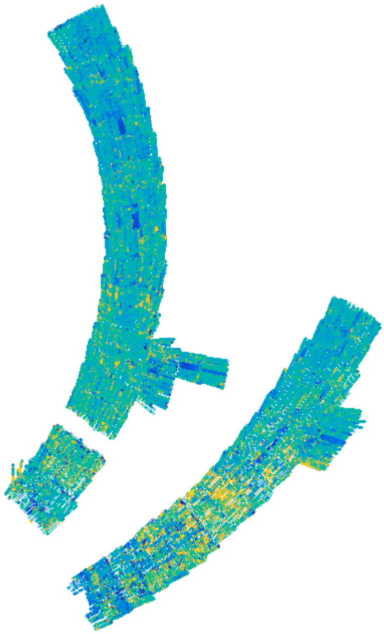
LICENTIATE THESIS

Analysis of Excavation Damage, Rock Mass Characterisation and Rock Support Design using Drilling Monitoring



Jeroen van Eldert

Mining and Rock Engineering

LULEÅ UNIVERSITY OF TECHNOLOGY

Licentiate Thesis

Analysis of Excavation Damage, Rock Mass Characterisation and Rock Support Design using Drilling Monitoring

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Cover image: MWD data displaying penetration rate in blast holes at the Stockholm bypass

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PREFACE

In front of you lies a thesis containing three and half years of research. For me, this journey started over six years ago while I was working for Boliden Mines in Sweden. During my work as a development engineer in Mining Technology, I became interested in the opportunities of emerging technologies, especially Measurement While Drilling (MWD) technology. The work for this thesis has given me much more understanding of this subject, and I see even greater potential in MWD. The main potential of MWD is during the excavation cycle, especially short-term adjustments during tunnel excavation. The study's purpose is to predict excavation damage and the ground support requirements by using MWD data to characterise the rock mass. The thesis 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. In addition, it underlines the opportunities and excavation applications of Measurement While Drilling technology in tunnelling.

The work in this thesis was funded by the Swedish Rock Engineering Research Foundation (BeFo) and Swedish Blasting Research Centre (Swebrec), representing the rock excavation community in Sweden. The data for this research project were provided by Subterra Sweden, Swedish Transport Administration (Trafikverket), Veidekke Sverige, WSP Sweden, the Swedish Nuclear Fuel and Waste Management Co (SKB) and ÅF Consultancy. Software applications for processing the Measurement While Drilling (MWD) data were provided by Epiroc (former Atlas Copco) (Underground Manager MWD) and Sandvik (iSure). I sincerely acknowledge these organisations for their contributions to this research project.

First and foremost I would like to express my gratitude to my supervisors, Håkan Schunnesson, Daniel Johansson and David Saiang (who were involved in reviewing the thesis). Furthermore, I would like to express my appreciation to Ulf Nyberg, Henrik Ittner and Gurmeet Shekhar for the fruitful discussions and an outside review of my work and thesis. In addition, my special thanks goes to Aron Bodén and Mathias Widenbrandt (both former Veidekke), Magnus Gunnarsson (Subterra) and Lars Hjalmarsson (SKB) for on-site support during the field tests. I would also like to express my appreciation to Lars Martinsson, Urban Åkeson, Hans-Åke Mattsson, Mats Olsson, Robert Sturk, Rolf Christiansson, Per Tengborg and Johan Jonsson, for their involvement and guidance in this project. Last but not least, I would like to thank my family and friends for supporting me with love and encouragement during this project, especially during the times of despair.

Jeroen van Eldert, November 2018, Luleå, Sweden

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 thesis 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 thesis applies Ground Penetrating Radar, core logging (for fractures) and P-wave velocity measurements to determine the extent of the blast damage.

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

APPENDED PAPERS

Paper A

Van Eldert, J., 2017. Measuring of Over-Break and the Excavation Damage Zone in Conventional Tunneling. World Tunnelling Congress (WTC2017), 9–15 June 2017, Bergen, Norway

Paper B

Van Eldert, J., Schunnesson, H., Johansson, D., 2017. The History and Future of Rock Mass Characterisation by Drilling in Drifting - From sledgehammer to PC-tablet. 26th International Symposium on Mine Planning and Equipment Selection (MPES2017), 29-31 August 2017, Luleå, Sweden

Paper C

Van Eldert, J., Schunnesson, H., Johansson, D., Saiang, D. 2018. Measurement While Drilling to Predict Rock Mass Quality and Support. Submitted to an International Journal

Paper D

Van Eldert, J., Ittner, H., Schunnesson, H., Johannsson, D., 2016. Evaluation of Alternative Techniques for Excavation Damage Characterization. World Tunnelling Congress (WTC2016), 22–28 April 2016, San Francisco, United States of America

Paper E

Van Eldert, J., Schunnesson, H., Johansson, D., Saiang, D., 2018. Measurement While Drilling (MWD) technology for blasting damage calculation. Fragblast12, 9–15 June 2018, Luleå, Sweden

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
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)
SH	Shotcrete
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 Stockholm city 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 Road Authority 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 might affect 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.

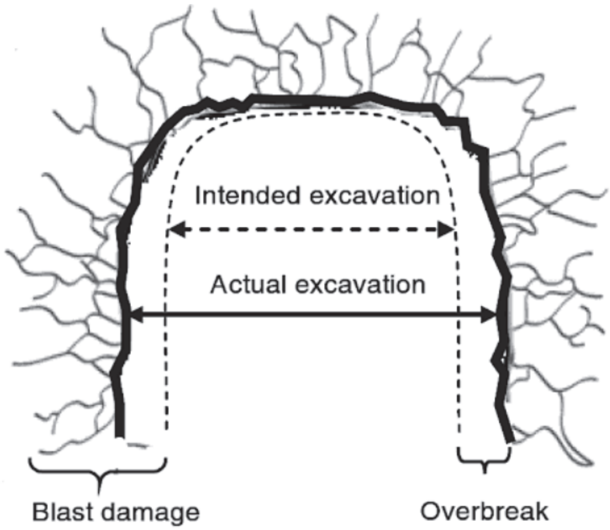


Figure 1.1 Blasting induced damage and over-break outside the intended tunnel perimeter (Warneke et al., 2007).

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 thesis 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 thesis, 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?

The relations between the formulated research questions and the appended papers are presented in Table 1.2. Paper A 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 Paper D and Paper E. Paper B gives a historical over-view of MWD technology and its applications today. Paper C investigates the differences and similarities between grout and blast hole MWD. Paper C also presents a case study of the application of MWD technology for validation and re-characterisation of the rock mass in a tunnelling project. Paper D investigates the usage of MWD data to predict blast damage at one site. Paper E extends the findings in Paper D and correlates MWD parameters with the measured EDZ at two additional sites. In addition, Paper D and Paper E address the influence of the rock mass on MWD parameters and the excavation damage. In all of the appended papers, the first author is the main contributor. The tasks of the main contributor included data collection, data compilation and analyses and write-up. The co-authors provided technical and scientific guidance and contributed to the write-up.

Table 1.2 Relationship between appended papers and research questions.

	Paper A	Paper B	Paper C	Paper D	Paper E
RQ1:	X		(X)	X	X
RQ2:		X	X	X	X
RQ3:		(X)	X		
RQ4:	(X)	(X)		X	X

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.

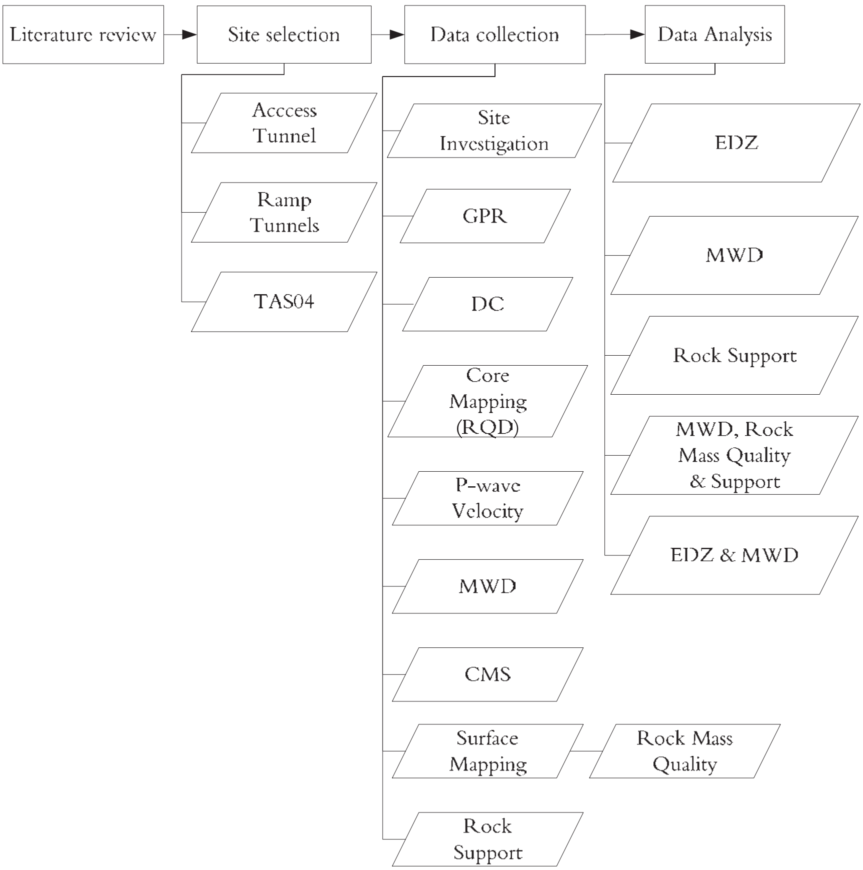


Figure 2.1 Methodology to study excavation damage in tunnelling.

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.

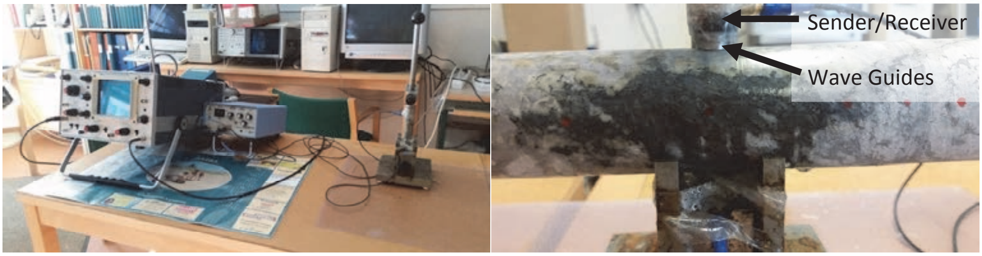


Figure 2.3 Setup for diametrical P-wave measurements on the collected drill cores.

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.

