitzenberger, 2012).

Scaling time is the duration of the scaling activity during the excavation cycle; in scaling, the loose rocks are broken away by either hand-held bars or a mechanical hammer. Scaling time is an indication of blast damage, but the actual extent of blast damage is difficult to quantify, since the operator and the rock mass conditions have a major influence on the duration of this activity (Lizotte et al., 1996).

Loading tonnage and *loading time* are based on the total rock mass amount that has been excavated (Lizotte et al., 1996). This method can be used to quantify over-break and to indicate the EDZ based on the expected loading before and actual loading after blasting.

Peak Particle Velocity (PPV) is a method measuring the wave amplitude of a pressure wave after blasting (Holmberg and Persson, 1979). The PPV is back-calculated from the measurement point to the detonation point. In the 1970s, the PPV was correlated to the fracture growth after blasting with a certain type and amount of explosives (Holmberg, 1978; Holmberg and Hustrulid, 1981).

3.5 Measurement While Drilling Technology

Measurement While Drilling (MWD) technology monitors and records drilling parameters. A significant amount of research on drill parameter logging in tunnelling and mining was done in the 1960s and 1970s in the United Kingdom (Schunnesson, 1987), in the 1970s and 1980s in the United States of America (Schunnesson, 1987) and since the middle of the 1980s in Sweden (Schunnesson, 1987). The findings of these studies are discussed below.

MWD Parameters

MWD data are a record of the drilling operation. The data contain basic drilling information, e.g. drill hole ID, hole type, navigated drill rig location, hole collar location, hole depth, time-stamp, as well as the drilling and recording settings. The data file also includes the actual drilling data, recorded at a set sample distance. A sample of Epiroc MWD data is shown in Figure 3.4. The sample resolution ranges from 2cm to 20cm (Atlas Copco, 2009).

```
[MWD 2.7]
Hole number
3
Hole type
4
Date and time at rockcontact
2015/05/16 15:14:56
Boom
1
Section number * 1000
142638
x       y       z       mm
-3737   1137    75
Lookout Lookoutdirection(Degrees*10)    sample interval(cm)
46      -1764   2
Rig serial number
8991487100
RCS 3.7 rev. 8
Tunnel
IT
[MWD DATA]
HD mm   PRdm/minHP bar  FP bar  DP bar  RSr/min RP bar  WFl/min WP bar  Time
0       0.00    3.42    20.92   37.89   319.89  61.91   69.09   10.67   15:14:56
22      0.00    76.01   14.94   38.73   314.78  65.76   92.61   17.08   15:14:59
47      19.90   115.72  15.80   39.57   314.78  66.18   103.49  16.65   15:15:00
73      19.92   118.71  14.09   39.15   317.84  67.04   103.78  16.65   15:15:01
97      18.84   122.55  16.65   38.73   319.89  67.04   109.07  15.80   15:15:02
121     18.74   125.54  14.94   38.73   312.73  63.62   113.48  15.80   15:15:02
137     17.15   127.67  14.94   39.57   318.86  62.77   112.01  15.80   15:15:03
[EXTRA INFO]
Mine=
Front=
TypeOfNavigation=12
Transform=[  0.52083594    0.85365679      0.00000000
        -0.02966000     0.01809626      0.99939622
        -0.85314137     0.52052147     -0.03474464
        6582551.43      -155452.14          4.51 ]
Lookout Lookoutdirection(Degrees*10)    sample interval(cm)
31      1398    2
```

Figure 3.4 Sample of MWD data from the Atlas Copco (now Epiroc) drill rig, including hole type, location, drilling direction and drilling parameters recordings every 2cm.

The drilling parameters can be divided into independent and dependent parameters (Brown and Barr, 1978). The independent parameters are not influenced by the rock mass but solely by the rig capacity, the drilling settings, the operator and the control system. These parameters are bit thrust or percussive pressure, feed pressure and rotation speed; see Figure 3.5. The dependent parameters are those influenced by the drill system's response to varying rock conditions. These typically are penetration rate, torque or rotation pressure, damper or stabilization pressure, as well as flushing flow and pressure; see Figure 3.5. Additional dependent parameters, e.g. vibration and machine temperature, might be recorded, depending on drill rig type (Van Eldert, 2018).

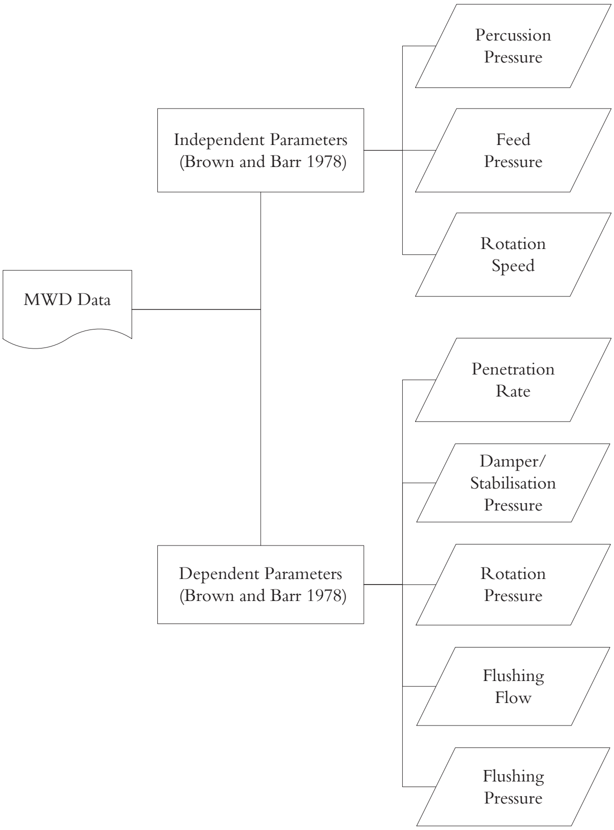


Figure 3.5 Independent and dependent MWD parameters available in Atlas Copco (now Epiroc) MWD data after Brown and Barr (1978).

Field and production data often contain faulty or unrealistic data samples. This is normally the case for MWD data, e.g. negative, very low or high values for operational pressures and penetration rate (Ghosh, 2017). Filtering the MWD data may be complicated and tedious, but must be done before analysis can be performed to distinguish rock mass conditions. However, a conservative filtering approach might be applied without losing the general pattern of the large data set. For the drilling data to analysed, they must be normalised. This normalisation process uses the regression lines according to the hole depth and the drill parameter interaction (Schunnesson, 1990). Normalisation removes the influence of the rig control system and operator (Schunnesson, 1998). As a result, the filtered and normalised data only portray features of the rock mass.

Un-calibrated MWD data can give relative rock mass properties within one tunnel excavation (Atlas Copco, 2009), if the settings are similar (rig, hole diameter, hole depth etc.). Drill bit type and hole diameter influence the MWD data significantly (Brow and Barr, 1978; Schunnesson, 1997; Thuro, 1997). For example, small diameter drill bits give higher penetration rates than large diameter drill bits for the same feed pressure and rotation pressure. Therefore, MWD data from different sources, e.g. grout holes and blast holes, must be compared with great care. The drilling data are analysed and often calibrated against measured rock mass properties (Bever Control, 2015; Rockma, 2018; Schunnesson et al., 2012). For this calibration to be accurate, extensive measurement and testing campaigns are necessary.

Rock Mass Characterisation using MWD Data

The main application of MWD is to find anomalies or zones of weakness within the rock mass and use this information to optimise the excavation. Several indices determined from the MWD data are used in this process. The most common ones are discussed below.

The ***"hardness"*** parameter or Hardness Index portrays the drillability of the rock mass according to the filtered and normalised penetration rate (Bever Control, 2015). In the case of UM, this can be found in the computer code. Its Hardness Index is calculated based on the hole depth, normalised penetration rate and normalised percussive pressure. The slopes of the regression lines are pre-set within the software package. A higher Hardness Index value normally indicates soft or fractured rock masses (higher drillability), and a lower Hardness Index value normally indicates solid competent rock masses (lower drillability) (Schunnesson, 1998).

The ***"fracturing"*** parameter or Fracture Index is based on variation of the normalised MWD data. Schunnesson (1990) and Ghosh (2017) used normalised penetration rate and normalised rotation pressure with their residuals to calculate the Fracture Index. Navarro et al. (2018b) used normalised percussive pressure, normalised feed pressure and normalised rotation pressure to calculate the Fracture Index. In the case of UM, the Fracture Index calculation is based on the deviation of the pre-set regression line of the normalised penetration rate and rotation pressure. This parameter reflects the heterogeneity of the rock mass, where open and clean fractures result in an increased penetration rate, rotation speed and reduced torque, thrust and water pressure (Schunnesson, 1996; Schunnesson, 1998). In weak and highly fractured rock masses, the drill holes may cave. This results in increased rotary friction and increased torque and could cause jamming of the drilling rod (Schunnesson, 1998). Therefore, the result could be reduced penetration rate. In addition, when jamming occurs the anti-jamming mechanism intervenes.

The ***water*** parameter or Water Index displays the normalized water flow. The changes in the water pressure during drilling are measured to give an indication of both water-bearing structures and dry fractures (Schunnesson et al., 2011).

Development and Applications of MWD Data

The development of MWD data started with a series of laboratory experiments have investigated the correlation between MWD parameters and concrete or rock blocks. The known hardness and voids of the casted concrete blocks were correlated to the MWD data (Andersson et al., 1991; Frizzell et al., 1992) Later the data correlation was tested on rock blocks; in this case the data was verified with diamond core data or borehole camera (Andersson et al., 1991; Frizzell et al., 1992; Finfinger et al., 2000; Mirabile et al., 2004).

Andersson et al. (1991) discussed the use of drilling parameter logging for rock mass characterisation in Zinkgruvan and Kirunavaara Mine. The focus in the rock mass characterisation was on fracture indications. Andersson et al. (1991) also discussed methods for processing MWD data and validating them using Ground Penetrating Radar and geological mapping.

Schunnesson (1996) employed MWD data logging in the Glödberget tunnel to assess the rock mass quality. In general, the results showed a good correlation between the RQD and the penetration rate and torque pressure. His findings indicated that an increased RQD leads to a decreased penetration rate and decreased torque pressure.

Schunnesson (1997), Schunnesson and Sturk (1997) and Lindén (2005) studied the use of MWD during the construction of the Hållandsås tunnel in Sweden. Their study demonstrated both the practical benefits and the challenges of MWD data recording and predicting the rock mass conditions ahead of the face. Lindén (2005) investigated the MWD data from the grout holes during the TBM excavation of the Hållandsås tunnel. The study found that the MWD of the grout holes was well correlated with the rock conditions ahead of the cutter head.

Finfinger et al. (2000), Peng et al. (2003), Tang et al. (2004), Mirabile et al. (2004), Sasoka et al. (2006) and Kahraman et al. (2015) described the development of a Mine Roof Geological Information system (MRGIS), where drilling parameters were linked to the drillability (strata hardness), fractures and voids. The system was trained on the laboratory data and later validated in field tests in coal mines (Peng et al., 2003; Tang et al., 2004; Mirabile et al., 2004) with diamond coring and drill hole filming. The MRGIS was able to identify single fractures, fractured areas, different rock types and UCS and had the ability to produce a 3D image of the mine roof.

Apelqvist and Wengelin (2008) studied MWD data from grout holes during the excavation of the North Link tunnel in Stockholm. The calculated Fracture Index (Schunnesson, 1996) and drill water flow during the drilling were compared with the mapped fracture frequency after blasting. Apelqvist and Wengelin recommended a calibration for each drill rig, boom and construction site based on the calculated Fracture Index. The calibrated rigs were used to identify the grout class of the excavation. Carlsvärd and Ekstam-Wallgren (2009) and Martinsson and Bengtsson (2010) continued the study of the MWD data from the North Link tunnels. These data were used to optimise the grouting during the construction of the tunnel. Martinsson and Bengtsson (2010) also discussed the limitations of this method, including of the time required for the rig calibration (up to several days), inaccuracies due to intra- and extrapolation of the drilling data, data imprecision due to the measurement of indirect values, i.e. oil pressures, and influence of the operator on the drilling performance.

Kim et al. (2008) investigated MWD technology in sedimentary rock masses in the Soran tunnel, South Korea. The MWD data for probing holes showed inconclusive results. However, sharp changes of feed pressure were observed in fractured zones.

Hjelme (2010) investigated the rock mass quality with MWD data from probing holes in the Løren tunnel, Norway. The study showed a relatively good correlation between the penetration rate and the geotechnical mapping of the tunnel; e.g. weaker rock mass areas had a higher penetration rate.

Valli et al. (2010) calibrated the recorded penetration rate with the hardness of crystalline rock masses in Olkiluoto, Finland. The majority of the investigated rock types were within a similar strength (UCS) range. Interestingly, these rock types could be separated based on the drilling performance (penetration rate), due to the difference in mechanical properties (drillability). Furthermore, the variations in the MWD parameters could determine the degree of fracturing of the rock mass.

Fjæran (2012) investigated the correlation between the rock mass quality and MWD data from probe holes for the Vågsbyggsporten in Norway. The correlation between the Fracture Index, calculated in Rockma's GPM+ software, and observed fracture frequency in the tunnel was good to very good for 72% of the investigated tunnel sections.

Schunnesson et al. (2012) employed MWD technology in the Chenano-Nashri tunnel in India. The rock mass consisted of sedimentary rock types. The MWD Hardness Index was calibrated using Schmidt Hammer measurements. It was able to portray the sedimentary strata of the rock mass.

Rødseth (2013) correlated the MWD data to hardness, jointing and water inflow in the Løren, Oppdølstranda and Eikrem tunnels in Norway. The study showed a good to moderate correlation between the MWD data and the RQD, but a low to moderate correlation between the MWD and jointing.

Høien and Nilsen (2014) studied the quality of grouting in the Løren tunnel, Norway. MWD indices, such as hardness, fracturing, and water flow, were calibrated with field data (point load tests and fracture mapping). The study made a statistical comparison between grout consumption, the degree of fracturing, water leakage and MWD Hardness, Fracture and Water Indices. The studied showed a correlation between the MWD Fracture and Water Indices and the grout consumption.

Navarro et al. (2018c) applied MWD to predict over-break at Bekkelaget in Oslo. The correlation between the gathered CMS data and processed MWD parameters (normalisation and data variation) showed a good correlation for the over-break (R^2: 0.74).

4 SITE DESCRIPTIONS

Three excavation sites were investigated for this study. Two are located in the Stockholm area, and the third is in the south of Sweden in the Oskarshamn area. These sites are described below.

4.1 Ramp Tunnels 213 and 214 of Stockholm Bypass

Ramp tunnels 213 and 214 are part of the Stockholm bypass. The Stockholm bypass consists of 21km of new roads, of which 18km will be located underground (Trafikverket, 2018). The construction of the first access and ramp tunnels started in 2015 in Skärholmen in Stockholm; see Figure 4.1 and Figure 4.2.

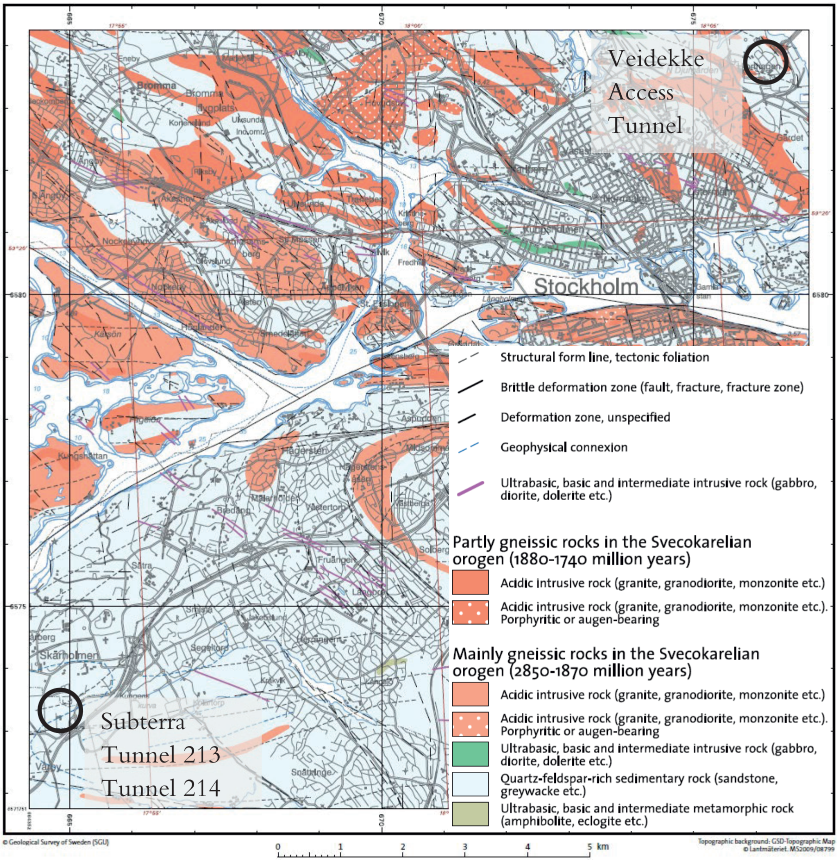


Figure 4.1 Stockholm's geological map of the Stockholm area with the two Stockholm investigation sites in this study (after SGU, 2017).