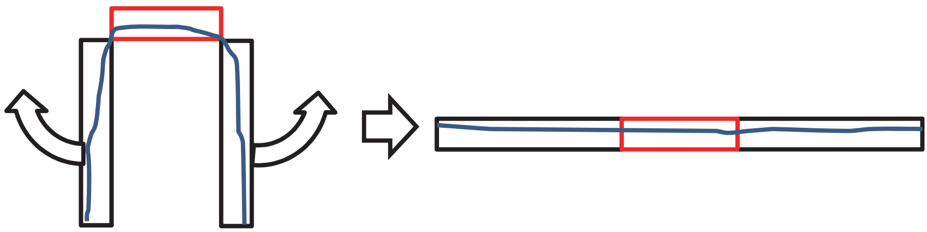
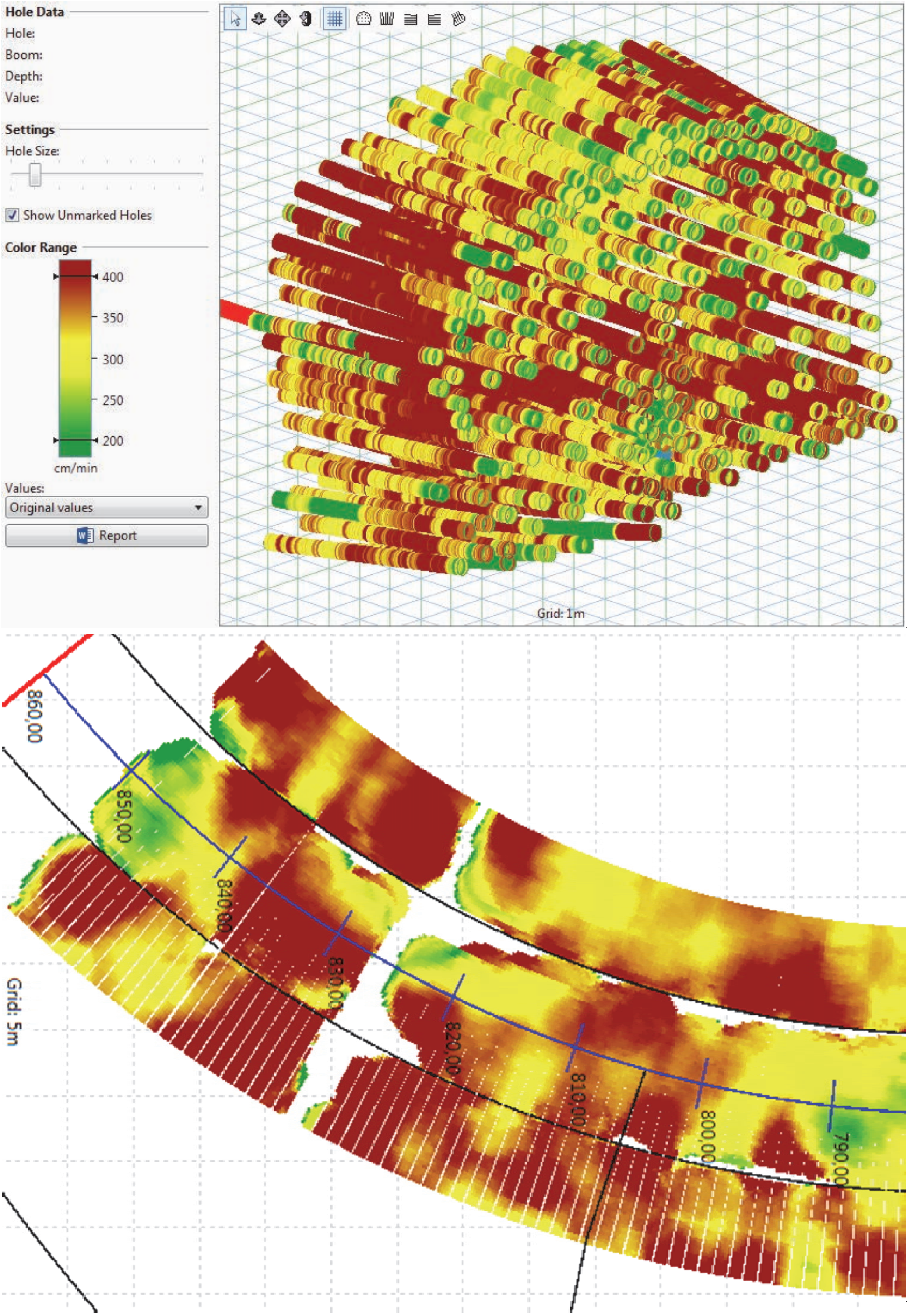


Figure 2.4 Folding out of tunnel contour for visualisation of tunnel mapping and 2D visualisation of tunnel walls and roof



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.

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 3.4) and its current applications (Section 3.5).

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.

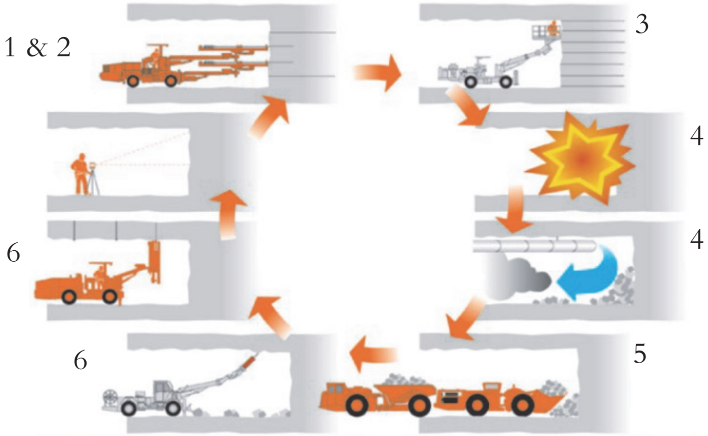


Figure 3.1 Excavation cycle in tunnel excavation (Modified after Tamrock, 1999).

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.

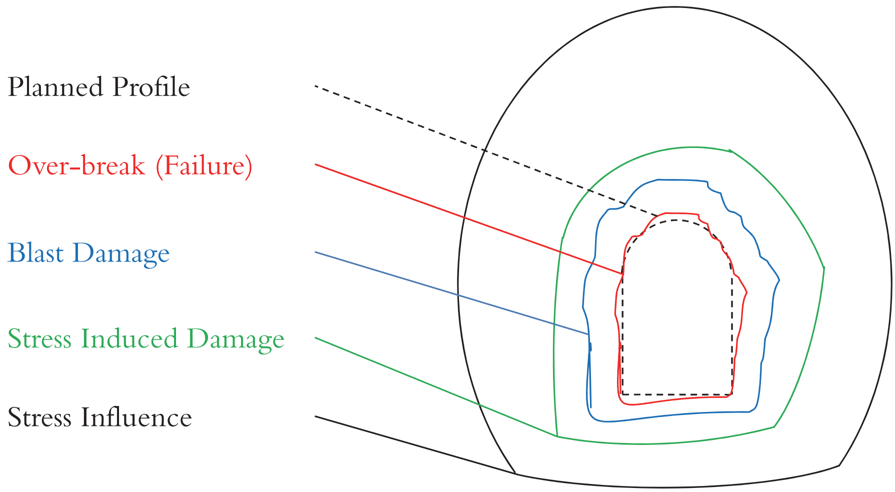


Figure 3.2 Excavation damage divided into subzones: over-break, blast damage, stress induced damage and stress influenced areas.

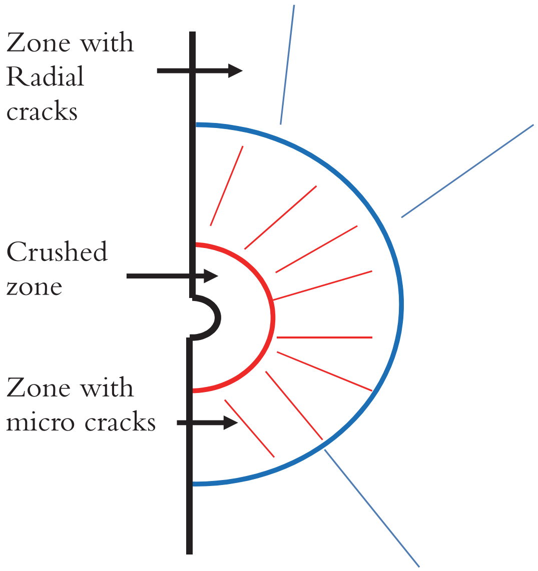


Figure 3.3 Development of Excavation Damage around the blast hole and characteristic zones.

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).

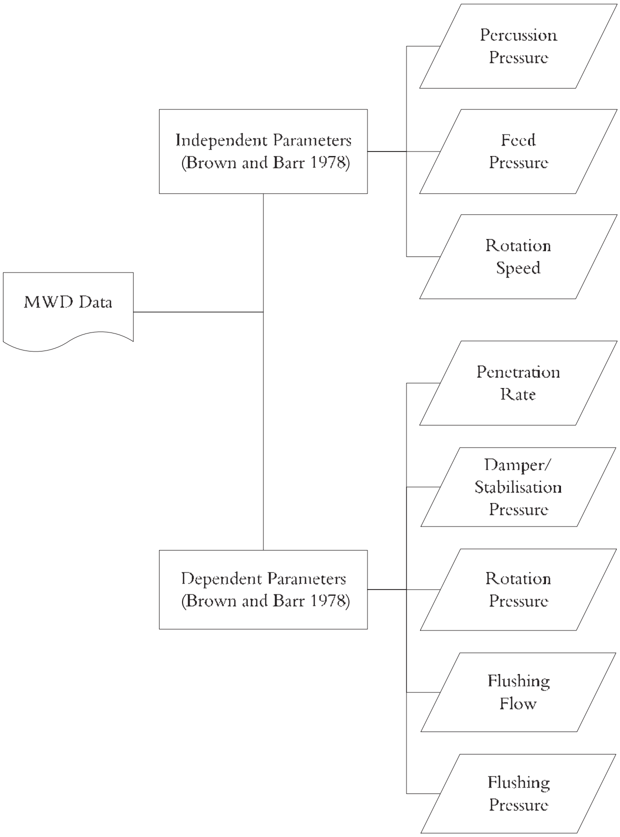


Figure 3.5 Independent and dependent MWD parameters available in Atlas Copco (now Epiroc) MWD data after Brown and Barr (1978).

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.

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). 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 good correlation between the MWD 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).

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.

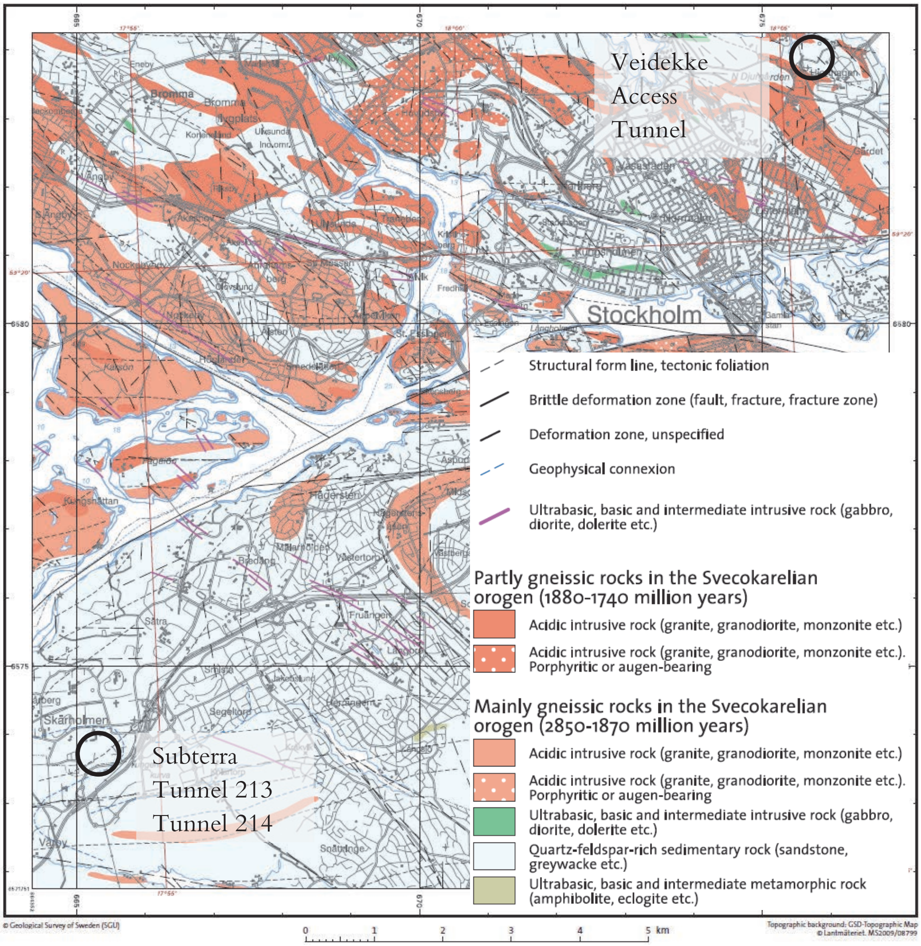


Figure 4.1 Stockholm's geological map of the Stockholm area with the two Stockholm investigation sites in this study (after SGU, 2017).

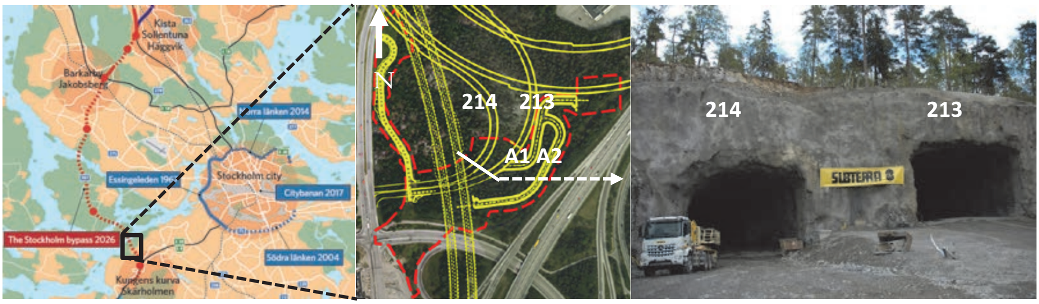


Figure 4.2 Stockholm bypass and Tunnels 213 and 214 location, layout and tunnel entrances (Illustrations courtesy of Trafikverket).

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.	Section	Rock Class	Q-value	Rock Cover	Remarks	Bolt		Shotcrete Thickness	
						Spacing	Length	Wall	Roof
213	200 to 210	III	1.5	3.5m	SRF=5, Jn=6x2	1.7 m	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	1.7 m	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)	1.5 m	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	1.5 m	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: SRF = stress reduct. factor, Jn = joint set numb., Jw = joint water param., S = Select. bolt									

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*).

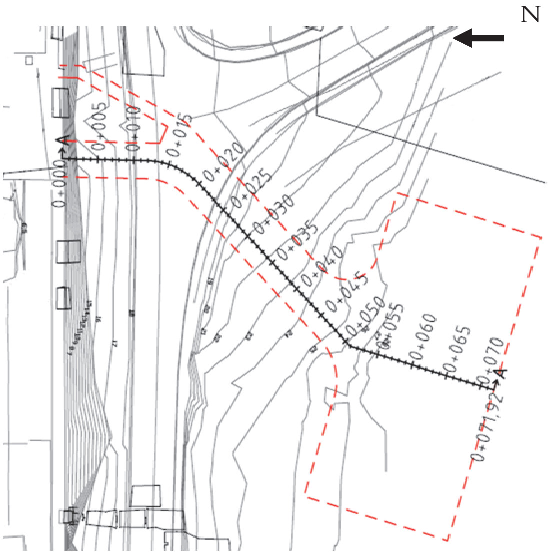


Figure 4.3 Layout of Veidekke access tunnel and gallery excavated at Norra Djurgården, Stockholm (Karlsson, 2014).

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*).

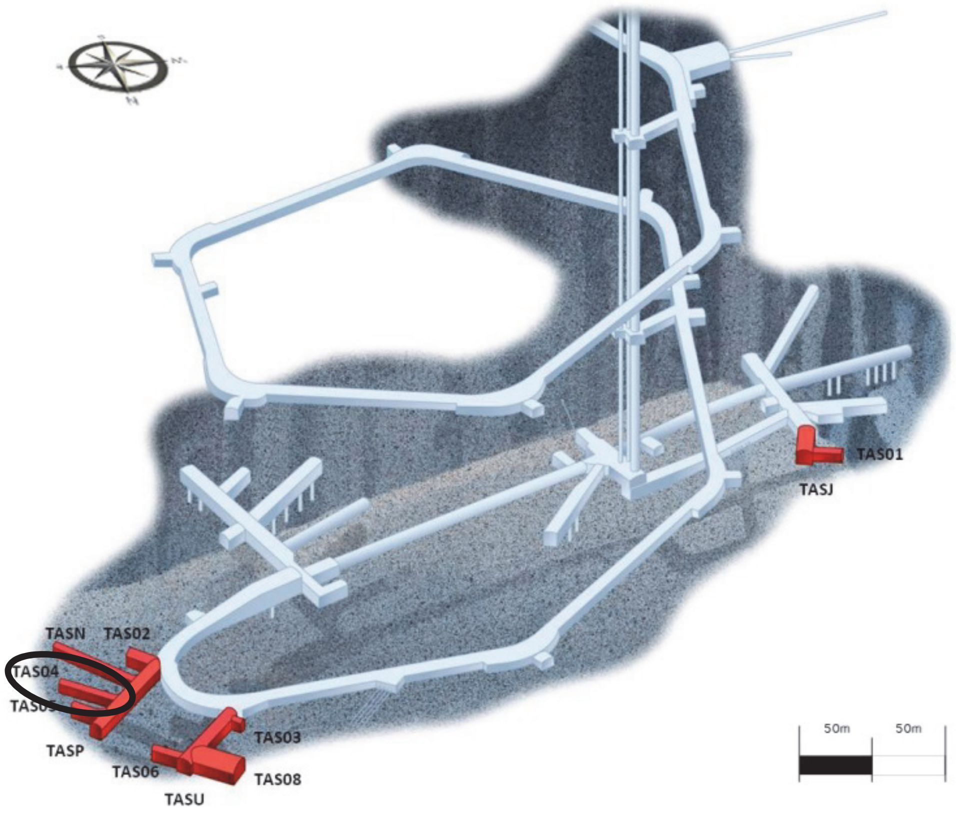


Figure 4.4 Äspö Hard Rock Laboratory layout. The data collected in this study were from the TAS04 tunnel, denoted by the black circle (modified after Johansson et al., 2015).

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.

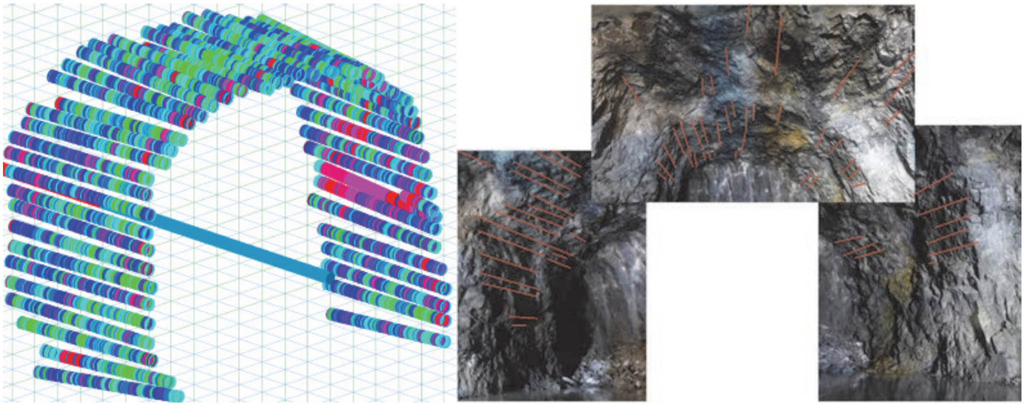


Figure 5.1 Blast holes and Half Casts in a section in Tunnel 213 (photographs modified after ÅF 2016).

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 thesis, 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 (Profile & length)	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.

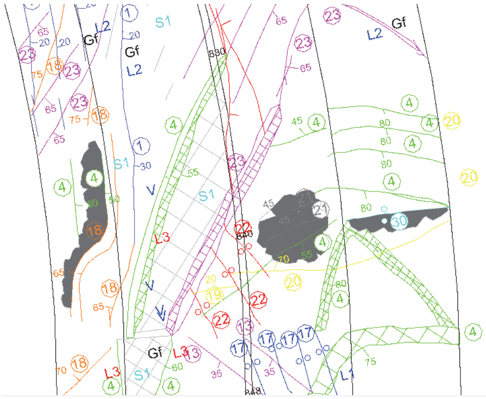


Figure 5.3 Geological mapping of Tunnel 214 between sections 848 and 827 at Stockholm bypass (modified after ÅF 2016).

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 unidentified fractures in Figure 5.4 are most likely blast induced fractures.