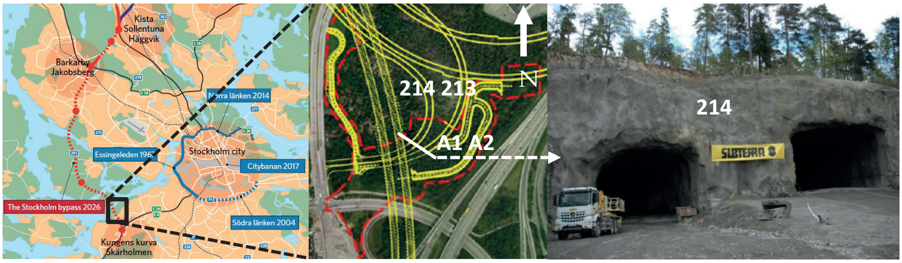


Figure 4.2 Stockholm bypass and Tunnels 213 and 214 location, layout and tunnel entrances (Illustrations courtesy of Trafikverket).

The rock types in the excavation area are mainly gray, medium to large grained gneiss (Arghe, 2013; Arghe, 2016). Lightly foliated granite, pegmatite and greenstone veins are also observed in the rock mass. The surface outcrops indicated severely weathered and oxidized rock masses, expected to extend into the tunnel (Arghe, 2016). The description of the rock classes and their measured parameters can be found in Table 4.1. The rock class and initial rock support prognosis along the two ramp tunnels are listed in Table 4.2. The rock conditions were expected to be generally favourable in both tunnels.

Table 4.1 Rock classes and Q-value applied for the rock mass classification at the Stockholm bypass (after Arghe, 2016).

Rock Class	Q-value	Rock Quality	Description of rock mass
I	$Q > 10$	Very good	Sparsely fractured or large blocky granite, gneiss-granite, pegmatite or rarely slaty gneiss. Mainly rough fracture surfaces with no or little fracture filling. Average edge length >2m. Three or fewer fracture sets.
II	$4 < Q \leq 10$	Good	Large or medium blocky granite, gneiss-granite, pegmatite or moderate slaty gneiss. Mainly rough fracture surface with little fracture filling. Average edge length 0.6-2m. Three or more fracture sets.
III	$1 < Q \leq 4$	Fair	Medium to small blocky granite, gneiss-granite pegmatite or slaty gneiss. Fracture surfaces are rough to smooth, with moderate fracture filling. Average edge length 0.2-0.6m
IV	$0.1 < Q \leq 1$	Poor	Small blocky to crushed, metamorphic granitic rock mass or heavenly slated gneiss with mineral-filled fractures. Average edge length <0.2m.
V	$Q \leq 0.1$	Very poor	Tectonically heavily affected, disjointed rock mass, fracture and crush zones. Mainly smooth, polished fracture surfaces filled with large amounts of soft minerals.

Table 4.2 Expected rock condition from the site investigation in tunnels 213 and 214 in the Stockholm bypass (Arghe, 2013; Arghe 2016).

Tun.	Sect.	Rock Class	Q-value	Rock Cover	Remarks	Bolt		Shotcrete Thickness	
						Spac.	Length	Wall	Roof
213	200 to 210	III	1.5	3.5m	SRF=5, Jn=6x2	1.7m	3m	50mm	75mm
213	210 to 215	II	6	5 – 10m		S	3m	0mm	50mm
213	215 to 245	III	3	5 – 10m	Weak zone #189 at section 245 to 250	1.7m	3m	50mm	75mm
213	245 to 270	II	6	5 – 10m		S	3m	0mm	50mm
213	270 to 366	II	4.2	14 - 34m		S	3m	0mm	50mm
214	848 to 836	IV	1	10 - 13m	SRF =2.5, corrected Q-value (Jn x2)	1.5m	3m	50mm	75mm
214	836 to 825	II	5	>10m	SRF =1	S	3m	0mm	50mm
214	825 to 810	IV	0.7	17-22m	weak zone #189 at section 820, corrected Jn, Jw=0.66, SRF=5	1.5m	3m	50mm	75mm
214	810 to 792	II	6.1	>20m	Only gneiss, correct for Jn	S	3m	0mm	50mm
214	792 to 615	I	12.2	>20m	corrected Q, after excavation	S	3m	0mm	50mm
Note: Tun.= Tunnel, Sect.=Section, Spac.= Spacing, SRF = stress reduction factor, Jn = joint set numb., Jw = joint water param., S = Selective bolting									

The excavation of the 97-119m² tunnels was conducted with an *Atlas Copco WE3* drilling rig for ø48mm drill holes with a specific drilling of 1.44m/m³. The contour holes were spaced 50-90cm apart along the tunnel perimeter and charged with 0.350kg/m string emulsion and 0.4kg bottom charge (*Forcit Kemiiti 810*). Pyrotechnical detonators (*Austin Powder*) were used at this excavation site.

4.2 Veidekke Access Tunnel in Norra Djurgården, Stockholm

The Veidekke access tunnel is a 50m long tunnel connected to an underground collection depot for household waste in Norra Djurgården, Stockholm (Figure 4.1). Figure 4.3 shows the layout of the construction of the 60-76m² tunnel (8m x 6.5m) and the cavern (50m x 20m x 12.5m) (Karlsson, 2014) The rock mass consists mainly of fine-grained granite and gneiss. The Rock Mass Rating (RMR) was estimated to be between 60 and 80 in the site investigation (Karlsson, 2014). During the excavation in 2015, an *Atlas Copco XE3* drill rig drilled ø48mm drill holes at an average specific drilling of 1.60m/m³. The contour holes were spaced 45-50cm apart along the tunnel perimeter. These were charged with emulsion 0.350kg/m string charge with 0.4kg bottom charge (*Orica Civec*) to reduce the blast damage. The blasting rounds were initiated with an electronic blasting system (*Orica eDev2*).

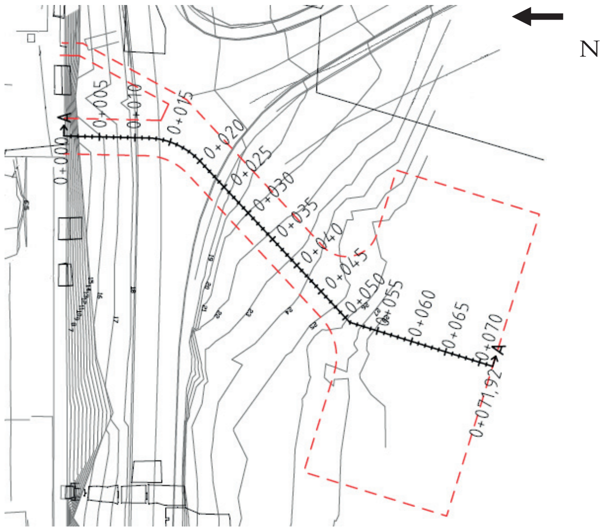


Figure 4.3 Layout of Veidekke access tunnel and gallery excavated at Norra Djurgården, Stockholm (Karlsson, 2014).

4.3 SKB TAS04 Tunnel at Äspö HRL, Oskarshamn

The test site was located at Äspö Hard Rock Laboratory (HRL), an underground research facility of the Swedish Nuclear Fuel and Waste Management Co. (SKB) close to Oskarshamn, Sweden. During 2012, several new tunnels were excavated at the 410m level (see Figure 4.4). The geology of this particular 36m long and 19.7m² tunnel consists mainly of fine-grained granite, diorite, granodiorite and pegmatite (Ericsson et al., 2015). The excavation was performed as a show case for best practices in Drill and Blast tunnelling. Therefore, it was excavated with great care, quality assurance and quality control (Ericsson et al., 2015). A brand new *Sandvik DT920i* drilled ø48mm drill holes with an average specific drilling of 4.04m/m³ in eight rounds. The contour holes were spaced 40-50cm apart along the tunnel perimeter. They were charged with a 0.350kg/m string emulsion and 0.5kg bottom charge (*Forcit Kemiiti 810*). Blasting was initiated with an electronic blasting system (*Orica i-kon VS*).

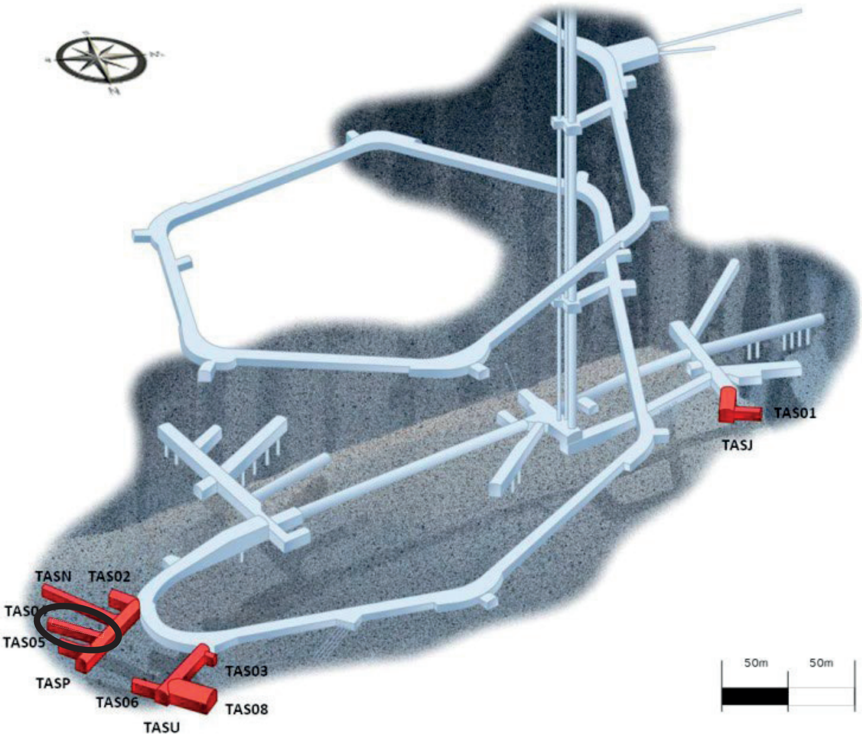


Figure 4.4 Äspö Hard Rock Laboratory layout. The data collected in this study were from the TAS04 tunnel, denoted by the black circle (modified after Johansson et al., 2015).

5 RESULTS AND DISCUSSION

This chapter presents the findings from the appended papers with a focus on the research questions. It begins with an evaluation of potential blast damage measurement methods to estimate the extent of the blast damage, as described in the literature review (Section 5.1). This is followed by applications of six methods in Section 5.2. Section 5.3 describes the application of Q-values at the ramp tunnels and their related rock support designs. This is followed by a correlation analysis of MWD data with rock mass characterisation (Section 5.4), rock support (Section 5.5) and blast damage (Section 5.6).

5.1 Comparison of Methods for Blast Damage Investigation

The literature review in Chapter 3 gives an extensive overview of the most common methods to investigate blasting damage. Table 5.1 compares these methods' benefits and limitations.

Methods such as Peak Particle Velocity, Standardised Blasting Tables, operational times and tonnages hardly interrupt the tunnel excavation. These methods are relative low cost but give only an indicative value of the blast damage because of their nature. More specifically, these methods only collect indirect parameters of the operation and discard effects of the geology and other operational parameters, e.g. simultaneous initiation and drill hole deviations. More advanced methods (e.g. Half Cast Factor (HCF), Cavity Monitoring System (CMS), Ground Penetrating Radar (GPR), tunnel mapping and photogrammetry) need access to the excavation face or walls. This access often results in minor production interruptions in the range of one hour (ÅF, 2016). In addition to this access, CMS and GPR need specialised equipment and direct contact with the rock mass. There cannot be any shotcrete, as it introduces a measurement error in the tunnel volume and needs to be corrected (Navarro et al., 2018c). In addition, the metal fibre in the shotcrete is impermeable to the GPR as it reflects the radar waves. The most direct methods, e.g. core hole drilling, rock slicing and P-wave velocity, measure the rock mass properties directly. These methods are time consuming and costly. They need physical sampling of the rock mass in the form of drill cores or rock slices. The physical extraction of these samples may cause excavation delays and requires special equipment. All these methods give reliable data in competent rock masses. In poor rock mass conditions, the methods may not be able to extract usable data on the excavation damage, e.g. the HCF, rock slicing, scaling time etc.

The most common methods are compared in Table 5.1. Based on the methods' limitations described above and this comparison, the most suitable investigation method for blast damage can be selected for each occasion. The selection should be based on I) the aim (an indication or specific information on a single fracture), II) the allowed production interruption and III) the available funds.

Table 5.1 Comparison of methods for over-break and Excavation Damage Zone investigation, based on the results of this study.

Method	Benefits	Limitations
Peak Particle Velocity (PPV)	No production interruption	Ignores many parameters, site-specific
Standardized Blast Tables	No production interruption, based on charge	Ignores many parameters, site-specific
Half Cast Factor (HCF)	Limited interruption, simple	Only surface data, minimal depth
Scaling Time	No production interruption	Indication only, depending on operator
Cavity Monitoring System (CMS) (Scanning)	Accurate, objective	Needs contour & hole, time-consuming
3D Photogrammetry	Limited interruption, good indication	Shadowing needs contour & hole
Loading Tonnage	Tonnage, production data	Needs contour & hole, needs rock density & swell factor
Loading Time	Indication of amount of rock, production data	Indication needs “loading tonnage”, influenced by fragmentation & loading
Tunnel Mapping	Clear picture, fracture orientation etc.	Only surface data, interruption of production
Ground Penetrating Radar (GPR)	Detects microfractures, limited interruption, “3D”, penetrates rock mass	No shotcrete, metal objects interfere, calibration needed
(Diamond) Core Drilling (DC)	Fracture type & filling penetrates	Time-consuming (interruption), sparse data collection, expensive
Rock Slicing	Fracture type & filling penetrates, 3D	Very expensive, time-consuming
P-wave Velocity	Detects microfractures, standardized method	Need drill cores

5.2 Application in Blast Damage Investigation

Half Cast Factor

In the tunnel excavations examined, the Half Cast Factor (HCF) was not continuously determined. The HCF was calculated for one section in Tunnel 213 based on the recorded MWD data and the pictures taken in the tunnel (Figure 5.1). Figure 5.1 shows the recorded drilling data from 49 perimeter blast holes. After blasting, 21 half casts or barrels were observed in this tunnel section, resulting in a HCF of ~40%. Singh and Narendrula (2007) correlated the HCF with Rock Mass Rating (RMR) and showed that a 40% HCF corresponds to a RMR of 70, i.e. indicating good rock mass quality.

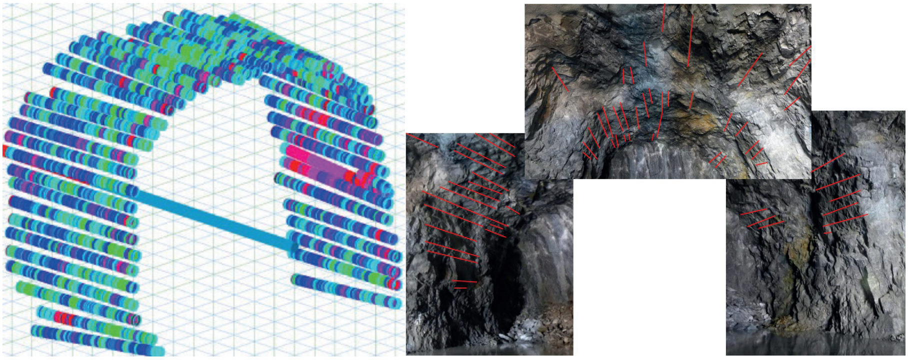


Figure 5.1 Blast holes and Half Casts in a section in Tunnel 213 (photographs modified after ÅF 2016).

Cavity Monitoring System

Tunnel scanning with the Cavity Monitoring System (CMS) gives volumetric calulation after excavation (including over-break). In the cases of the tunnels investigated in this report, CMS scans were performed on a regular basis, i.e., after every two to three blasts (10-15m of advance). Their volume was compared with the designed profile, the drilling plan and the drilling reports (Table 5.2). These reports included the hole location, direction and length. The differences between the tunnel design profile and the actual drill plan resulted in a 5.0% increase of rock volume excavated (Table 5.2). In addition, drilling deviation (the difference between the drill plan and the actual drill log) resulted in a 1.6% increase in volume, as shown in Table 5.2. This deviation included both collaring deviation and blast hole deviation along the drill hole. In this case study, the CMS scan did not collect data on the four to five bottom rows of the blast round, i.e. the last two to three metres of the tunnel, because the floor was covered with loose rock. The tunnel scan showed the over-break from the blasting and scaling was relatively low, 3.7%, outside the drill log report's perimeter (Figure 5.2). This indicated limited over-break failure outside the tunnel profile. Overall, there was a 9.1% increase in volume over the design profile (Table 5.2).

Table 5.2 Volumetric changes in Tunnel 214 (section 845 to 831) of the Stockholm bypass from the design profile to the post-blast results, including the scaling operations.

		Volume Deviation		
	Volume	Design profile	Drill Plan	Pre-Blast
Design Profile	1577m³	-	-	-
Drill Plan (incl. bottom holes)	1656m³	+79m³ (5.0 %)	-	-
Pre-Blast (incl. bottom holes)	1683m³	-	+27m³ (1.6%)	-
Pre-Blast (excl. bottom holes)	1633m³	+56m³ (3.6 %)	-23m³ (-1.4 %)	-
Post-Blast (excl. bottom holes)	1694m³	+117m³ (6.6%)	+ 38m³ (2.2%)	+61m³ (3.7%)
Estimated Total	1721m³	+144m³ (9.1%)	+ 65m³ (3.9%)	+38m³ (2.3%)

Figure 5.2 Over-break based on the drill log in Tunnel 214, from sections 845 (3m from tunnel entrance) to 831 (14m from tunnel entrance) and the volumetric scan (CMS).

Tunnel Mapping During the Excavation Cycle

Geological mapping of the tunnel surface was performed after each mucking cycle to produce updated geological maps of the tunnels. An example of these maps is given in Figure 5.3. The map shows geological structures (coloured areas), fractures (coloured lines) and areas with over-break (grey areas). Based on the mapping data, actual Q-values were determined for each section of the tunnel. The general mapping and the actual Q-values were used to identify areas where over-break and extensive fracturing had occured.

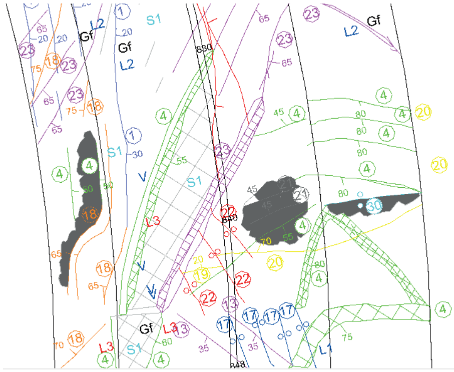


Figure 5.3 Geological mapping of Tunnel 214 between sections 848 and 827 at Stockholm bypass (modified after ÅF 2016).

Ground Penetrating Radar

Ground Penetrating Radar (GPR) data were collected in the four tunnels (Veidekke access tunnel, Tunnel 213, Tunnel 214 and TAS04). An example of these data is shown in Figure 5.4. This figure displays GPR data on the left wall of Tunnel 214 from section 848 to section 827. The image shows a significant energy loss and reflections of the signal in the first 20cm of the rock mass. The band of energy loss is caused by small reflections from micro fractures within the rock mass (Jern, 2001; Silvast and Wiljanen, 2008). The depth of this zone of dispersion is displayed by the blue dashed line in Figure 5.4. The red circles in the figure are wave reflections indicating macro fractures. Nearly all of these reflections could be related to the natural fractures shown in Figure 5.3, the numbering of the fractures is based on the fractures described during the mapping of the tunnel. The unidentified fractures in Figure 5.4 are most likely blast induced fractures. Based on these fractures the extent of the blast damage zone could be determined.

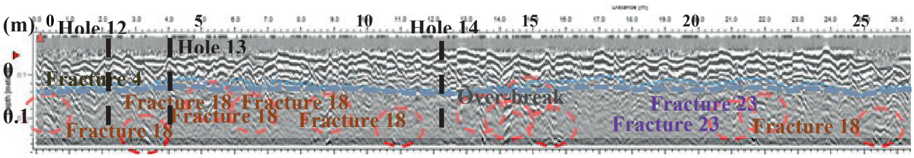


Figure 5.4 Ground Penetrating Radar showing recorded fractures (red circles) for left wall section 848, the tunnel portal, to section 827, 21m into the tunnel, in Tunnel 214 at the Stockholm bypass; the majority were related to fractured mapped, and others are likely to be caused by blasting. The blue line shows the zone of dispersion (micro fracture reflections) in the rock mass.

In this study, a total of 59 GPR data lines were collected. Out of these lines, 34 were analysed further for blast damage. The GPR data showed an EDZ ranging from 12cm to 30cm at the position of the string charge (Figure 5.5). The same data showed the position of bottom charges with a more extensive blast damage zone, from 25cm to 40cm (Figure 5.5). The extent of the GPR EDZ depth at the selected samples is indicated in Figure 5.6. This figure shows the influence of the rock type on the GPR EDZ depth. In general, the gneiss shows more extensive damage than the other rock types, likely because of its grain size, grain elongation and rock mass texture (foliation) (Howard and Rowlands, 1987).

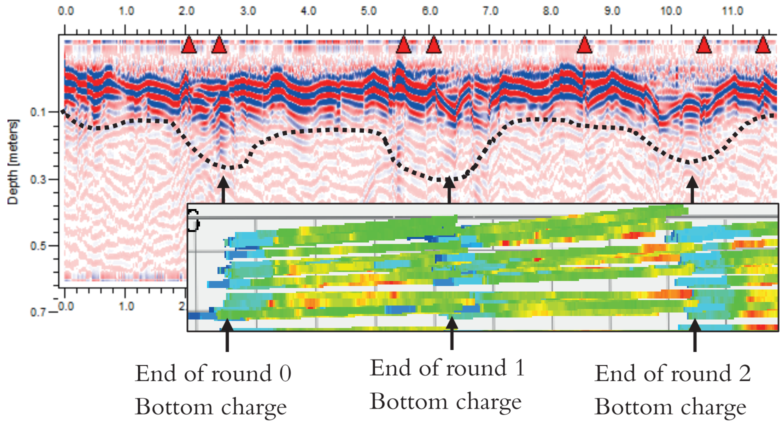


Figure 5.5 GPR and drilling data for the left wall of TAS04 tunnel showing an increase of the extent of the GPR blast damage at the drill hole bottoms with a higher charge concentration. The red triangles are marking points manual added during the recording and have no influence on the recorded data. The black dashed line is the interpretation of the extent of the blast damage zone.

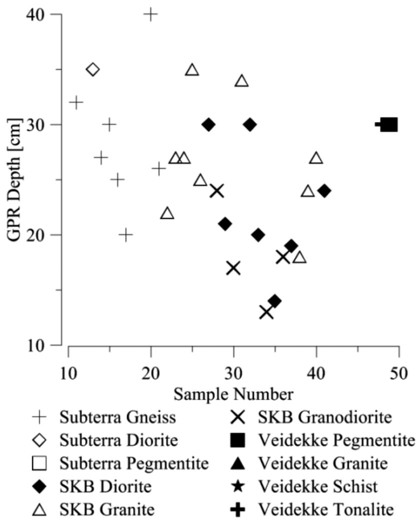


Figure 5.6 GPR reflection at depth due to blasting fractures of different rock types at the three investigated sites at the locations of the drill cores. The figure shows the influence of grain size on GPR recordings: pegmatite and gneiss have greater GPR depth than diorite and granodiorite.

Diamond Drill Cores

A total of 49 drill cores (DCs) were drilled in this project. These cores are shown by the red dots in Figure 5.7. A total of eight DCs (six in the tunnel and two in the carvern) were drilled at the Veidekke access tunnel (Figure 5.7A), eight at ramp Tunnel 214 (Figure 5.7B), 13 at ramp Tunnel 213 (Figure 5.7C) at the Stockholm bypass and 20 at the Äspö HRL TAS04 tunnel (Figure 5.7D). To select the locations of the DCs, variations in the Hardness and Fracture Indices were used as guides in the Veidekke access tunnel and the two ramp tunnels in the Stockholm bypass. In the TAS04 tunnel, the drill cores were drilled with a regular 3m spacing along the tunnel walls.

From the drilled 49 cores, 12 cores were selected for detailed analysis. Out of these 12, four cores were from Tunnel 214, between section 848 and section 827 (see Figure 5.8A) and eight from the Veidekke access tunnel (see Figure 5.8B).

The 12 drill cores extended to a depth of 40cm to 168cm into the rock mass (Figure 5.9). Figure 5.9 displays the different rock types and RQD observed along the two tunnels. The RQD values of the different rock are generally high, indicating competent rock masses. Seven out of the 12 drill cores have an RQD exceeding 70%. The drill cores show in general, more extensive fracturing at the start of the core hole, see for example in Figure 5.10. These fractures were newly formed (fresh fracture surfaces) in

Hole #1, Hole #2 and Hole #4 (Figure 5.10). They are most likely related to the excavation process and can therefore be used as an index of the depth of the EDZ. This depth is estimated to range from 10cm to 30cm based on this visual observation of the newly formed fractures in the drill cores. A more detailed description and comparison of these 12 cores is given in the next sections.

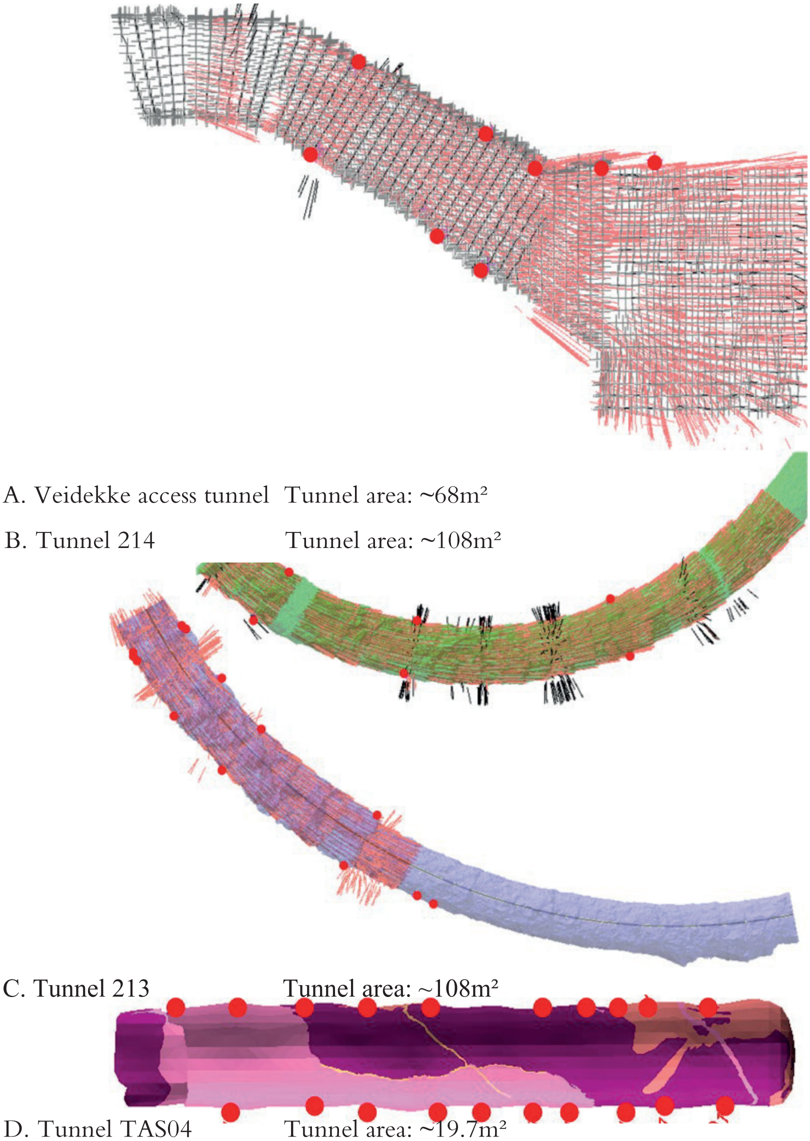
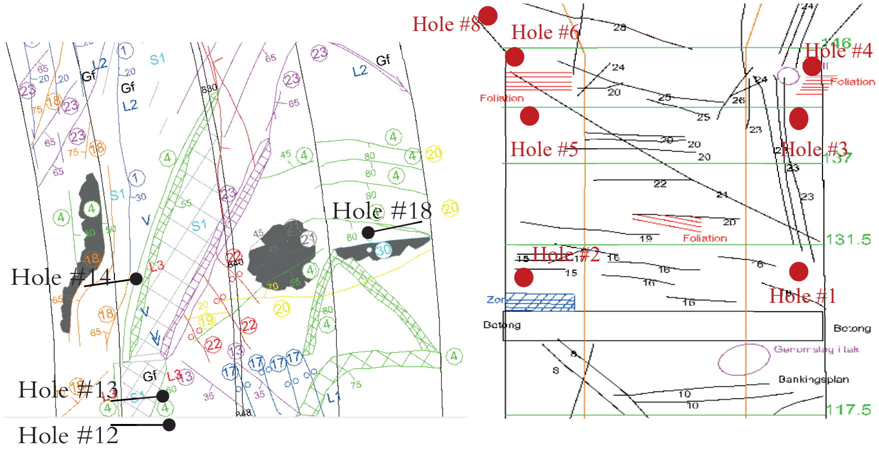


Figure 5.7 Location of diamond core holes in the four tunnels: eight drill cores in the Veidekke access tunnel (A), eight in Tunnel 214 (B), 13 in Tunnel 213 (C) and 20 in TAS04 tunnel (D).



A. Tunnel 214 B. Veidekke access tunnel

Figure 5.8 Fracture mapping and diamond core hole locations of Tunnel 214 (A) from the tunnel entrance (section 848) to 21m into the tunnel (section 827) and the drill cores taken in the Veidekke access tunnel (B) from section 117.5, 67.5m into the tunnel to the entrance to gallery at section 146, 96m into the tunnel.

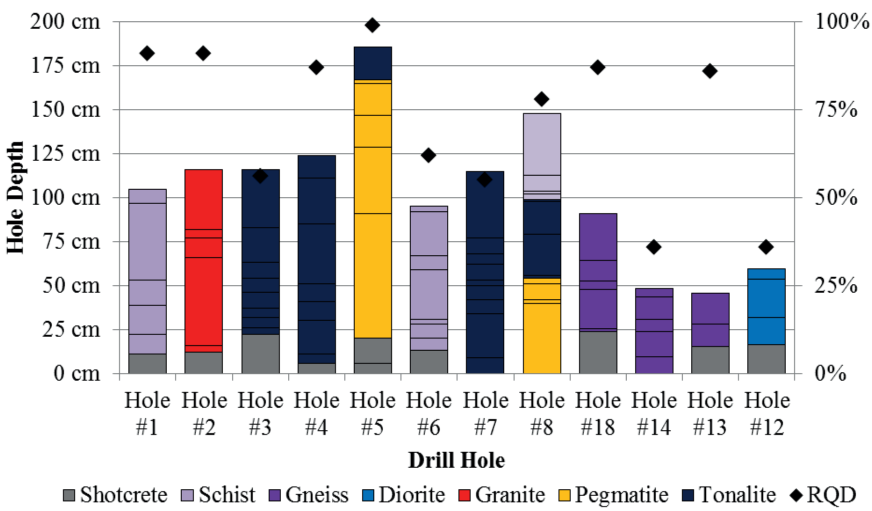


Figure 5.9 Drill core mapping from tunnel 214 and the Veidekke access tunnel for the blast damage investigation. The left axis displays the total core length; the black lines within the core display the separation of the core parts. The black diamonds in the figure show the RQD with their values on the right axis. The RQD values ranges from 30% to 100%. This value is a combination of natural and blast induced fractures.

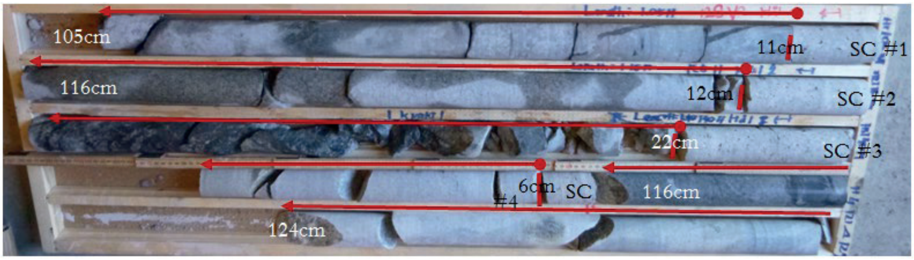


Figure 5.10 Cores from Hole #1 to Hole #4 from the Veidekke access tunnel. The cores are more extensively fractured at the start of the drill hole, most of the fractures were newly formed (SC = Sprayed Concrete).