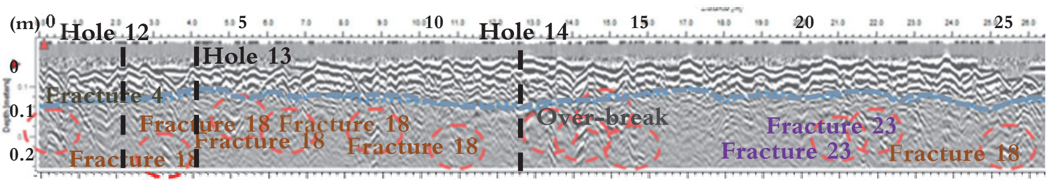


Figure 5.4 Ground Penetrating Radar showing recorded fractures (red circles) for left wall section 848, the tunnel portal, to section 827, 21m into the tunnel, in Tunnel 214 at the Stockholm bypass; the majority were related to fractured mapped, and others are likely to be caused by blasting. The blue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

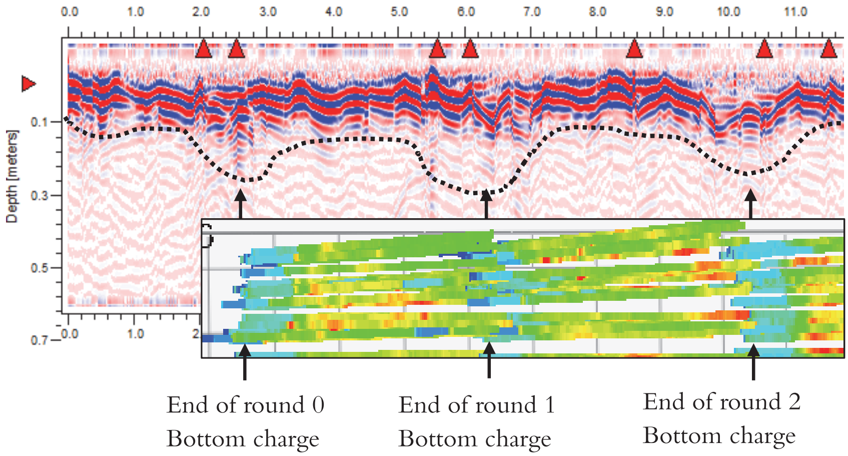


Figure 5.5 GPR and drilling data for the left wall of TAS04 tunnel showing an increase of the extent of the GPR blast damage at the drill hole bottoms with a higher charge concentration.

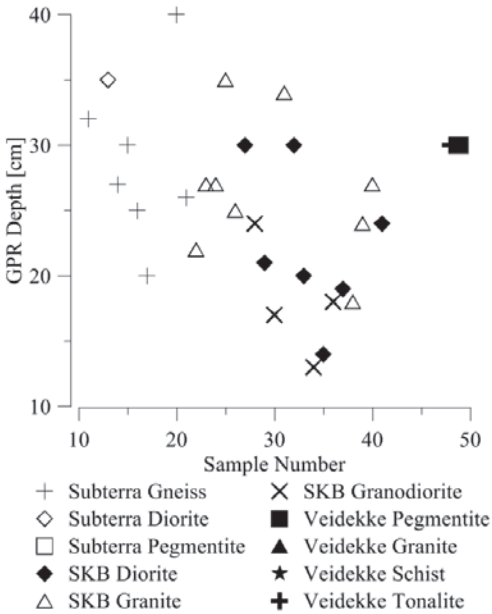


Figure 5.6 GPR reflection at depth due to blasting fractures of different rock types at the three investigated sites at the locations of the drill cores. The figure shows the influence of grain size on GPR recordings: pegmatite and gneiss have greater GPR depth than diorite and granodiorite.

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, as denoted by the black lines in the drill cores in Figure 5.9. 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.

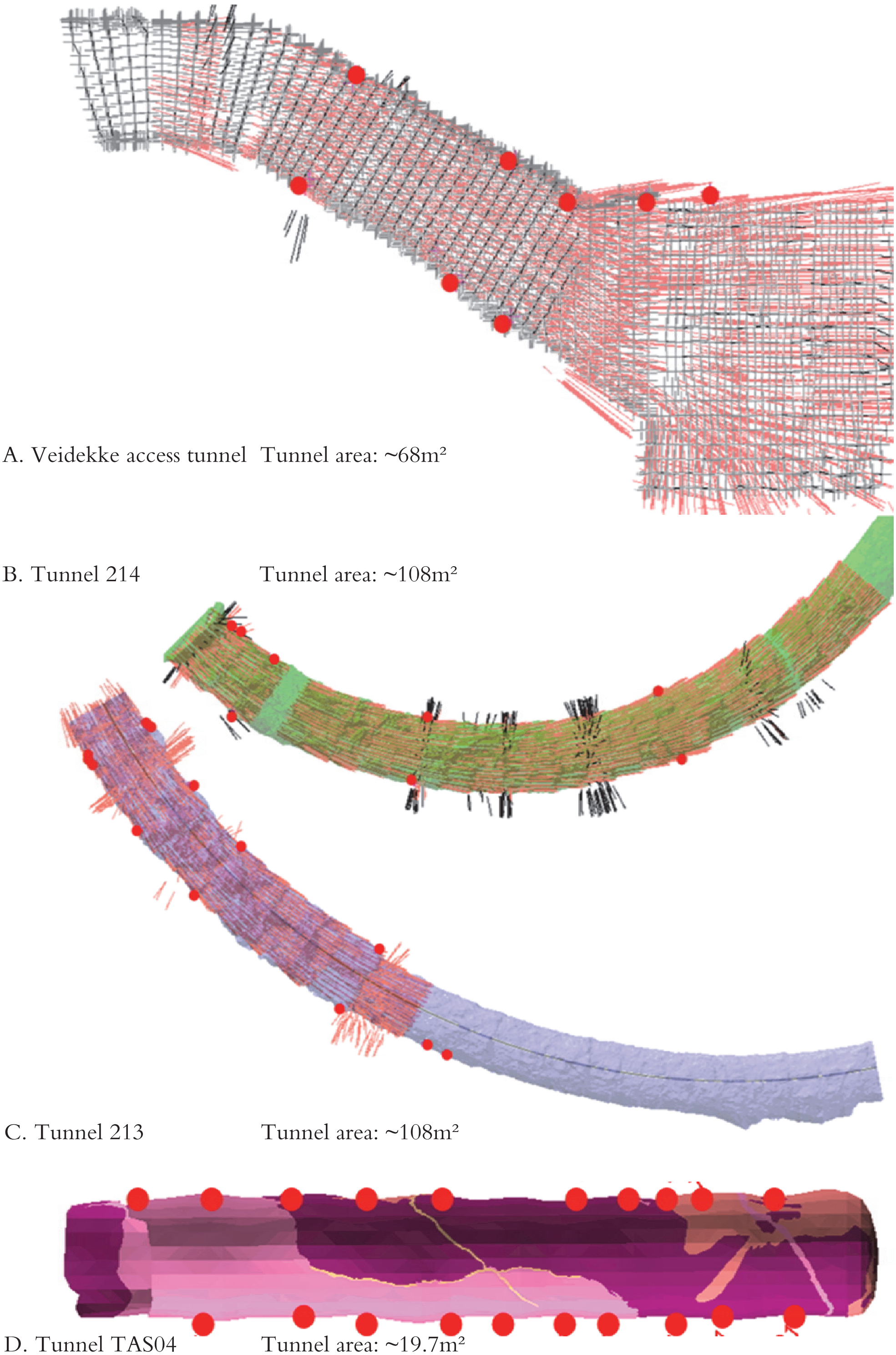
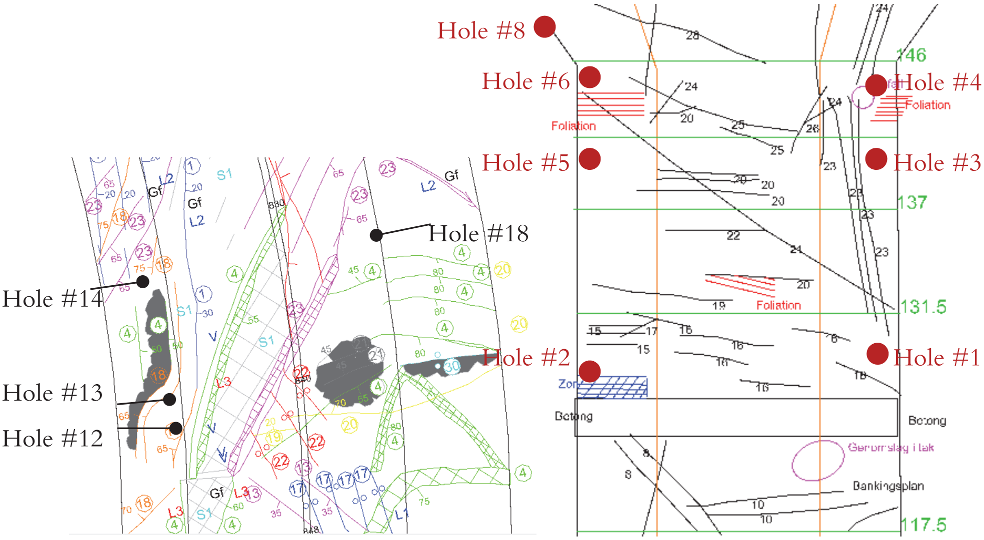


Figure 5.7 Location of diamond core holes in the four tunnels: eight drill cores in the Veidekke access tunnel (A), eight in Tunnel 214 (B), 13 in Tunnel 213 (C) and 20 in TAS04 tunnel (D).



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.

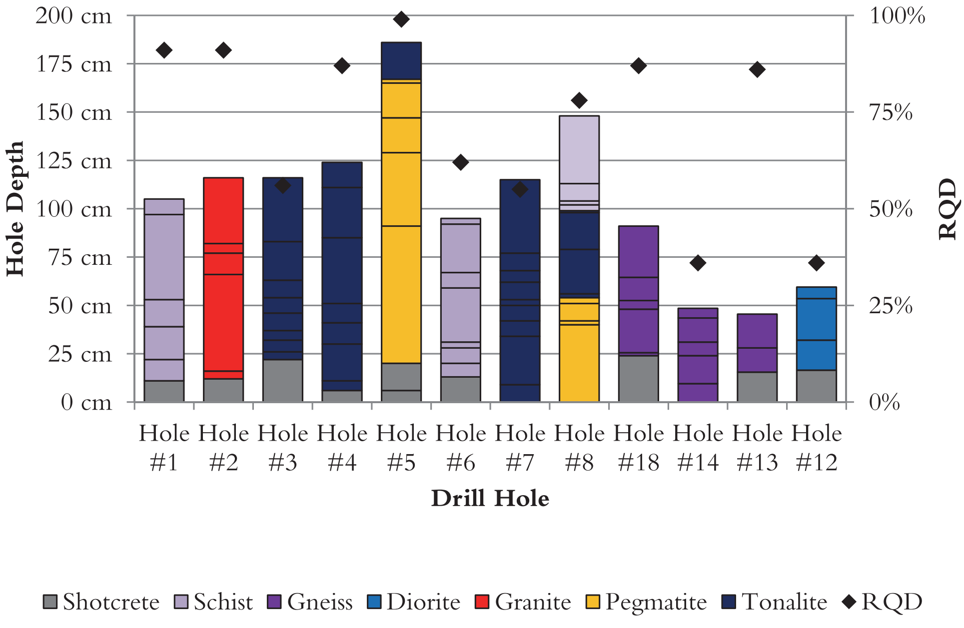


Figure 5.9 Drill core mapping from tunnel 214 and the Veidekke access tunnel for the blast damage in the investigation. The RQD ranges from 30% to 100%.

Figure 5.10 Cores from Hole #1 to Hole #4 from the Veidekke access tunnel (SH = Shotcrete).

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 (SH=Shotcrete).

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).