Drill core #12 contained a highly fractured diorite, while drill core #13 contained almost unfractured gneiss (Figure 5.9 and Figure 5.11), even through the cores were located less then two metres apart. This highlights the significant variation in the rock mass quality. The majority of fractures in drill core #12 and drill core #14 were natural fractures and clay filled. Drill core #13 and drill core #18 had minor fresh fractures caused either by the excavation or by the core drilling itself (Figure 5.11). In the majority of the drill cores, the fracture density increased within the first 10cm, corresponding to an estimated EDZ depth of 10cm.

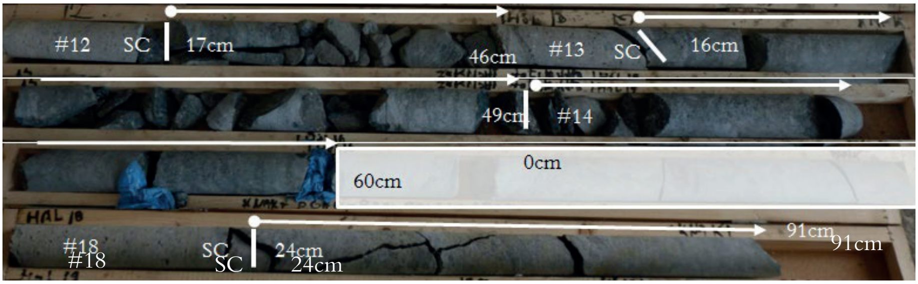


Figure 5.11 Cores from Hole #12, #Hole #13, Hole #14 and Hole #18 of Tunnel 214. The cores show a more extensive fracturing at the start of the drill hole, the majority of the fractures were newly formed (SC = Sprayed Concrete).

Furthermore, the cores from the Veidekke access tunnel had a high variation in rock mass quality (Figure 5.9 and Figure 5.10). Drill core #1, drill core #2 and drill core #5 showed favourable rock conditions; i.e. these cores were drilled in areas with relatively few fractures. Drill core #4 and drill core #6 showed a low RQD, likely caused by breaking along foliation in that area of the tunnel wall. Drill core #3 had an unfavourable rock condition, with several natural and clay-filled fractures observed in the rock mass. The two drill cores in the gallery (drill core #7 and drill core #8) were also affected by the pre-existing clay filled fractures observed by Karlsson (2015). The degree of fracturing in the core samples corresponded to the expectation of increased fracturing at the hole collar due to blasting. For these cores the EDZ was estimated as 10cm to 30cm.

The collected data from the drill cores at the three excavation sites (four tunnels) are summarised in Figure 5.12. Overall, the drill cores showed a wide range of rock types from large phenocryst pegmatite, fine-grained granite to foliated gneiss (Figure 5.14). The RQD ranged from 0% (naturally crushed rock) to 98% (solid rock) (see Figure 5.14). The black separation lines within the bars in Figure 5.12 display fractures observed during the core logging. The majority of these fractures had fresh fracture surfaces, induced during the excavation (see Figure 5.10, Figure 5.11 and Figure 5.13). The drill cores showed the effect of the grain size on the fracturing density. Fine grain material (e.g. granite) had a lower RQD, while large grain phenocrysts (pegmatite) had a higher RQD (Figure 5.14). This difference indicated that finer grained rock masses were more prone to blast damage. This was probably a result of the differences in required force for fracturing; separating grains and crystals requires less energy than breaking the grains and crystals (Howarth and Rowlands, 1987).

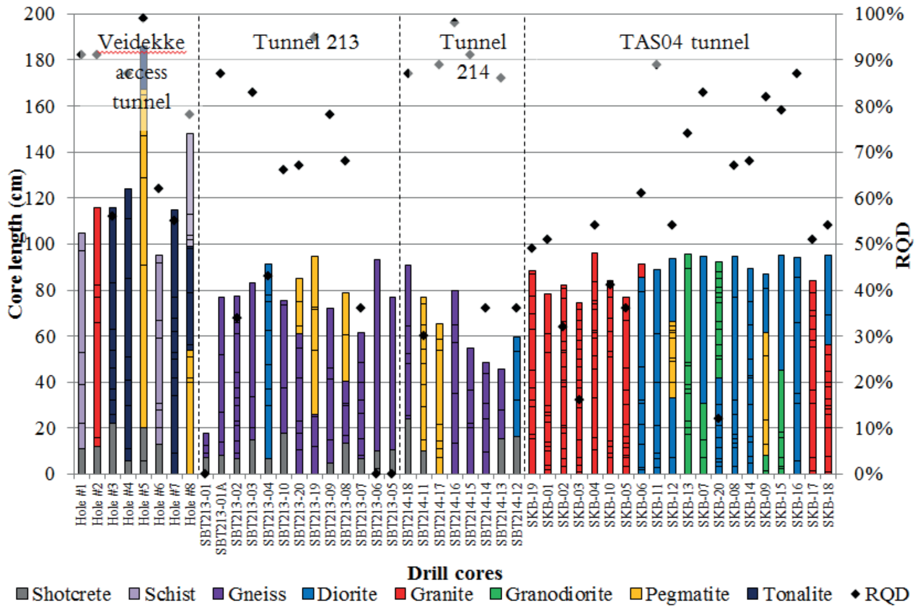


Figure 5.12 Drill core mapping the four investigated tunnels. The left axis displays the total core length; the black lines within the core display the separation of the core parts. The black diamonds in the figure show the RQD with their values on the right axis. The RQD values ranges from 0% to 100%. This value is a combination of natural and blast induced fractures.

Figure 5.13 Cores #12, #14, #15 and #16 from TAS04 tunnel. The cores show a more extensive fracturing at the start of the drill hole, these fractures were determined to be blast induced.

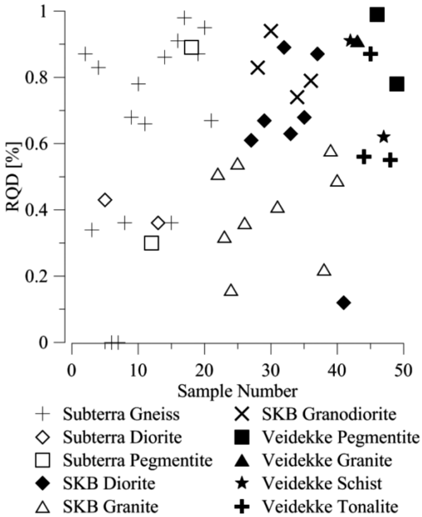


Figure 5.14 RQD per rock type for each sample collected in the four tunnels at the three investigated sites. The figure shows a lower RQD in fine-grained rock masses (granite and diorite) and higher RQD in large-grained rock masses (pegmatite). In addition, the variation in the RQD for the gneiss may show the effect of the orientation of a rock masses foliation. Where breakage along the foliation requires general less energy, on the other hand It may have be caused by variation of charge concentration.

P-Wave Velocity Measurements

Diametric P-wave measurements were taken for all cores, even though some parts of the cores were heavily fractured and could not be measured. A total of 527 P-wave velocity measurements were made. The P-wave velocity in the rock close to the contour was reduced (Figure 5.15), indicating a more fractured area likely caused by blasting. The P-wave velocity increased as the distance from the tunnel wall increased. This increase corresponded to the improving rock mass conditions and indicated progress into an undisturbed rock mass.

The transition point (or the threshold point) was determined for each drill core. At this point, the P-wave velocity levelled out, and from here on was not considered to be affected by blasting (no micro factures). The transition point was used to estimate the EDZ depth. The EDZ based on the P-wave velocity varied from 8cm to 45cm (Figure 5.15).

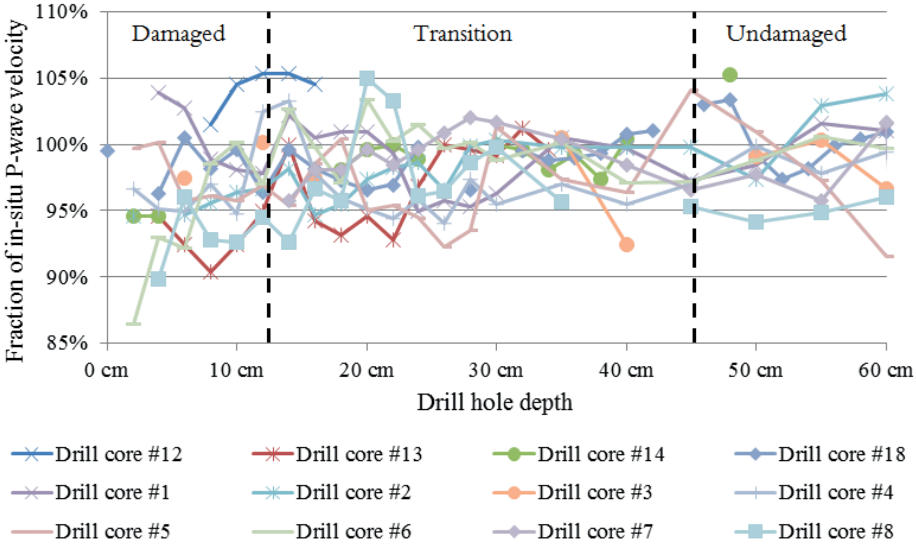


Figure 5.15 P-wave velocities displayed in percentages of the intact rock mass for the investigated drill cores. The missing samples indicate fracturing of the drill core. At the damaged area all the displayed drill cores show a significant reduction of P-wave velocity, at the undamaged area all the displayed drill cores show a stable P-wave velocity around a 100% of the fraction of the in-situ P-wave velocity. In between, the transition area, some of the core measurements show a reduced P-wave velocity (damaged) and some show a P-wave velocity close to the in-situ P-wave velocity (undamaged).

The average P-wave velocity behaviour for each of the excavation sites is displayed in Figure 5.16 (i.e. Veidekke access tunnel, two Stockholm bypass ramp tunnels and TAS04). The figure shows the 21 measurements in the ramp tunnels. It indicates a clear trend for the Stockholm bypass ramp tunnels. In these tunnels, undisturbed rock is reached at an average distance of 20cm from the tunnel wall. At this distance, the P-waves level out, indicating an EDZ depth of 20cm. A similar observation can be made for the eight drill cores from the Veidekke access tunnel, although the trend is not as clear as at the Stockholm bypass tunnels (Figure 5.16). The difference in the EDZ depth is probably caused by the limited number of drill cores and/or the electronic detonators used in blasting. This trend does not appear in the figure for the TAS04 tunnel at all. The reduced EDZ depth indicates an exceptional level of care and quality control during its excavation (Ericsson et al., 2015). A major part of the differences in the P-wave velocity trends between the sites is most likely related to the excavation and detonation method used. Pyrotechnical detonators were applied in all blast holes at the Stockholm bypass ramp tunnels, while electronic detonators were used in the contour holes at the TAS04 and the Veidekke access tunnel. The use of pyrotechnical detonators is known to result in a larger EDZ than simultaneous blasting with electronic detonators (Olsson and Ouchterlony, 2003; Ouchterlony et al., 2009; Ittner et al., 2018). Therefore, a lower EDZ depth is expected with electronic detonators.

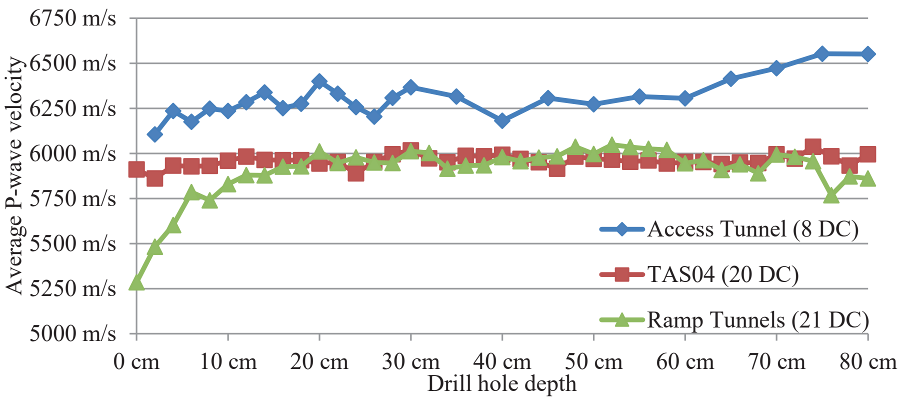


Figure 5.16 Average P-wave velocities at the drill cores at the three investigated sites. The lower velocities indicate damaged rock mass. The figure shows the effects of the initiation system, whereby the Veidekke access tunnel and TAS04 tunnel used electronic detonators (simultaneous initiation) in the perimeter holes and the two Stockholm bypass ramp tunnels used pyrotechnical (“*non-el*”) detonators (time scatter in initiation) in the contour holes. This results in more extensive average blast damage in the Stockholm bypass ramp tunnels than the Veidekke access tunnel and TAS04 tunnel.

The P-wave velocity thresholds (limits of EDZ depth) for all 49 investigated cores are presented in Figure 5.17. The individual threshold values vary significantly, from 2cm to 46cm, indicating a significant influence of varying in-situ conditions. These differences might be related to the influence of the rock types on the measured EDZ depth or the rock mass behaviour during blasting. The effects of the rock types can be seen in this case study; the large grain rock types (pegmatite) seemed to be more prone to micro fracturing during blasting than fine grained rock types (e.g. fine grained granite), as shown in Figure 5.17. This behaviour was previously noted by Howarth and Rowlands (1987).

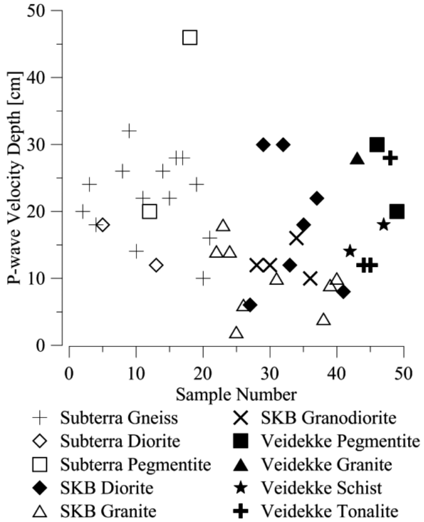


Figure 5.17 Depth of blast damage based on P-wave velocity measurements for different rock types at the sites investigated. Lesser extent of the EDZ is observed in the fine-grained granite.

5.3 Rock Mass Characterisation in Tunnel Investigations

The rock mass was characterised by the initial Q-values obtained from the site investigation. Initial Q-values between 0.7 and 12.2 were recorded along the span of Tunnel 213 and Tunnel 214 (Figure 5.18). During the tunnel construction, the actual Q-values were obtained from the surface mapping of the tunnel perimeter (ÅF, 2016). These actual Q-values were much lower than the initial Q-values (Figure 5.18). The initial Q-values were used to determine the required rock support for the different tunnel sections (see Table 4.2. and Figure 5.19). During the excavation, the actual Q-values were applied to adjust the initial rock support design.

The initial Q-values obtained from the site investigation were compared to the actual (mapped) Q-values obtained from mapping during tunnel excavation. The Q-values for the first 100m of Tunnel 213 (Figure 5.18A) were ten times lower than expected (sections 200 to 258, 58m), with ratios up to 180 times lower (sections 210 to 212). The initial Q-values for the first 220m of Tunnel 214 (Figure 5.18B) were also significantly lower (up to three times lower in sections 849 to 847 and sections 800 to 755). In Tunnel 213, 87% of the observed sections had Q-value two times lower than expected; see Figure 5.18A. In this tunnel, 63% of the sections had Q-value at least ten times lower than expected. In Tunnel 214 (Figure 5.18B), the Q-value of 49% of the observed sections were two times lower than expected. Based on these lower values, it can be concluded that the site investigation significantly overestimated the rock mass quality for these tunnels. As a result of this overestimation, the rock support in both tunnels had to be increased (Figure 5.19 and Figure 5.20). To this end, the bolt spacing was reduced (from selective bolting to 1.5m spacing) and twice the amount of planned shotcrete was used, and 200 mm shotcrete arcs were installed at the tunnel entrances (Figure 5.19 and Figure 5.20). Unexpectedly, the bolt length was increased (from 3m to 6m) in the least favourable parts of the tunnel (especially in Tunnel 213). This extended bolt length is not conforming to the Q-system. In the Q-system the bolt length is only depending on the span with or height divided by the Excavation Support Ratio (ESR) (Barton et al. 1974).

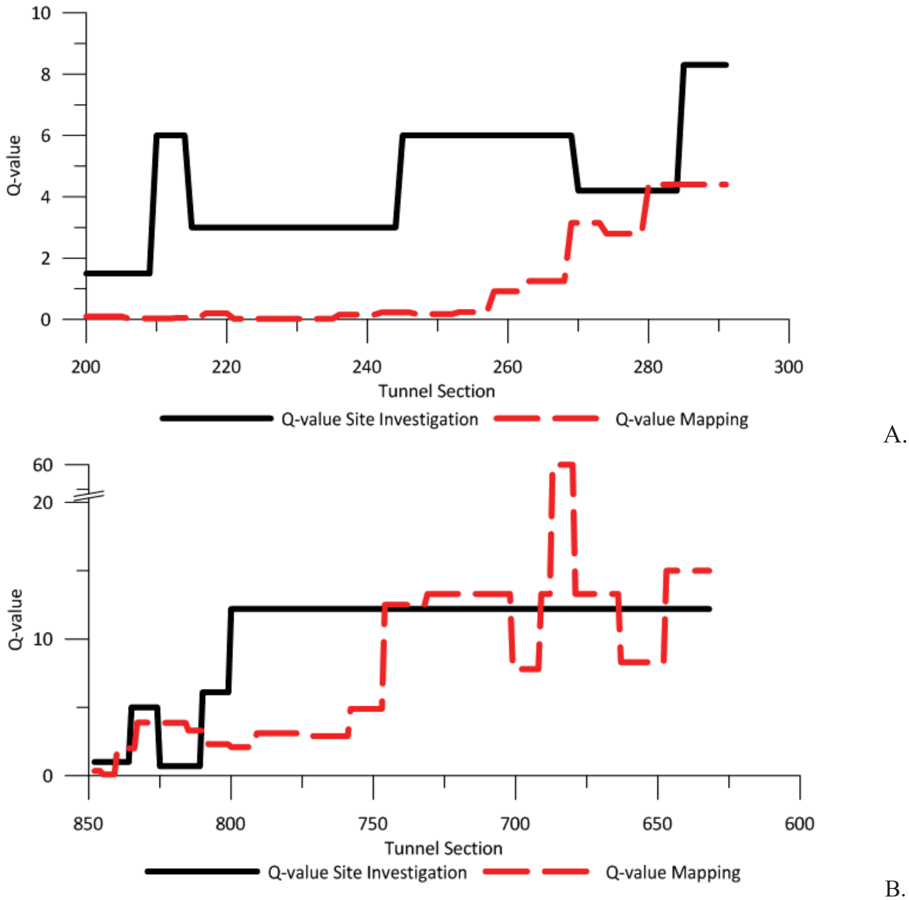
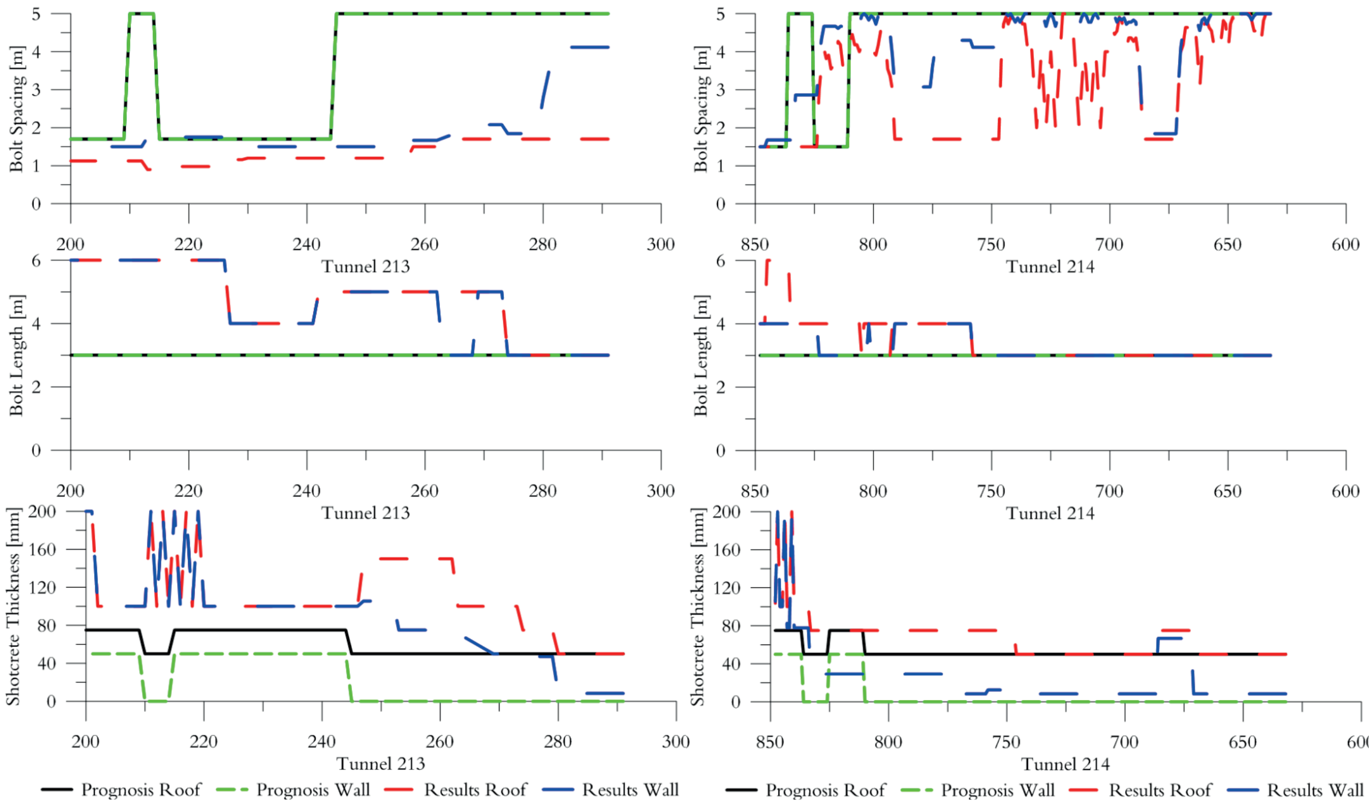


Figure 5.18 Prognosis and realisation of rock mass quality in (A) the first 100 m of Tunnel 213 and (B) the first 220 m of Tunnel 214.



A. B.

Figure 5.19 Prognosis and installed rock support in Tunnel 213 (A) and Tunnel 214 (B) (Selective bolting “ spacing” =5 m).

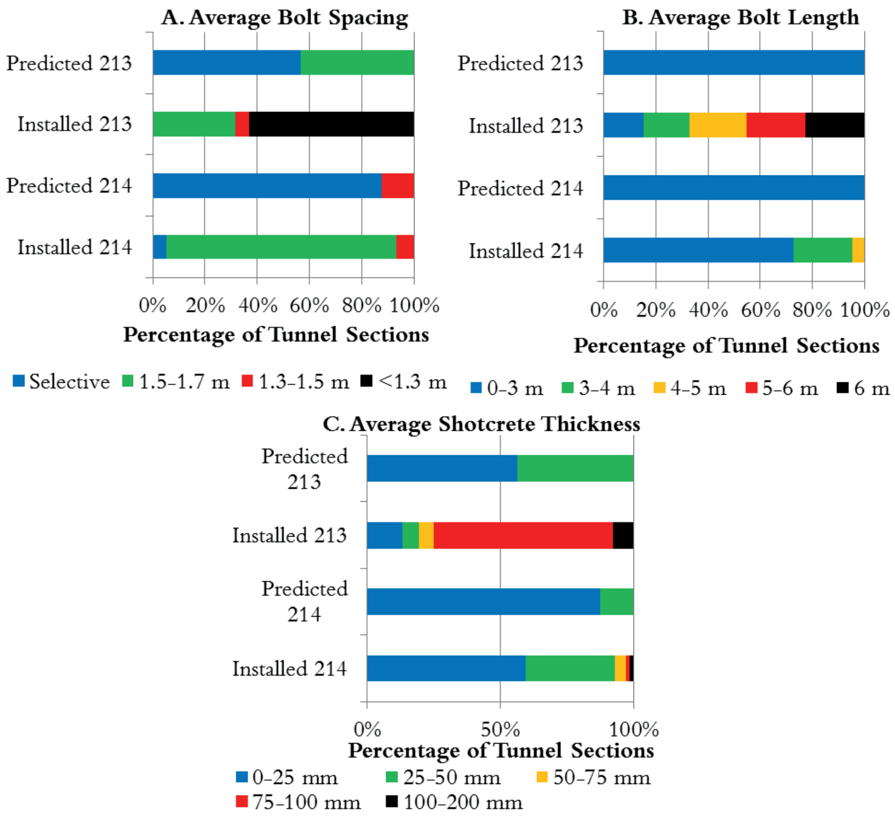


Figure 5.20 Predicted and installed bolt spacing (A), bolt length (B) and shotcrete thickness (C) in Tunnel 213 and Tunnel 214. The length is defined as the percentage of tunnel section, i.e., 90m for Tunnel 213 and 220m for Tunnel 214.

In this study the extended bolt length was caused due to up to 4.5-metre zones with alternating graphite and clay bands. In between these zones the rock mass was observed to be consisting of small blocks and oxidised (ÅF 2016). This graphite and clay rich deformation zone was not deem suitable for anchorage of rock bolts. To overcome this safety risk, longer 6-metre bolts were selected to secure proper rock bolt anchorage and thus rock support. Therefore, the increased bolt length is correlated to the rock mass quality in this case study. In addition, the effects of the graphite-clay zones required the installation of 200mm thick sprayed concrete arches at the tunnel entrances.

5.4 Use of MWD Data to Characterise Rock Mass

Measurement While Drilling data can be used to characterise the rock mass. The following four-step process was used undertaken on the MWD data from the two ramp tunnels at the Stockholm bypass and the Veidekke access tunnel:

1. Filtering MWD data (removing outliers);
2. Normalising MWD data (influence of hole length and inter parameter correlation);
3. Comparing MWD Indices for grout and blast holes;
4. Validating MWD interpolation against mapped rock mass conditions.

Filtering MWD Data

In the first step, outliers in the MWD data must be removed from the original data set. The MWD data must be filtered to remove faulty and improbable data. Examples of these faulty data include negative rotation speeds (reverse rotation) and very high penetration rates, e.g. rates over 48m/min or 0-values among the drilling parameters. The drilling data set will also have data that are correct but unlikely. In this case, there is a sliding transition from faulty data to abnormal drilling behaviour. Abnormal behaviour may be caused by drilling operations procedures, e.g. drill hole collaring and rod changes, and will cause problems filtering data. Fortunately, the data density for MWD is high. Therefore, a conservative statistical filtering procedure was used to remove the outliers without losing the general pattern of the data. More specifically, the highest and lowest values were removed from the data set. Ultimately, 99% of the data points were preserved, with removing the lower and higher 0.5% of the MWD data. If one or more of the values at the sample point fell outside the interval, the entire sample was rejected. The filtering process for the data gathered at the ramp tunnels is shown in Table 5.3.

Table 5.3 Filter limits for the MWD parameters in Tunnel 213 and Tunnel 214.

Recorded parameters	Ranges of recorded raw data	Selected filter limit
Penetration rate (m/min)	0 and 48.8	≥0 and ≤7.5
Percussive pressure (bar)	0 and 215	≥110 and ≤200
Feed pressure (bar)	0,42 and 192	≥20 and ≤90
Rotation pressure (bar)	1,28 and 162	≥35 and ≤100
Rotation speed (RPM)	-196 and 374	≥170 and ≤340
Damper pressure (bar)	9.75 and 176	≥35 and ≤90
Flushing water pressure (bar)	0 and 122	≥8 and ≤35
Flushing water flow (L/min)	0 to 261	≥60 and ≤210

Normalising MWD Data

After filtering, the data are normalised for drill operational dependencies, e.g. drill hole length and inter parameter correlations, as well as machine influences. The process of normalising data is described by Schunnesson (1996; 1998), Ghosh (2017) and Navarro et al. (2018c). For this report, the normalisation was performed in UM (Epiroc, 2018b). The percussive energy, the rotation energy and flushing become less effective at depth. Since the friction along the drill hole increases between the rod and hole wall and between the cuttings left in the holes, these influences have to be removed before using the data to characterise rock mass.

Comparing MWD Indices for Grout and Blast Holes

Recorded MWD responses often vary significantly with hole length and hole diameter. An example is the penetration rate: this parameter is higher for smaller, shorter holes and lower for larger, longer holes. In this case, the analysis concentrated on the MWD data from the drill holes at the two ramp tunnels of the Stockholm bypass. At the two other sites (Veidekke access tunnel and TAS04), limited to no grout hole MWD data were collected. At the bypass tunnels, two types of holes were drilled: blast holes with a diameter of 48mm and a length of ca. 5.7m and grout holes with a diameter of 64mm and a length of 20-25m.

In the third step, the Fracture Index and Hardness Index distributions of grout and blast hole MWD were numerically compared at the two ramp tunnels (Tunnel 213 and Tunnel 214). This comparison was followed by a visual interpretation of the interpolated Fracture Index and Hardness Index at the two ramp tunnels at the Stockholm bypass.

The Hardness Index and Fracture Index were calculated using the filtered and normalized MWD parameters in UM. The distributions of the indices are plotted in Figure 5.21A and Figure 5.21D. As the figures show, the distributions of the grout and blast holes differed significantly. Next, the indices of both hole types were normalised. The normalisation process is displayed in Figure 5.21 and explained below:

1. Calculate the mean or median (for symmetrically distributed data) or the mode (for asymmetrically distributed data) and standard deviation of the Fracture Index distribution and the Hardness Index.
2. Normalise the Fracture Index and normalise the Hardness Index of grout and blast holes (for other MWD parameters, use the residual) with the standard score using Equation 1 and Equation 2 (Figure 5.21B and Figure 5.21E). This scaling makes the comparison of the two Fracture Indices possible.

3. Scale the populations of the grout and the blast holes proportionally (Figure 5.21C and Figure 5.21F) for a better comparison of the hole types (only required if there is a large numerical difference between the populations).

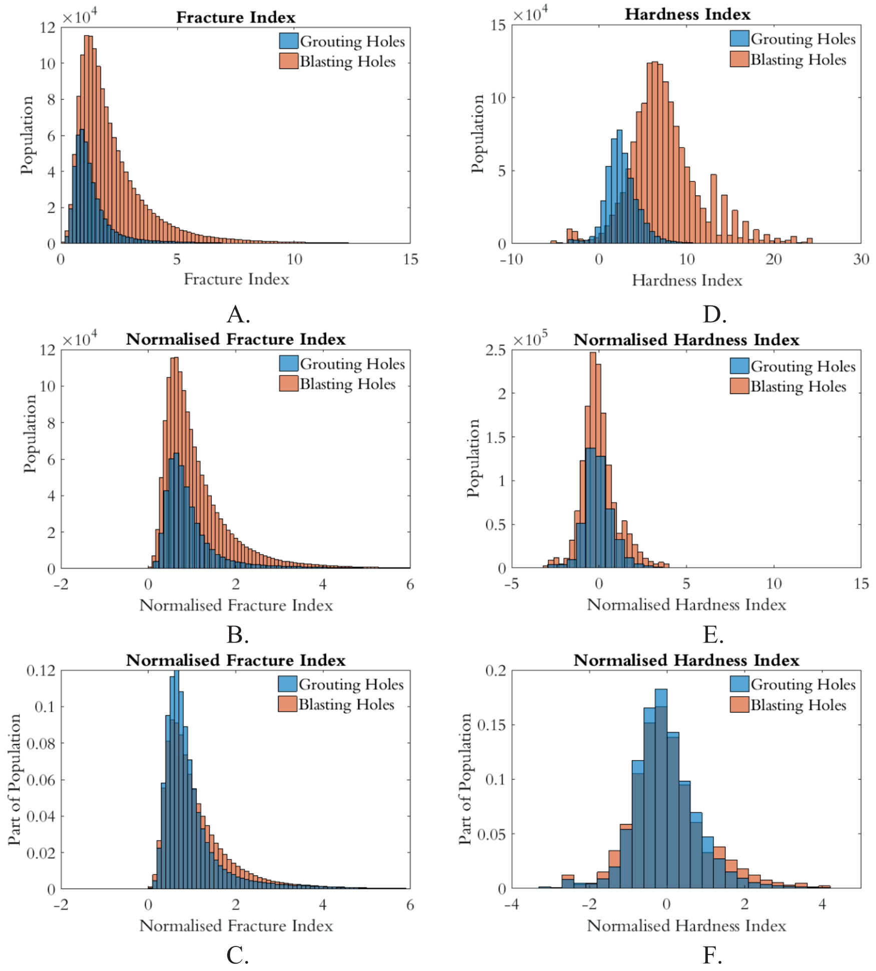


Figure 5.21 Normalisation procedures for Fracture Index and Hardness Index for MWD data for Tunnel 213 and Tunnel 214 at Stockholm bypass. The raw Fracture Index (A) and the normalised data (B) show a similar distribution for the grouting and blasting hole Fracture Index (C). The raw Hardness Index (D) shows a normal distribution (E); the distribution is similar for grouting and blasting (F).

$$\text{Normalise Fracture Index}_i = \frac{\text{Fracture Index}_i - \text{mean (mode) Fracture Index}}{\sqrt{\sigma_{Fracture\ Index}}^2} \qquad \text{Equation 1}$$

$$\text{Normalise Hardness Index}_i = \frac{\text{Hardness Index}_i - \text{mean } (mode)\text{Harness Index}}{\sqrt{\sigma_{Hardness\ Index}}^2} \qquad \text{Equation 2}$$

The normalised distributions of the Fracture Index for the blast holes and grout holes showed similar distributions for the grout and blast holes (Figure 5.21C). The normalised distribution for the Hardness Index had a similar distribution (Figure 5.21F). The normalisation was carried out using the standard deviation. The Residual Index was not calculated as the data set had a very similar response.