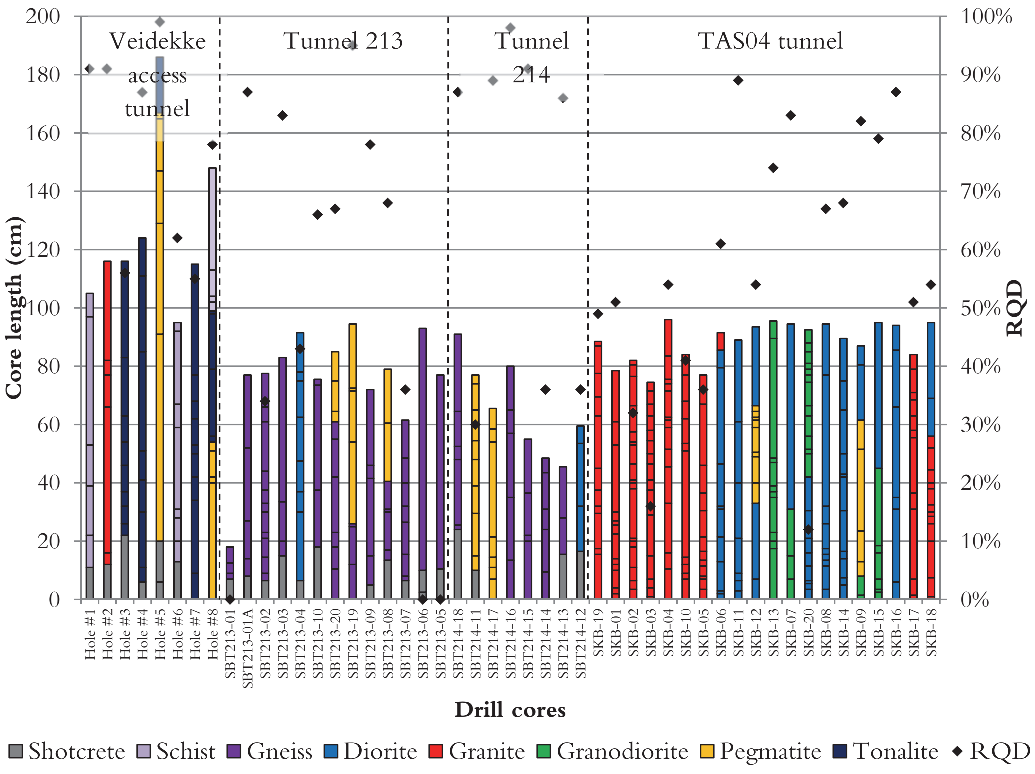


Figure 5.12 Results from the diamond core showing the variation of rock types, the EDZ depth determined by GPR, P-wave velocity and RQD.

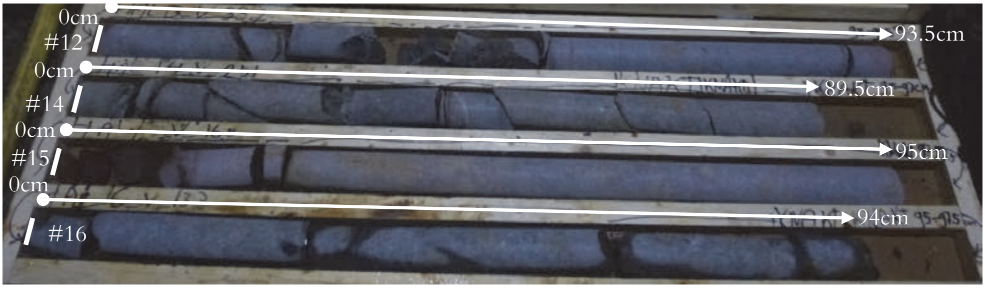


Figure 5.13 Cores #12, #14, #15 and #16 from TAS04 tunnel.

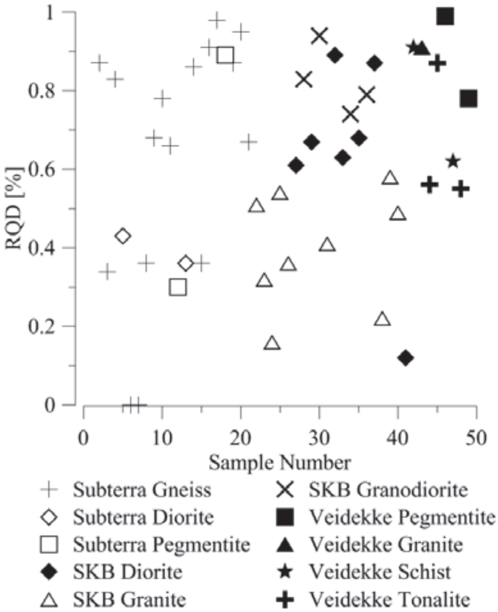


Figure 5.14 RQD per rock type for each sample collected in the four tunnels at the three investigated sites. The figure shows a lower RQD in fine-grained rock masses (granite and diorite) and higher RQD in large-grained rock masses (pegmatite and gneiss).

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).

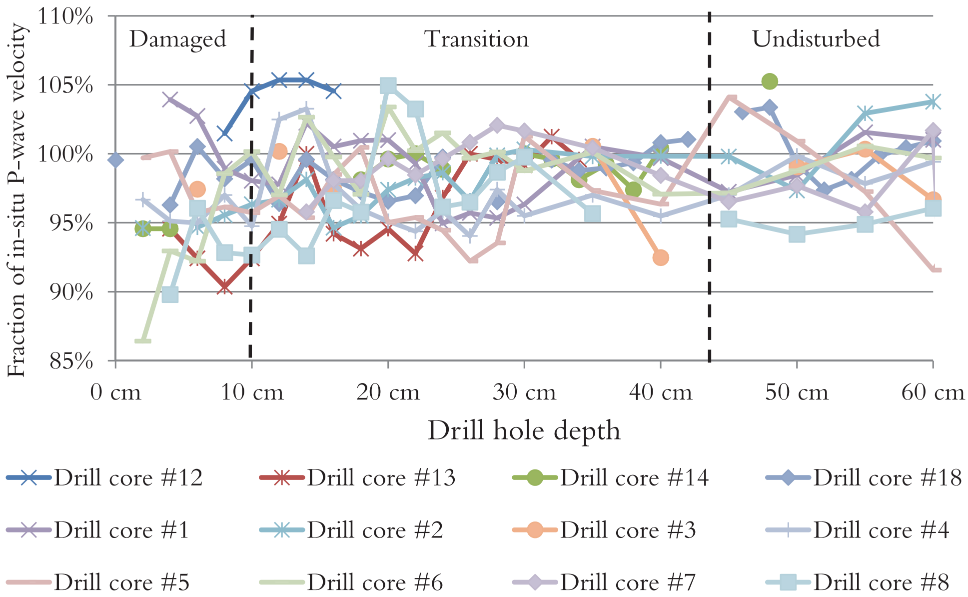


Figure 5.15 P-wave velocities displayed in percentages of the intact rock mass for the investigated drill cores. The missing samples indicate fracturing of the drill core.

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.

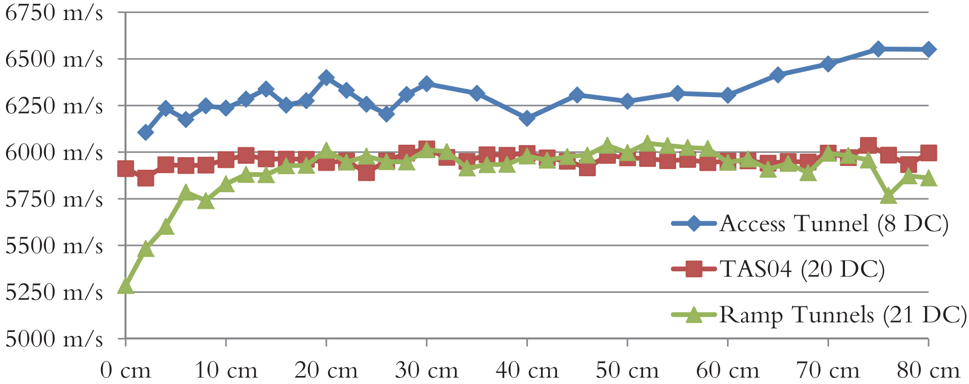


Figure 5.16 Average P-wave velocities at the drill cores at the three investigated sites. The lower velocities indicate damaged rock mass. The figure shows the effects of the initiation system, whereby the Veidekke access tunnel and TAS04 tunnel used electronic detonators in the perimeter holes and the two Stockholm bypass ramp tunnels used pyrotechnical detonators in the contour holes.

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).

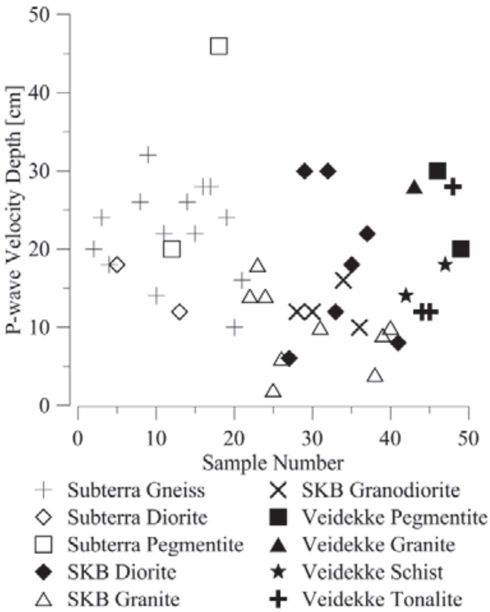


Figure 5.17 Depth of blast damage based on P-wave velocity measurements for different rock types at the sites investigated. Lesser extent of the EDZ is observed in the fine-grained granite.

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), the bolt length was increased (from 3m to 6m) in the least favourable parts of the tunnel (especially in Tunnel 213), 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).