Validating MWD Interpolation Against Mapped Rock Mass Conditions

In the fourth step, the interpolated MWD Fracture Index is visually compared to the geological mapping. The Fracture Indices were interpolated for the grout holes (Stockholm bypass tunnels) and blast holes (Stockholm bypass tunnels and Veidekke access tunnel). Geological mapping was done in the two ramp tunnels (ÅF, 2016) and the Veidekke access tunnel (Karlsson, 2015) and the fractures were mapped for these three tunnels. In addition, the interpolated Fracture Indices for the blast holes in the ramp tunnels were compared with the mapped Q-values at the same location (Figure 5.25).

The visual comparison requires a holistic approach, in that the actual general geo-mechanical structure is visually compared with the interpolations of the Fracture Index and Hardness Index. This analysis was performed on the plots from the interpolated MWD of both grout and blast holes along the tunnel contours of Tunnel 213, Tunnel 214 and the Veidekke access tunnel.

In Tunnel 213 (Figure 5.22), the Fracture Indices showed highly fractured areas at the tunnel portal (section 200 to section 265), in section 230 to 232 and section 238 to 260. Similar fractured areas were observed during the geotechnical mapping of the tunnel (ÅF, 2016). The main fracture zones were on a 30° angle from the tunnel centre line. These areas are denoted by the black lines in Figure 5.22. The accentuated areas show a high level of similarity across the three data sets. The Hardness Indices for the grout and blast holes for Tunnel 213 are displayed in Figure 5.23. Here, the pattern of the Hardness Indices is similar to that of the Fracture Indices (Figure 5.22), correlating the two Indices. In addition, Tunnel 213 shows “harder” rock masses further in the tunnel. This was later confirmed by an inspection of the tunnel which discovered a change of rock mass in the tunnel.

In Tunnel 214, fracture zones were observed at the tunnel portal (the first 10m of the tunnel) and in sections 850 to 835 (Figure 5.24). These fracture areas locations and the orientations are denoted by the black lines in Figure 5.24. These areas showed a similar geotechnical structure in the three data sets, as also seen in Tunnel 213.

Within the two tunnels, the Indices showed similar behaviour. In general, the mapping of tunnels 213 and 214 showed a strong resemblance with the fracture zones displayed in the MWD data. The discussed fracture zones were observed in the drilling data (Figure 5.22 and Figure 5.24). In this case, the fracture zones were correlated to graphite occurrences in the tunnels (hashed pattern in the figures). These occurrences were observed as highly fractured in the MWD Fracture Index (Figure 5.22 and Figure 5.24).

In both tunnels, the mapped rock mass characterisations are better portrayed by the blast hole MWD data than the grout MWD data. This is caused by the geometry of the drilling fans, where the grout holes are located up to 5-metre from the tunnel perimeter and the blast holes are located at the tunnel perimeter.

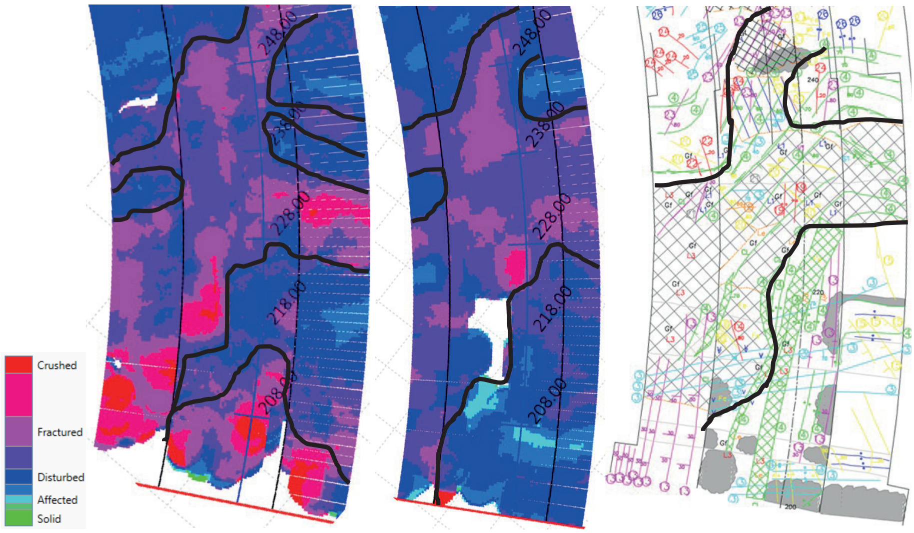


Figure 5.22 Fracture Index and geotechnical mapping for section 200 to section 250 (50m) in Tunnel 213 showing grout holes, blast holes and mapping. The black lines show the shape and orientation of the fracture zones.

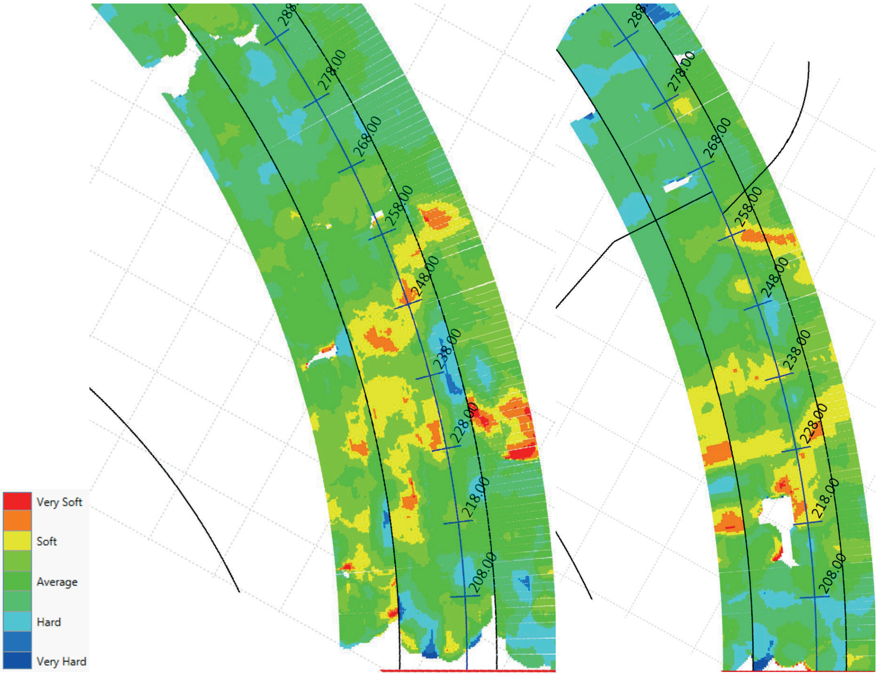


Figure 5.23 Hardness Index for section 200 to section 290 (90m) in Tunnel 213 showing grout holes on the left and blast holes on the right. Softer rock is massed in section 218 to section 243 (25m).

Figure 5.24 Fracture Index for section 848 to section 820 (28m) in Tunnel 214 showing grout holes on the left, blast holes in the middle and mapping on the right. The black lines indicate the shape and orientation of the fracture zones in the MWD data and tunnel mapping.

In addition to the visual comparison, the mapped Q-value was compared to the Fracture Index for the 24 sections in Tunnel 213 and Tunnel 214, as shown in Figure 5.25. The figure indicates that lower Q-values were correlated to higher Fracture Index values. This correlation confirms the observed behaviour and visual correlation between the Fracture Index and the rock mass conditions.

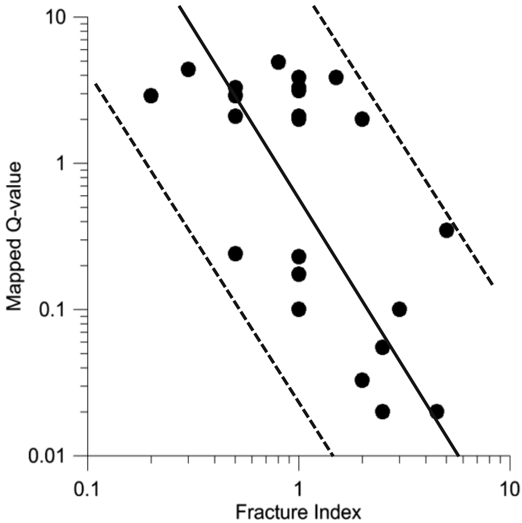


Figure 5.25 Relation of mapped Q-value and MWD Fracture Index on a log-log scale.

The Fracture Index and the geotechnical mapping were also compared for the Veidekke access tunnel. Unfortunately, in this tunnel, only limited data from the grout holes were available. Therefore, only the blast hole data were analysed. Figure 5.26 compares the Fracture Index, the Hardness Index and tunnel mapping of the access tunnel. The black circle in the figure indicates a highly “fractured” area, noted in the MWD data. In fact, this “fractured” area occurred when operators drilled through the tunnel’s roof into the soil above (Figure 5.26). The “fracturing” was indicated as crushed rock in the MWD data. Another fractured zone was observed on the east wall of the tunnel; this is indicated by the red dashed circle in Figure 5.26. This zone was indicated as crushed and soft rock (high drillability) in the MWD data. A similar indication was found during the tunnel mapping. The fractured areas resulted in poor rock mass conditions. In this tunnel, these unfavourable rock mass conditions required additional rock support. Lastly, the orange squares in Figure 5.26 show foliation perpendicular to the drilling direction. This foliation consisted of various layers, causing variations in the drilling parameters. The variations in the drilling parameters appeared as increased fracturing in the MWD data.

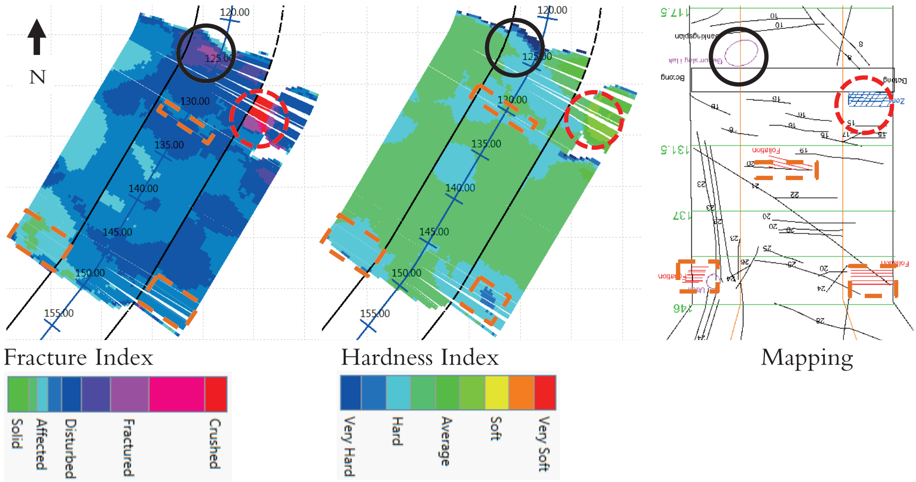


Figure 5.26 Fracture and Hardness Indices based on the grout holes, and mapping of the access tunnel; the encircled areas indicate fractured areas and the rectangles show the effects of the foliation.

5.5 Application of MWD Data in Rock Support Design

Today, the final rock support in tunnel excavation is adjusted or, in extreme cases, redesigned on an ad-hoc basis for specific sections of a tunnel. This updated design is directly based on the mapping of the walls and roof. In some cases, the redesign renders the original support design obsolete. However, the mapping data are gathered during the excavation cycle, so there is little time to adjust or redesign the preliminary rock support system. In the next step for this work, the Fracture Indices from the Stockholm bypass ramp tunnels and the Veidekke access tunnel were compared to the installed rock support.

Comparison of MWD Data and Installed Rock Support

The MWD Fracture Index from the blast holes and the installed rock support at 24 selected locations along Tunnels 213 and 214 were analysed. This analysis can be found in Table 5.4, Table 5.5 and Figure 5.27. In these sections, the rock mass conditions varied considerably, resulting in a large diversity of rock support installed. The rock support in each tunnel section often had a combination of different bolt lengths, different bolt spacing and different shotcrete thickness, as displayed in Table 5.4 and Figure 5.27.

The calculated Fracture Index is plotted against the installed rock support in Figure 5.27. The points in this figure represent the average value of the rock support and

Fracture Index for each section. Figure 5.27 compares the MWD Fracture Index and installed rock support. It shows a correlation in both poor and favourable rock conditions. In poorer rock conditions (high Fracture Index), the bolt spacing decreases and the shotcrete thickness increases. In favourable rock mass conditions (low Fracture Index), there is much less rock support. In the unfavourable rock mass, the bolt spacing decreases more than 30%, the average bolt length is two times longer, and the average shotcrete thickness increases more than 2.5 times. The figure also shows a clear trend in the data; an increased Fracture Index value corresponds to an increased demand for rock support. Based on this correlation, the domains of different support parameters can be established. These domains can be used to select the most suitable rock support based on the Fracture Index as the tunnels continue. Figure 5.27 shows several sections with a low Fracture Index but and relative thick sprayed concrete layer and tight bolt spacing. This phenomenon suggests the rock mass in some sections may be over-supported.

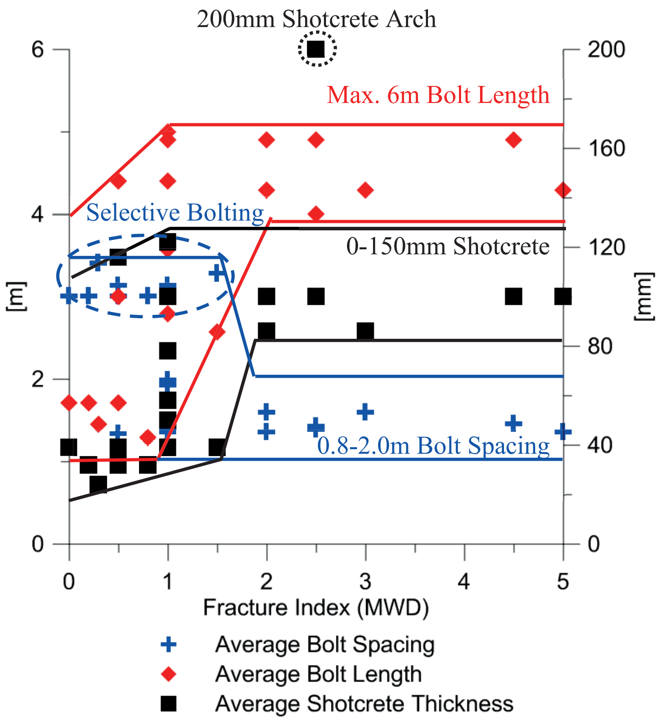


Figure 5.27 Correlations between the Fracture Index and installed rock support in 24 sections along Tunnel 213 and Tunnel 214.

Table 5.4 Fracturing from MWD, observations and rock support in Tunnel 213 and Tunnel 214.

Tunn. Sect.	Fractures MWD (Fracture Index avg.)	Fracture Mapping (Q-value)	Support Class, see Table 5.5			Remarks
213-			Shotcrete Thickness	Bolt Length	Bolt Spacing	
205	Yes (1.0)	Yes (0.1)	0-100mm	4-6m	< 1.5m	Probably fracture #13 (left and right in grouting MWD)
210	Yes, left wall (2.0)	Yes (0.033)	0-100mm	4-6m	< 1.5m	Small fracture #3 and water containing crush zone, L3 zone right wall
215	Yes (2.5)	Yes (0.055)	75-200mm	4-6m	< 1.5m	Fracture zone L3 interacting with fracture #3, whole tunnel
225	Yes (4.5)	Yes (0.02)	0-100mm	4-6m	1.5-2.0m	Fracture zone L3 interacting with fractures #4, #18, #20 and #19
235	Yes (2.5)	Yes (0.02)	0-100mm	4-5m	< 1.5m	Structure and fracture #4
242	Yes, roof (1.0)	Yes, roof (0.23)	0-100mm	4-5m	< 1.5m	In the right wall fractures #4, in the roof an interaction fractures #4, #23 and #24
250	Yes, roof (1.0)	Yes (0.175)	75-150mm	4-5m	< 1.5m	Left roof structure V1 intersects with fractures #23, #27 and structure S1
255	Yes, right wall (0.5)	Yes, right wall (0.24)	75-150mm	4-5m	< 1.5m	Roof and left wall fractures #23, #26, #28 and #29 interact with structure V1
270	Yes, local, roof (1.0)	Yes, local roof (3.15)	0-100mm	0-4m	1.5-2.0m	(Left) roof, structure S1, fractures #23, #28 and #31
275	No, local, roof (1.0)	No (2.0)	0-75mm	0-3m	1.7-2.0m	Walls fractured; fractures #23
280	No (0.3)	No (4.4)	0-75mm	0-3m	1.7-2.0m	Left roof; rock fall
214-						
848	Yes (5.0)	Yes (0.35)	75-200mm	4-6m	1.5-2.0m	Fracture/crushed zone in the right roof of the tunnel portal, fractures #17
840	Yes (3.0)	Yes (0.1)	75-200mm	4-5m	1.5-2.0m	Fracture zones #4 and #18; rock fall in the left wall
835	Yes, right wall (2.0)	Yes (2.0)	0-100mm	4-5m	1.5-2.0m	Rapid following, parallel fractures #4 in the right wall
825	Local (1.0)	Local (3.88)	0-75mm	3-4m	Selective	Fractures #23 observed in the right wall
820	Minor (1.5)	Yes (3.87)	0-75mm	0-4m	Selective	Fracture zone #23 in the right roof and wall, #26 and #1 in roof
815	No (0.5)	Local (3.3)	0-75mm	0-4m	Selective	Fracture zones #26
810	Local (1.0)	Local (3.3)	0-75mm	0-4m	Selective	Frequent fractures #27 and #29 in the right roof
800	No (0.5)	No (2.1)	0-75mm	0-4m	Selective	
795	Local, right roof (1.0)	Local, right roof (2.1)	0-75mm	0-4m	Selective	Crush and fracture zone V1 in the right roof
785	No (0.0)	No (3.1)	0-75mm	0-4m	Selective	
775	Minor (0.2)	Minor (2.9)	0-75mm	0-4m	1.7-2.0m	Rapid following, parallel fractures #28 in the right roof and wall and #34 in left
765	Minor (0.5)	Minor (2.9)	0-75mm	0-4m	1.7-2.0m	Some fractures #34 in the roof
755	Minor right roof (0.8)	Minor (4.9)	0-75mm	0-3m	1.7-2.0m	Fractures #26, #28 and #29 and S1-S2

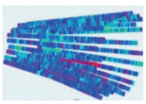
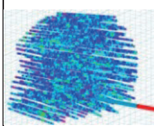
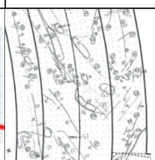
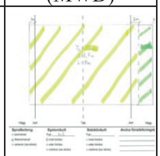
Table 5.5 Rock and support classes at the Stockholm bypass (modified after Arghe, 2013).

Rock Class	Q-value	Rock Quality	Shotcrete	Bolt Length	Bolt Spacing
I	Q > 10	Very good	0-50 mm	0-3 m	Selective
II	4 < Q ≤ 10	Good	0-75 mm	0-4 m	>2.0 m
III	1 < Q ≤ 4	Acceptable	0-100 mm	3-4 m	1.7-2.0 m
IV	0.1 < Q ≤ 1	Poor	75-150 mm	4-5 m	1.5-2.0 m
V	Q ≤ 0.1	Very poor	75-200 mm	4-6 m	< 1.5 m

5.6 Recommended Procedure for Support Design

Section 5.3 discusses the shortcomings of the rock support design process and Section 5.4 describes the potential to apply MWD in rock mass characterisation. Section 5.5 addresses the correlations between the MWD data (Fracture Index) and installed rock support. Based on this combined analysis, an alternative process to improve the rock support design during tunnel construction can be recommended. This novel procedure incorporates MWD data from both grout and blast holes, and tunnel mapping of the rock mass to implement rock support during excavation. The procedure is shown in Table 5.6 and described at greater length in the remainder of this section.

Table 5.6 Rock mass characterisation information sources and proposed use of MWD data during tunnel excavation to optimise the rock support decision-making process.

Process Stage	Site investigation (Geotechnical Prognosis)	Probing & Grouting	Drilling & Blasting	Rock Quality Investigation	Support Decision
Information Source	Desk Study/ Outcrops/ Core Drilling/ Seismic Lines/ Nearby Tunnels	MWD Grout & Probe Holes Volume Injected	MWD Blast Holes	Tunnel Mapping	Rock Mass Quality, Mapping, Desk Study, (MWD)
Example					
Application (Decision)	Preliminary Design/ Rock Class	Adjustments to Geotechnical Prognosis	Minor Adjustments to Geotechnical Prognosis	Rock Mass Classification (Rock mass Support)	Bolt Spacing & Length, Shotcrete Thickness, Spiling
Decision-making Interval	>-1 Year	-7--1 Days	-4-0 Hour	+4-+12 Hour	-

In this approach the original rock support design is based on the information gathered from the site investigation. During tunnel excavation, probe and/or grout holes are drilled 15-25m ahead of the face, and MWD data are logged. These data can be used to characterise the rock mass and adjust the original rock support design based on new observations. The blast holes are drilled (5-6m), and MWD data from these holes are used to verify the expected rock mass conditions. The verification data are used to alter the rock support design if necessary. Finally, the excavated tunnel is mapped, and the rock support design is updated. In each of these steps in the decision-making process, the accuracy of the information increase, but the available time for decision making decreases. This new approach reduces the risk and gives the opportunity to prepare for unexpected rock mass conditions, thus reducing excavation delays. Hence, the usage of MWD data in the rock support design may result in a more effective workflow. The proposed procedure gives time and opportunity to adjust and alter the support design, when and where necessary.

The following analysis explains how MWD data can be used in a tunnelling project to enhance the data collected earlier during the site investigation and consequently improve the rock support design. The proposed procedure provides an accurate prediction of the rock mass conditions and discontinuities ahead of the tunnel face, using data from both the grout and blast holes. The MWD data can be used to optimise the rock support design before blasting. As Table 5.6 shows, that this approach could reduce the risk of unexpected, poor ground conditions ahead of the tunnel face. The use of MWD for rock mass characterisation can provide a better understanding of the rock mass ahead of the tunnel face. This knowledge, in turn, might help to reduce the installation time of rock support, thus reducing the tunnel excavation cycle and possibly reducing the total rock support excavation costs.

5.7 Use of MWD Data for Blast Damage Evaluation

Discontinuities are known to influence over-break and the Excavation Damage Zone. The information on rock mass discontinuities may help in evaluating blast damage. Blast damage is influenced by many different factors, including blast planning, drilling parameters, explosive properties and rock mass properties (e.g. Olsson and Ouchterlony, 2003; Ouchterlony et al., 2009). The parameters of blast planning, drilling parameters and explosive properties are generally known, and the unknown operational information can be gathered quickly from operational procedures, drilling logs or the supplier's specifications. However, rock mass properties and hydrogeological conditions on a hole-by-hole basis are often unknown. Measurement While Drilling data might be able to supply this information. The drillability of the rock mass is likely related to the fracture toughness of the rock mass, and this parameter could be determined with the Hardness Index or penetration rate. The degree of fracturing could be investigated using the Fracture Index and incorporating previous geological and geotechnical knowledge on the rock mass. The condition of water in the drill hole could be determined by drill hole geometry, e.g. angle from the horizon, and the water pressure and water flow during drilling. The combined data might be used to predict the depth of the damage zone in the rock mass.

In the case study, the EDZ was determined based on GPR data, RQD data and P-wave velocity thresholds. The correlation between the measured extent of the blast damage and the recorded MWD data was investigated with Multiple Linear Regression (MLR). The blasting damage measurement depended on the site and local conditions. The number of data points used appear in Table 5.7. The raw MWD parameters were obtained for all drill holes, but the calculated parameters (Hardness Index and Fracture Index) were only determined for the holes drilled with an Atlas Copco drill rig (Veidekke access tunnel, Tunnel 213 and Tunnel 214). The aim of the MLR was to predict the extent of the damage zone based on the MWD parameters. In addition to the MWD parameters, certain design parameters, i.e. planned contour charges for the blast hole collar (no charge), pipe (0.35kg/m) and bottom (1.2kg/m), rock cover, cross section area and the contour spacing of each tunnel section, were taken into account; see Table 5.8. The MLR was used to obtain the constants in Equation 3 and Equation 4. The equations were applied to the obtained excavation parameters and compared with the determined extent of the blast damage. This comparison is plotted in Figure 5.28.

Table 5.7 Number of data points collected and used for Multiple Linear Regression analysis of blast damage and MWD parameters.

Number of data points	Veidekke access tunnel	Stockholm bypass tunnels	SKB TAS04	Total
GPR	2	8	20	30
P-wave velocity thresh hold	8	18	20	46
RQD	8	20	20	48
MWD raw	8	21	20	49
MWD calculated (Epiroc)	8	21	0 (Sandvik)	29
GPR-MWD raw	2	8	20	30
P-wave-MWD raw	8	18	20	46
RQD-MWD raw	8	20	20	48
GPR MWD calculated	2	8	0 (Sandvik)	10
P-wave MWD calculated	8	18	0 (Sandvik)	26
RQD-MWD calculated	8	20	0 (Sandvik)	28

$$Blast\ Damage_{Raw\ MWD} = K_1 * Charge\ Concentration + K_2 * Penetration\ Rate + K_3 * Feed\ Pressure + K_4 * Rotation\ Speed + K_5 * Water\ Flow + K_6 * Rotation\ Pressure + K_7 * Rock\ Cover + K_8 * Tunnel\ Area + K_9 * Contour\ Spacing \qquad \text{Equation 3}$$

$$Blast\ Damage_{Calculated\ MWD} = K_1 * Charge\ Concentration + K_2 * Hardness\ Index + K_3 * Fracture\ Index + K_4 * Rock\ Cover + K_5 * Tunnel\ Area + K_6 * Contour\ Spacing$$

Equation 4

The GPR blast damage depth showed a relatively good correlation with both raw MWD and the design parameters (R^2: 0.673 and R^2: 0.578, respectively; see Table 5.8 and Figure 5.28A). The most significant parameters based on the *p-value* (<5%) were flushing water flow (0.4%), charge concentration (0.7%), rock cover (1.6%), rotation speed (2.5%) and tunnel area (3.1%). The application of the calculated MWD parameters (Table 5.8) also showed a good correlation between the GPR blasting depth, the MWD and design parameters (R^2: 0.578). Here, the significance of the input parameters was low (*p-value* >5%).

The P-wave velocity damage depth showed a significantly lower correlation with the input parameters than the GPR-based damage depth, for both the raw MWD parameters (R^2: 0.363, Table 5.8 and Figure 5.28B) and the calculated MWD parameters (R^2: 0.107, Table 5.8 and Figure 5.28B). The statistical model also failed to identify significant input parameters for the correlation of the MWD data with the P-wave velocity damage depth.

The RQD along the drill hole displayed a medium correlation with the raw MWD data (R^2: 0.338, Table 5.8 and Figure 5.28C) and with the calculated MWD parameters (R^2: 0.359, Table 5.8 and Figure 5.28C). The MLR showed the significant parameters for the raw MWD parameters were dominated by the design parameters, i.e. tunnel cross section (1.4%), rock cover (1.5%) and charge concentration (4.8%). In addition, the feed pressure had a significance of 4.8%. For the calculated MWD parameters the *p-value* showed only high significance of the design parameters, i.e. tunnel cross section (4.0%), rock cover (4.7%) and charge concentration (3.0%), as displayed in Table 5.8.

The measured blast damage zone based on GPR data, P-wave velocity and the RQD showed a large variation from the anticipated damage zone. However, it still suggested the blast damage was significantly influenced by the charge concentration and the contour hole spacing (Table 5.8). This observed influence agrees with studies by Olsson and Ouchterlony (2003), Ouchterlony et al. (2009) and Ittner et al. (2018).

The statistical analysis showed correlations between MWD and the measured excavation damage, especially the damage indicated by the GPR. The variation in the measurements of the excavation damage could be explained by the design parameters; see Table 5.8. These findings are similar to those of earlier studies on over-break (Mohammadi et al., 2017; Navarro et al., 2018c) and the EDZ (Olsson and Ouchterlony, 2003; Ouchterlony et al., 2009). Using MWD data and design parameters to predict blast damage in underground excavations has clear potential.

Note that the effects of other parameters, such as initiation method (Olsson and Ouchterlony, 2003; Ouchterlony et al., 2009; Ittner et al., 2018; Figure 5.16), fracture toughness, rock mass texture (Howarth and Rowlands, 1987) and contour hole spacing, were not studied within this project and are not included in the prediction model. In addition, the collected data for this study do not include drill hole deviation, detonation sequence and timing, water in the drill holes or the effects of the distribution of emulsion within the blast holes. These parameters are all known to influence blast damage (Ittner et al., 2018).

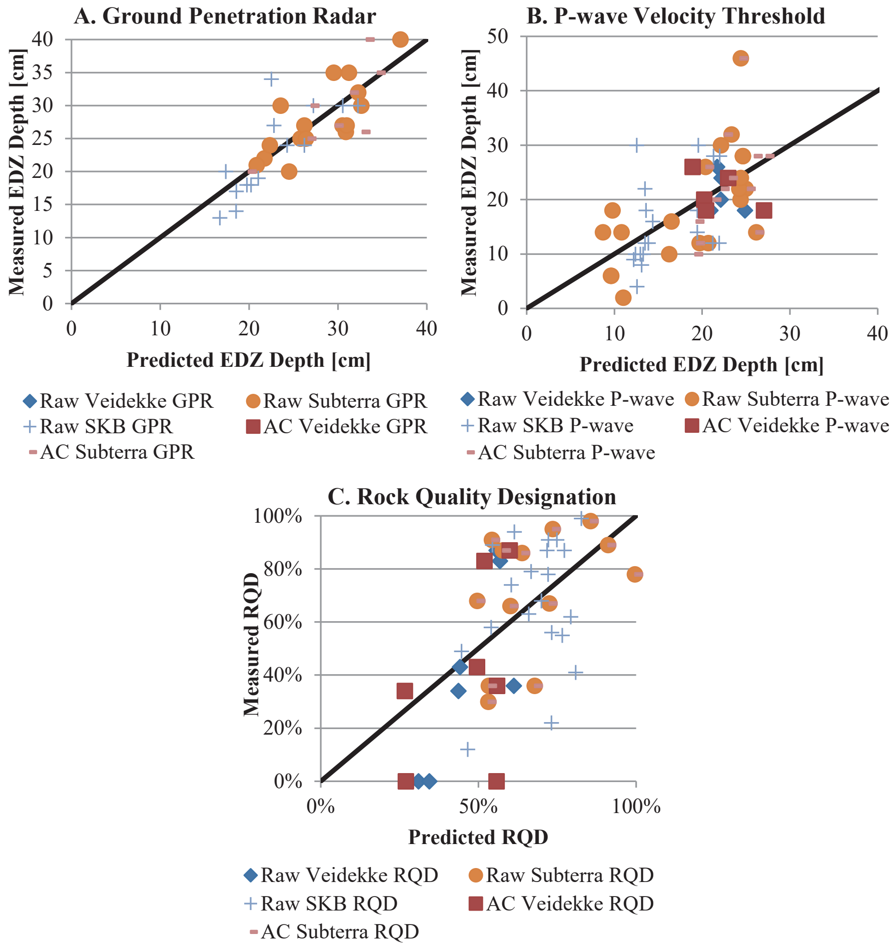


Figure 5.28 Correlation of the measured and predicted (by the factors derived from MLR analysis) extent of the blast damage at the three sites based on the GPR (A), P-wave velocity (B) and RQD (C).

Table 5.8 Multiple linear regression of blast damage and MWD parameters, top: Raw MWD parameters; Bottom: Calculated MWD parameters, including the estimated factor, the likelihood of a parameter having influence on the resulting factor (*p-value*) and the coefficient of determination (R^2).