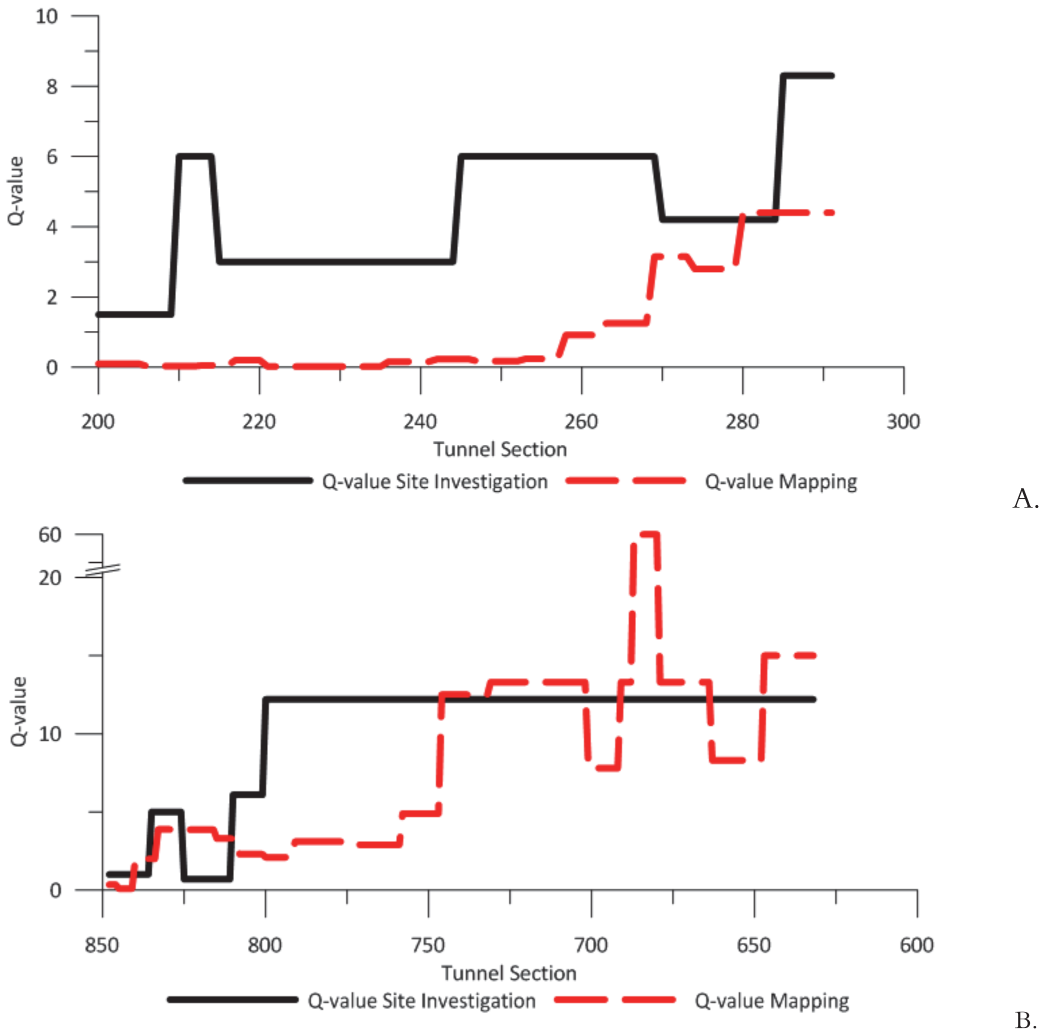


Figure 5.18 Prognosis and realisation of rock mass quality in (A) the first 100 m of Tunnel 213 and (B) the first 220 m of Tunnel 214.

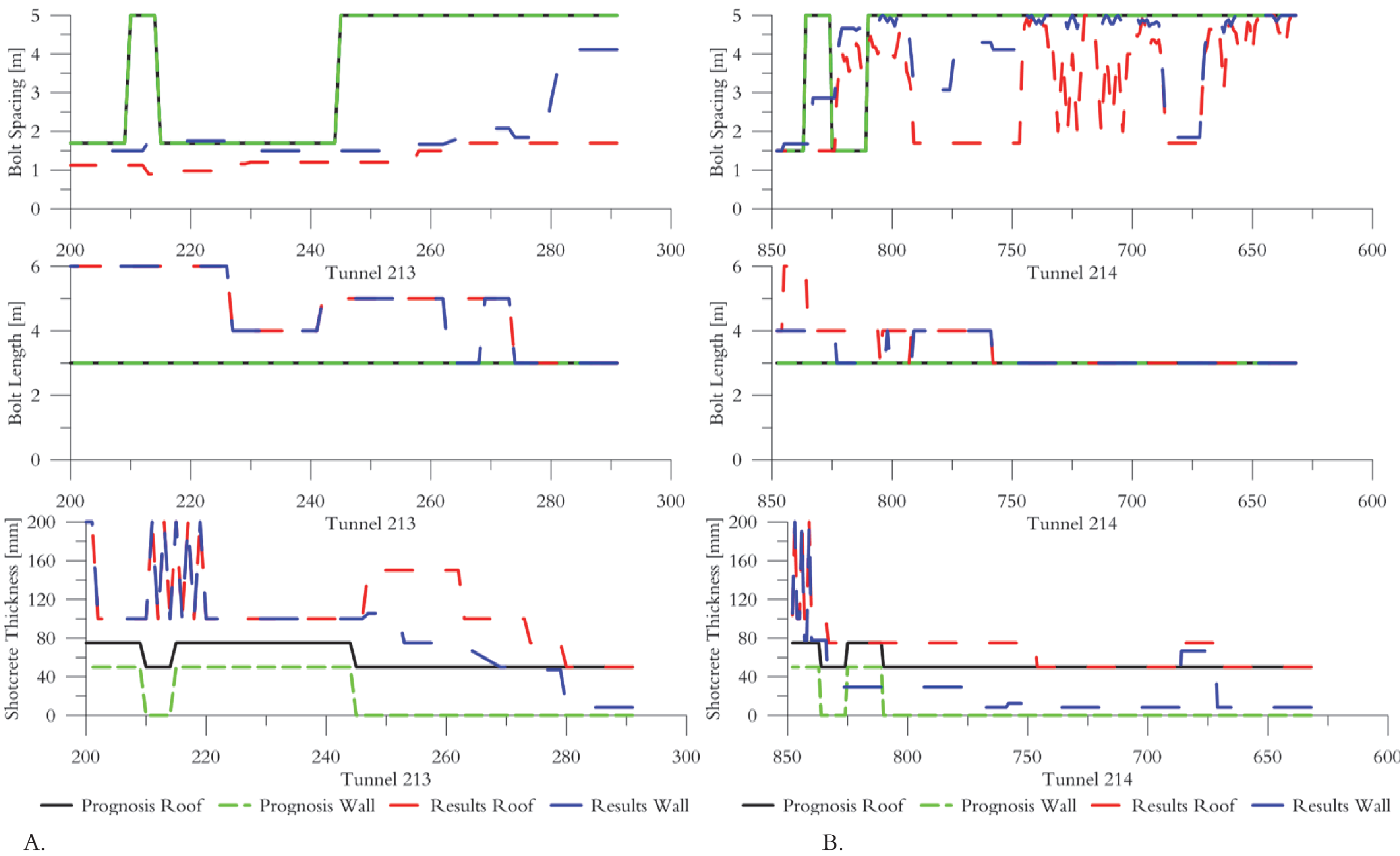


Figure 5.19 Prognosis and installed rock support in Tunnel 213 (A) and Tunnel 214 (B) (Selective bolting "spacing"=5 m).

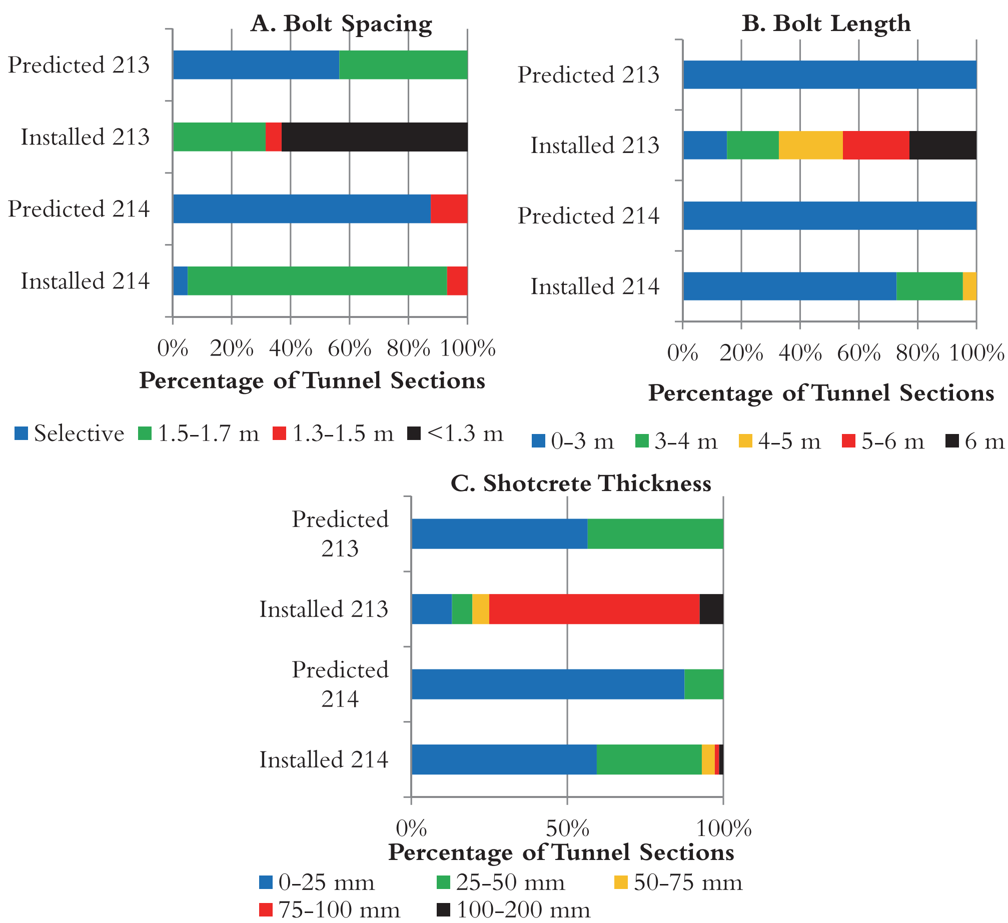


Figure 5.20 Predicted and installed bolt spacing (A), bolt length (B) and shotcrete thickness (C) in Tunnel 213 and Tunnel 214. The length is defined as the percentage of tunnel section, i.e., 90m for Tunnel 213 and 220m for Tunnel 214.

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 thesis, 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).

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

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

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.

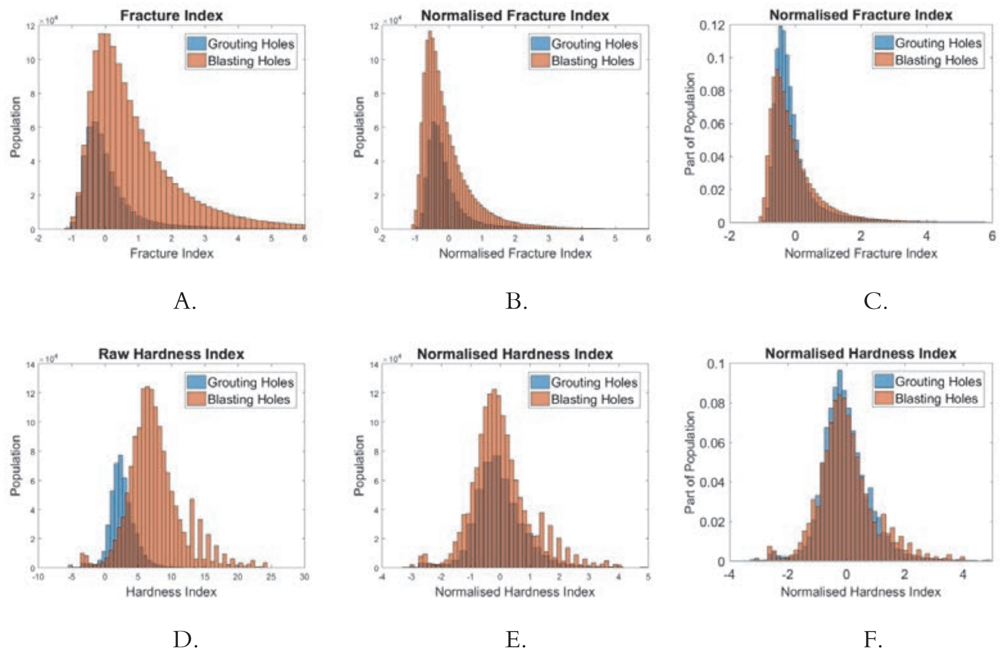


Figure 5.21 Normalisation procedures for Fracture Index and Hardness Index for MWD data for Tunnel 213 and Tunnel 214 at Stockholm bypass. The raw Fracture Index (A) and the normalised data (B) show a similar distribution for the grouting and blasting hole Fracture Index (C). The raw Hardness Index (D)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

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).