Raw MWD Constants Equation 3	GPR [cm]	R^2: 0.673	P-wave Velocity Threshold [cm]	R^2: 0.363	RQD [%]	R^2: 0.338
	Estimate	p-Value	Estimate	p-Value	Estimate	p-Value
(Intercept)	-15.022	0.427	-13.476	0.543	-0.450	0.465
Charge Concentration [kg/m]	7.111	**0.007**	-0.584	0.870	-0.212	**0.048**
Penetration rate [cm/min]	-0.044	0.063	0.020	0.498	-0.001	0.180
Feed Pressure [bar]	0.220	0.057	-0.054	0.721	0.008	**0.048**
Rotation speed [r/min]	0.135	**0.025**	0.064	0.300	0.001	0.625
Water Flow [L/min]	0.155	**0.004**	-0.019	0.792	0.003	0.112
Rotation Pressure [bar]	-0.091	0.425	-0.029	0.871	-0.001	0.908
Rock Cover [m]	-0.021	**0.016**	0.019	0.160	0.001	**0.015**
Tunnel Area [m²]	-0.070	**0.031**	0.089	0.071	0.004	**0.014**
Contour Spacing [m]	-2.932	0.836	11.843	0.446	-0.177	0.706

Calc. MWD Constants Equation 4	GPR [cm]	R^2: 0.578	P-wave Velocity Threshold [cm]	R^2: 0.107	RQD [%]	R^2: 0.359
	Estimate	p-Value	Estimate	p-Value	Estimate	p-Value
(Intercept)	82.491	0.105	-10.229	0.714	-0.429	0.605
Charge Concentration [kg/m]	16.792	0.286	-3.143	0.681	-0.440	**0.030**
Hardness Index	-0.804	0.442	0.177	0.850	-0.011	0.685
Fracture Index	-0.018	0.997	1.877	0.595	0.018	0.863
Rock Cover [m]	-0.095	0.262	0.049	0.315	0.003	**0.047**
Tunnel Area [m²]	-0.348	0.256	0.169	0.294	0.010	**0.040**
Contour Spacing [m]	-13.288	0.643	13.674	0.532	0.044	0.946

5.8 Limitations of MWD Data in Tunnel Excavation

The MWD data have limitations and must be carefully applied. The comparison of grout and blast hole MWD indicates differences in the resolution of the data sets. The grout hole drilling gathers data earlier in the excavation process and further ahead in the tunnel. The blast hole data give a better resolution because of the drilling geometry; the blast holes are tightly spaced and less interpolation is needed between the sample points. The grout MWD data give smoother values and therefore lose the particularities of the rock mass. In addition, the method of obtaining the MWD data has to be taken into consideration. The grout hole drilling can occur 5m outside the tunnel contour. This deviation from the tunnel can give inaccurate data in heterogeneous rock masses.

In this study, MWD data were collected in a Scandinavian hard rock mass. In softer, sedimentary or heavy weathered rock masses, the proposed system might be less accurate. The study shows that rock mass properties, e.g. foliation and crystal texture, affect the drilling parameters. This influence might also be seen in layered sedimentary rock masses, e.g. sandstone, limestone etc. In addition, the data collected during these studies have a high resolution, with samples recorded every 2cm to 3cm. In many other studies, the collected data might have a much lower resolution; in some cases 10cm to 20cm. A sparser setting might miss the nuances of the rock mass, because the MWD data are smoothed. Furthermore, Measurement While Drilling data require interpretation, based on previous knowledge or assumptions on the rock mass, ideally collected during the site investigation or during the tunnel excavation. Therefore, MWD data should be seen as an additional data source, not a replacement for tunnel mapping.

6 CONCLUDING REMARKS

Chapter 1 provides the four research questions that served as the basis for this report. This chapter discusses and answers the questions based on the literature study and the analysis of the gathered field data.

RQ1: How can the extent of excavation damage be measured?

The study shows multiple methods are applicable to the investigation of blast damage. The methods range from affordable and indicative to expensive and accurate measurements. The selection of the appropriate methods depends on the goal of the measurements, available funds and time. The majority of the methods do not affect the excavation process significantly. These non-interrupting methods can be applied easily, and by using a combination of several methods, accurate results can be achieved at lower costs. An overview of some of the methods appears in Table 5.1.

RQ2: How can drill monitoring data be used for rock mass quality assessment?

The site investigation gathers the available data on the rock mass prior to excavation. It gives general input for construction design, preliminary rock support and the tender process, but it lacks important details and is imprecise for large parts of the excavation. The unreliability of the geological and geomechanical properties has far-reaching effects, e.g. delays and cost increases. Therefore, updates for the rock mass characterisation are needed. This study shows MWD is a reliable data source and can reduce the risks involved in a tunnel excavation. MWD can assess the rock mass quality accurately; areas with many fractures and fracture zones are well portrayed. The drilling data give accurate locations and orientations of fracture zones. The data can be provided by both grout and blast holes. This study shows that a holistic approach renders good results; small errors are less dominant, as the method focuses on the general behaviour of the data. MWD data give a direct indication of the implications of the fracture zones; these are not immediately clear in mapping. Consequently, MWD can be used directly during excavation, due to its accuracy, simplicity, and clarity. However, MWD shows the effects of the rock mass on the drilling parameters, and interpretation of the data is required. Therefore, MWD should be used to back up the rock quality assessment; for example, it could be incorporated within the observational method. This study shows there is a correlation between MWD, rock mass quality and the installed rock support. This correlation can be applied to the rock mass support design. In this case, the normalised MWD data are an objective and reliable parameter, even though the data require verification. MWD data provide information within the tunnel contour, the blast holes, and beyond the contour, the grout holes, virtually expanding the on-site geologist’s or engineering geologist's field of vision. The additional knowledge might result in superior rock support design, and should, therefore, be incorporated into the rock support decision-making process, as proposed in this report. The use of this technology could provide a quick information flow and foster faster decision making.

RQ3: How can rock mass characterisation based on drill monitoring be used to improve the rock support design process?

The study shows the opportunities and benefits of drill monitoring (MWD) for rock support determination. The use of the MWD parameters reliably predicts the rock mass conditions ahead of the face, especially when the uncalibrated Fracture Index is used. This Index shows a good correlation with the Q-value and with the installed rock support in the form of bolt length, bolt spacing and shotcrete thickness. The proposed method gives an opportunity to incorporate drill monitoring into the rock support decision-making process by updating the existing information with the calculated drilling Indices. The technology provides an objective and reliable assessment of the rock mass conditions and has great potential to optimise the rock support installation process by verifying of the rock mass before the round blast.

RQ4: To what extent can excavation damage be predicted by using rock mass characterisation based on drill monitoring?

The extent of the fractures in the rock mass is affected by the explosive properties, blast design and rock mass properties. Excavation damage refers to the development of micro and macro fractures. Highly fractured rock mass will cause over-break when the jointing and fractures of the rock mass interact with the blast-induced fractures. This work shows the extent of EDZ is influenced by the rock mass properties. These effects can be measured with MWD parameters. Besides the drilling recordings, operational parameters have a large influence on the blast damage, e.g. the initiation system, specific drilling and specific charge, as well as geological features, e.g. grain size, fracture toughness and rock mass texture. The Multiple Linear Regression models show the MWD parameters describe the GPR EDZ depth reasonably well. In contrast the MLR models were less effective to describe the P-wave velocity threshold and the RQD. To create an improved EDZ prediction model, other factors of influence, e.g. initiation system and geological rock mass properties, should be investigated and incorporated.

7 FUTURE WORK

The blast damage investigation has shown a good indication of the rock mass status with respect to the blast damage. In the future, additional tests on the drill core samples can verify the results of this study. The investigation of the micro fractures in the drill core can be enhanced by additional P-wave velocity measurements on water saturated cores. These tests determine the Poisson ratio, and the effect of water inside micro fractures under freezing and thawing conditions can be studied. The use of polished core sections can further improve the results of this study. A fracture count can be established in the polished sections, verifying the P-wave velocity measurements.

Measurement While Drilling has not been used to its full potential, even though it has been available on drill rigs since the 1990s. MWD data are mainly collected in case of future liability investigations. Otherwise, MWD data are used to follow up on the tunnelling quality and operator performance, e.g. drill hole collaring, drill hole deviation. In some cases, MWD data are used for local rock mass characterisation and grouting optimisation. Hopefully, MWD will be used in the near future for continuous rock mass characterisation, blastibility and over-break investigation. The MWD data seem to be a good predictor of the rock mass quality and rock support requirements, but more case studies should investigate the validity of this finding. The rock support-MWD correlation should be tested in multiple case studies and in different rock mass conditions, e.g. sedimentary rock masses, as well as heterogeneous rock masses. Furthermore, uses of MWD data should be expanded to include evaluation of grouting performance and rock support design, and other possible applications, such as blast damage investigation and fragmentation control. A future model for these applications should include drill hole deviation, drill plan design, fixed explosive properties and geological parameters. The focus of MWD data for this purpose should be:

- Fracture toughness of the rock mass (drillability or Hardness Index)
- Degree of fracturing of the rock mass (Fracture Index)
- Water filled blast holes (Water Index, hole location, and hole direction)

Lastly, in this study, the rock mass indicated an influence of the grain size and rock texture on the blast damage. These effects should be quantified in future studies. The incorporation of the extent of fracture, the fracture toughness, rock texture, grain size, water content and initiation system into the blast damage prediction models should also be investigated.

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APPENDICES

Appendix A: Blasting Technology

Appendix B: Rock Mass Classification

APPENDIX A: BLASTING TECHNOLOGY

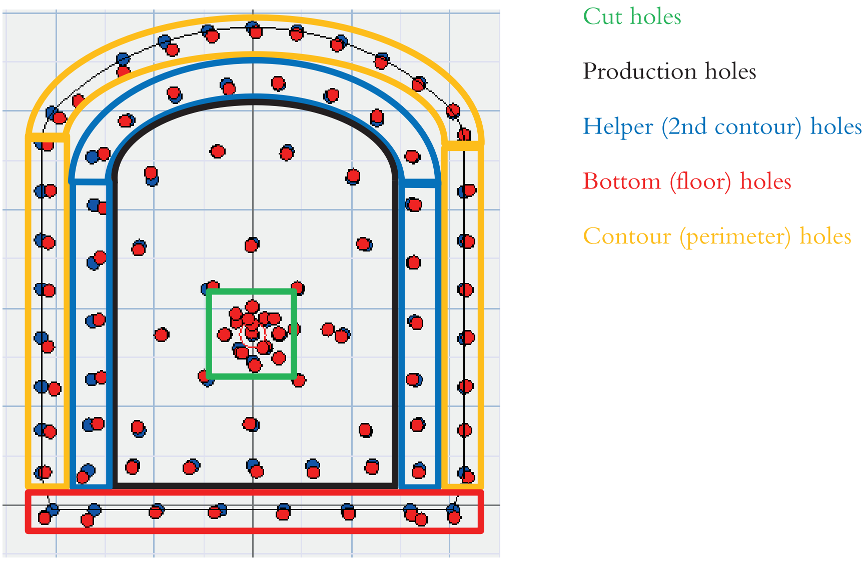

Figure A.1 Schematic layout of a blast hole drilling round for a tunnel blast. The blue dots show the drill plan and the red dots the drilling performance.

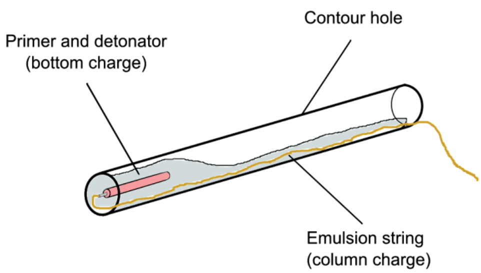

Figure A.2 Blast hole string or column and bottom charge in a contour hole (Ittner et al., 2018).

APPENDIX B: ROCK MASS CLASSIFICATION

Rock Mass Rating (RMR) System (Bieniawski, 1973)

Table B.1 Rock Mass Rating (RMR) classification (Modified after Bieniawski, 1989).

<table>
<tr><td>A</td><td colspan="9">Classification parameters and their ratings</td></tr>
<tr><td colspan="3">Parameter</td><td colspan="7">Range of values // ratings</td></tr>
<tr><td rowspan="3">1</td><td rowspan="2">Strength of intact rock material</td><td>Point-load strength index</td><td>>10MPa</td><td>4-10MPa</td><td>2-4MPa</td><td>1-2MPa</td><td colspan="3">Too low to measure</td></tr>
<tr><td>Uniaxial compressive strength</td><td>>250MPA</td><td>100-250MPa</td><td>50-100MPa</td><td>25-50MPa</td><td>5-25 MPa</td><td>1-5 MPa</td><td><1 MPa</td></tr>
<tr><td colspan="2">Rating</td><td>15</td><td>12</td><td>7</td><td>4</td><td>2</td><td>1</td><td>0</td></tr>
<tr><td rowspan="2">2</td><td colspan="2">Drill Core quality (RQD)</td><td>90-100%</td><td>75-90%</td><td>50-75%</td><td>25-50%</td><td colspan="3"><25%</td></tr>
<tr><td colspan="2">Rating</td><td>20</td><td>17</td><td>13</td><td>8</td><td colspan="3">5</td></tr>
<tr><td rowspan="2">3</td><td colspan="2">Spacing of discontinuities</td><td>>2m</td><td>0.6-2m</td><td>200-600mm</td><td>60-200mm</td><td colspan="3"><60mm</td></tr>
<tr><td colspan="2">Rating</td><td>20</td><td>15</td><td>10</td><td>8</td><td colspan="3">5</td></tr>
<tr><td rowspan="2">4</td><td colspan="2">Condition of discontinuities</td><td>Very rough surface
Not continuous
No separation
Unweathered wall rock</td><td>Slightly rough surfaces
Separation <1mm
Slightly weathered walls</td><td>Slightly rough surfaces
Separation <1mm
Highly weathered walls</td><td>Slickensides surfaces/Gouge <5mm/Separation 1-5mm continuous</td><td colspan="3">Soft gouge >5mm thick / Separation >5mm Continuous</td></tr>
<tr><td colspan="2">Rating</td><td>30</td><td>25</td><td>20</td><td>10</td><td colspan="3">0</td></tr>
<tr><td rowspan="4">5</td><td>Ground water</td><td>Inflow per 10m tunnel length</td><td>None</td><td><10L/min</td><td>10-25L/min</td><td>25-125L/min</td><td colspan="3">>125L/min</td></tr>
<tr><td>OR</td><td>Ratio joint water pressure/major principal stress</td><td>0</td><td>0.0-0.1</td><td>0.1-0.2</td><td>0.2-0.5</td><td colspan="3">>0.5</td></tr>
<tr><td>OR</td><td>General conditions</td><td>Completely dry</td><td>Damp</td><td>Wet</td><td>Dripping</td><td colspan="3">Flowing</td></tr>
<tr><td colspan="2">Rating</td><td>15</td><td>10</td><td>7</td><td>4</td><td colspan="3">0</td></tr>
</table>

<table>
<tr><td>B</td><td colspan="6">Rating adjustment for joint orientations</td></tr>
<tr><td colspan="2">Strike and dip orientations of joints</td><td>Very favourable</td><td>Favourable</td><td>Fair</td><td>Unfavourable</td><td>Very unfavourable</td></tr>
<tr><td rowspan="3">Ratings</td><td>Tunnels</td><td>0</td><td>-2</td><td>-5</td><td>-10</td><td>-12</td></tr>
<tr><td>Foundations</td><td>0</td><td>-2</td><td>-7</td><td>-15</td><td>-25</td></tr>
<tr><td>Slopes</td><td>0</td><td>-5</td><td>-25</td><td>-50</td><td>-60</td></tr>
</table>

<table>
<tr><td>C</td><td colspan="5">Rock mass classes determined from total ratings (sum)</td></tr>
<tr><td>Rating</td><td>100-81</td><td>80-61</td><td>60-41</td><td>40-21</td><td><20</td></tr>
<tr><td>Class number</td><td>I</td><td>II</td><td>III</td><td>IV</td><td>V</td></tr>
<tr><td>Description</td><td>Very good rock</td><td>Good rock</td><td>Fair rock</td><td>Poor rock</td><td>Very poor rock</td></tr>
</table>

<table>
<tr><td>D</td><td colspan="5">Meaning of rock mass classes</td></tr>
<tr><td>Class number</td><td>I</td><td>II</td><td>III</td><td>IV</td><td>V</td></tr>
<tr><td>Average stand-up time</td><td>10 years
15m span</td><td>6 months
8m span</td><td>1 week
5m span</td><td>10 hours
2.5m span</td><td>30 minutes
1m span</td></tr>
<tr><td>Cohesion of the rock mass</td><td>>400kPa</td><td>300-400kPa</td><td>200-300kPa</td><td>100-200kPa</td><td><100kPa</td></tr>
<tr><td>Friction angle of the rock mass</td><td>>45°</td><td>35°-45°</td><td>25°-35°</td><td>15°-25°</td><td><15°</td></tr>
</table>

Q-system (Quality system) (Barton et al., 1974)

$$Q = \frac{RQD}{Jn} \times \frac{Jr}{Ja} \times \frac{Jw}{SRF}$$ Equation B1 (Q-value, Barton et al., 1974)

where:

RQD =	Rock Quality Designation
J_n =	joint set number
J_r =	joint roughness number
J_a =	joint alteration number
J_w =	joint water number
SRF =	stress reduction factor

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