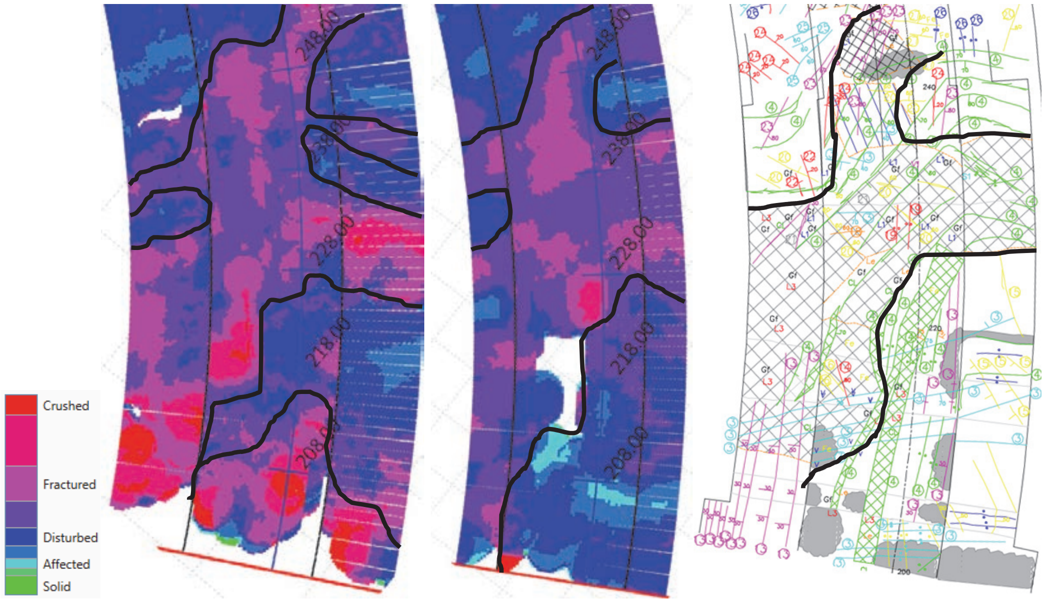


Figure 5.22 Fracture Index and geotechnical mapping for section 200 to section 250 (50m) in Tunnel 213 showing grout holes, blast holes and mapping. The black lines show the shape and orientation of the fracture zones.

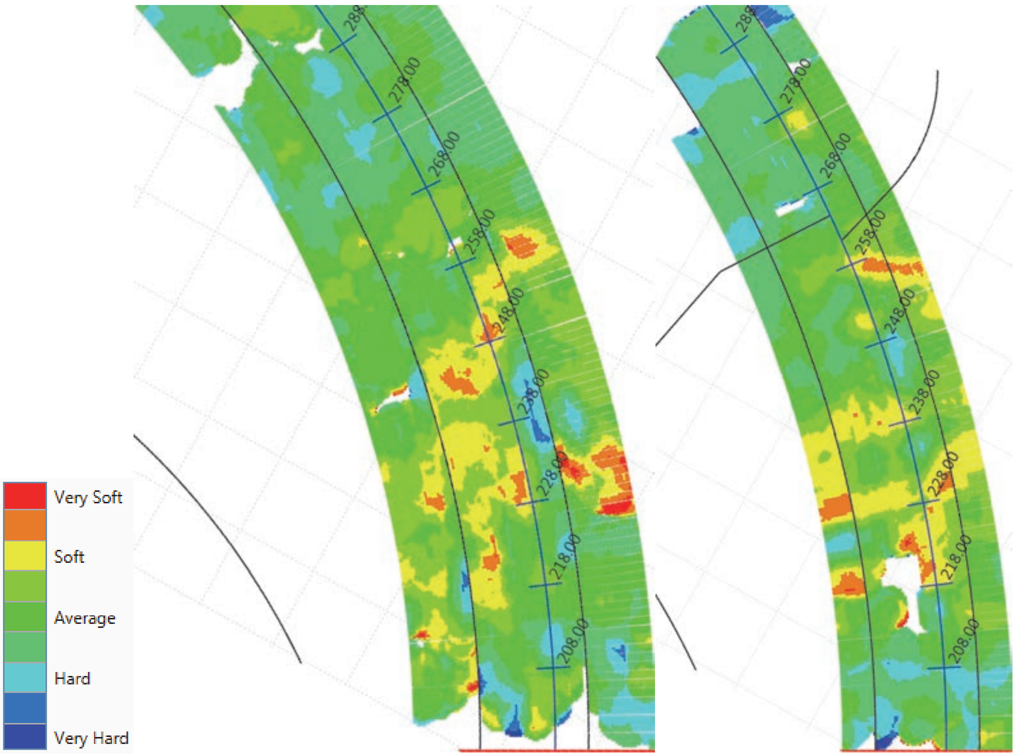


Figure 5.23 Hardness Index for section 200 to section 290 (90m) in Tunnel 213 showing grout holes on the left and blast holes on the right. Softer rock is massed in section 218 to section 243 (25m).

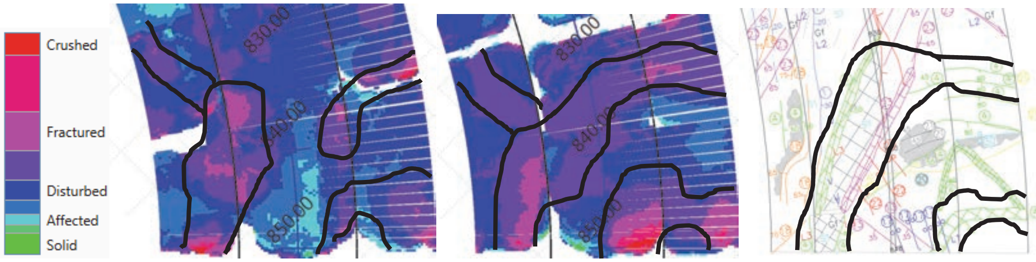


Figure 5.24 Fracture Index for section 848 to section 820 (28m) in Tunnel 214 showing grout holes on the left, blast holes in the middle and mapping on the right. The black lines indicate the shape and orientation of the fracture zones in the MWD data and tunnel mapping.

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.

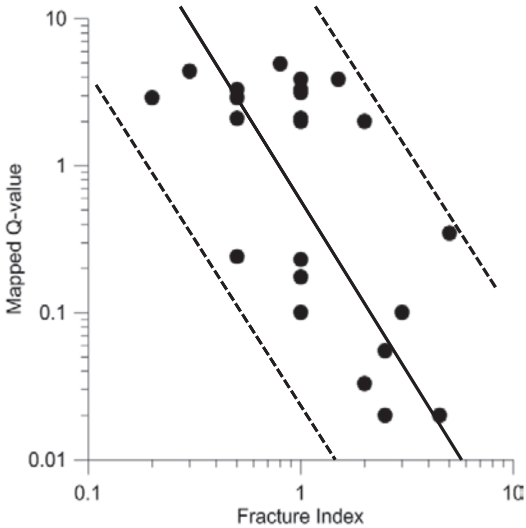


Figure 5.25 Relation of mapped Q-value and MWD Fracture Index on a log-log scale.

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.

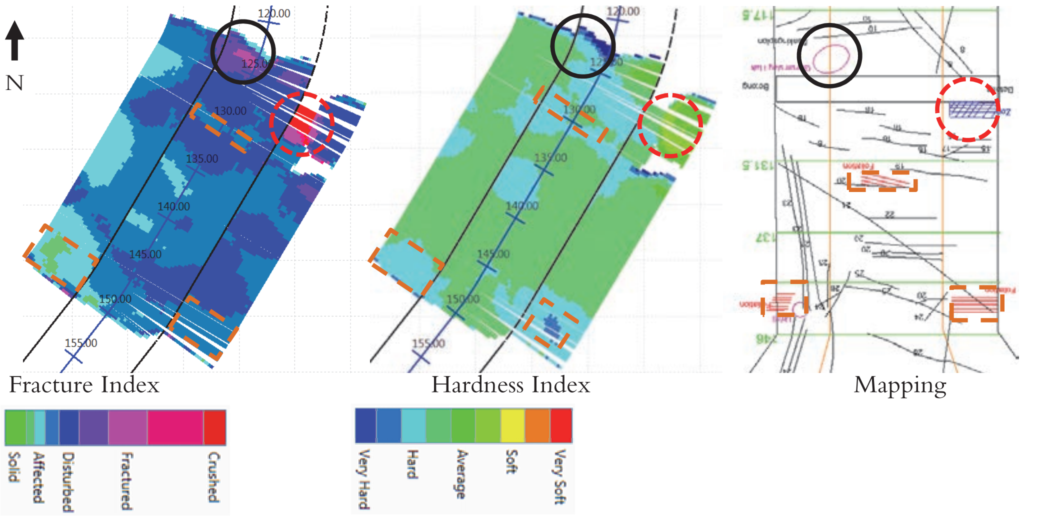


Figure 5.26 Fracture and Hardness Indices based on the grout holes, and mapping of the access tunnel; the encircled areas indicate fractured areas and the rectangles show the effects of the foliation.

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 thesis, 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.

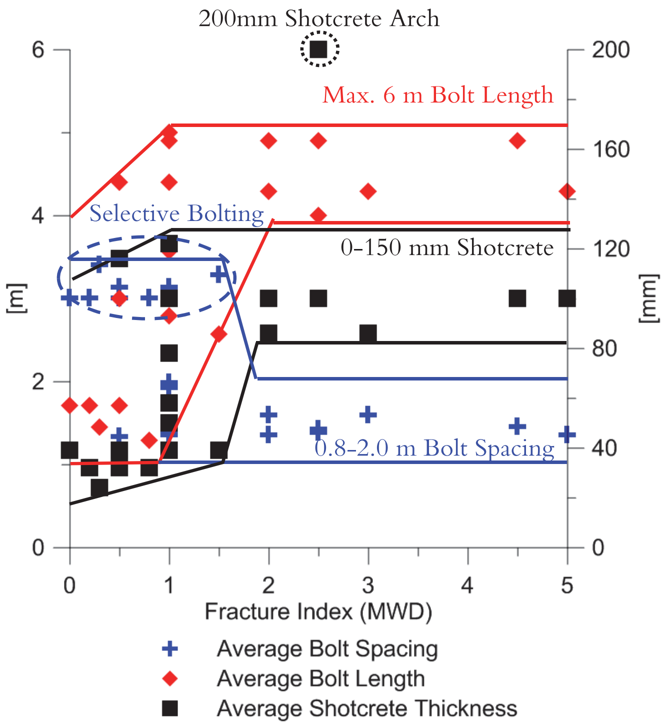


Figure 5.27 Correlations between the Fracture Index and installed rock support in 24 sections along Tunnel 213 and Tunnel 214.

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

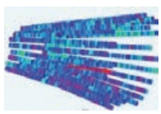
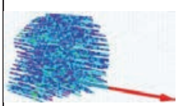
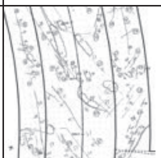
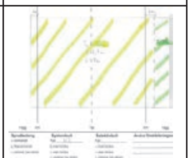
Table 5.5 Rock and support classes at the Stockholm bypass (modified after Arghe, 2013).

Rock Class	Q-value	Rock Quality	Shotcrete	Bolt Length	Bolt Spacing
I	$Q > 10$	Very good	0-50 mm	0-3 m	Selective
II	$4 < Q \leq 10$	Good	0-75 mm	0-4 m	>2.0 m
III	$1 < Q \leq 4$	Acceptable	0-100 mm	3-4 m	1.7-2.0 m
IV	$0.1 < Q \leq 1$	Poor	75-150 mm	4-5 m	1.5-2.0 m
V	$Q \leq 0.1$	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	1-7 Days	0-4 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).

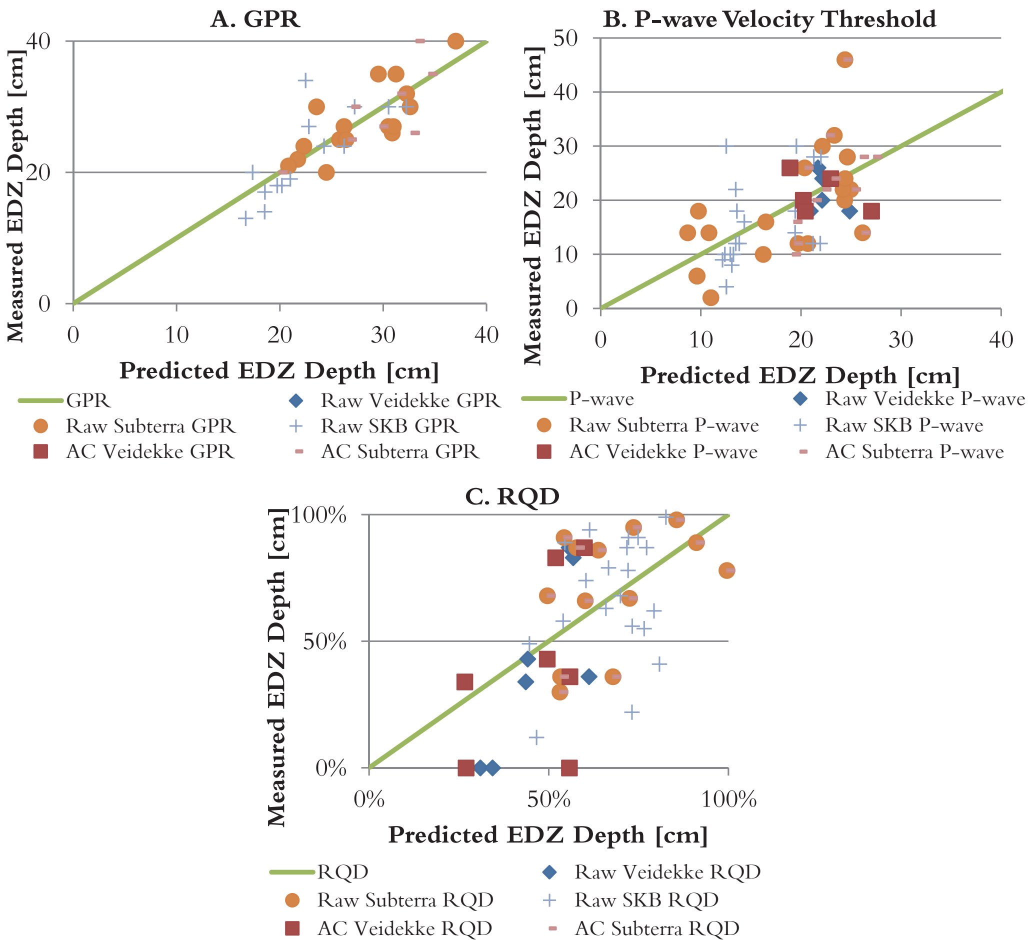


Figure 5.28 Correlation of the measured and predicted (by the factors derived from MLR analysis) extent of the blast damage at the three sites based on the GPR (A), P-wave velocity (B) and RQD (C).

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 thesis, 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 thesis. 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 thesis 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 thesis. 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 thesis 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

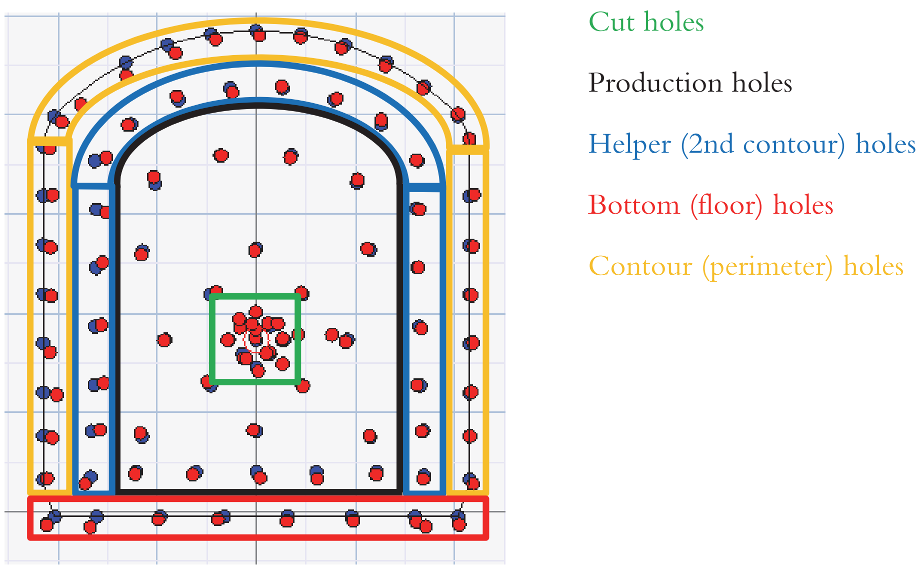

Figure A.1 Schematic layout of a blast hole drilling round for a tunnel blast. The blue dots show the drill plan and the red dots the drilling performance.

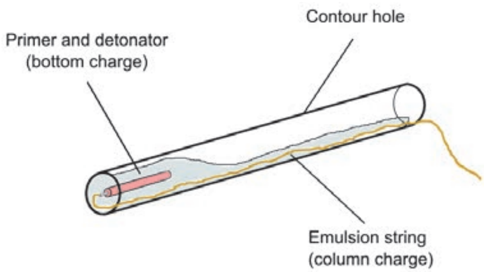

Figure A.2 Blast hole string or column and bottom charge in a contour hole (Ittner et al., 2018).

APPENDIX B: ROCK MASS CLASSIFICATION

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

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

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

B	Rating adjustment for joint orientations					
Strike and dip orientations of joints		Very favourable	Favourable	Fair	Unfavourable	Very unfavourable
Ratings	Tunnels	0	-2	-5	-10	-12
	Foundations	0	-2	-7	-15	-25
	Slopes	0	-5	-25	-50	-60

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

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

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

$$Q = \frac{RQD}{J_n} \times \frac{J_r}{J_a} \times \frac{J_w}{SRF}$$ 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

APPENDED PAPERS

Measuring of Over-Break and the Excavation Damage Zone in Conventional Tunneling

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ABSTRACT: During tunnel excavation, blast induced damage or Excavation Damage Zone (EDZ) interacts with existing geological structure of the rock-mass. This can and cause over-break and stability issues during and after excavation. Today the EDZ is determined by correlation-based methods, although direct measurement is possible. This paper presents investigation methods for EDZ quantification during excavation. The discussed methods are applied in the Stockholm By-Pass project. The literature review and the case study applications are summarized in a comprehensive table with benefits and limitations of the different investigation methods. The methods discussed can be used to achieve a better quality of the tunnel contour, especially the direct investigation techniques. The EDZ can be reduced by adjusting the blasting plan, specific charge as well as improving the quality of the drill and blasting procedures.

1 INTRODUCTION

State-of-the-Art excavations in Sweden are nowadays performed by smooth wall blasting (Langefors and Kihlström, 1978). These excavations use mostly emulsion and string emulsion in respectively cut/production holes, and contour/helper holes. The excavation cycle with drill and blast consist commonly of the following steps:

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 pumpable emulsions
4. Blasting and ventilation, the use of pyrotechnical (Nonel) and/or electronic detonators
5. Mucking and cleaning, large front end loaders in combination with dumpers or road lorries
6. Scaling and rock support, with fully mechanized equipment for scaling, shotcrete spraying and bolt hole drilling (Jumbo)

The conventional excavation causes over-break and an Excavation Damage Zone (EDZ). This paper focusses on the methods of measurement of over-break and excavation damage as well as geological structures influencing the EDZ.

1.1 Excavation Damage

During the excavation process the rock mass outside the tunnel contour (theoretical contour) can be damaged by the blasting. The excavation damage is known to have a major effect on the rock mass and can cause over-break and tunnel instability (Saiang and Nordlund, 2009; Saiang, 2010). The EDZ is an irreversible change of rock mass properties (fractures) (Siren et al., 2015). The damage zone is influenced by the excavation procedures, the properties of the rock mass, the tunnel design (tunnel area and shape) and the stresses in the rock mass. The research on the EDZ started in the 1980s in the Stripa mine in Sweden (Andersson et al., 1989; Emsley et al., 1997) and is still under investigated (Siren et al., 2015; Ittner et al., 2016). The main focus of the EDZ in previous investigations has been the transmissibility of water (Christiansson et al., 2005; Olsson et al., 2009; Ittner et al., 2014; Silvast and Wiljanen, 2008; Siren et al., 2015).

A excavate tunnel can be divided in three different parts: 1) the failure zone; the rock has fallen and is excavated (Over-Break), 2) the EDZ and 3) the Excavation disturbed Zone (EdZ), where a change of stress regime can be observed, but no irreversible change occurs. In other words the original stress regime could be, theoretical, restored. The failure zone is the purpose of the excavation but can be enlarged when the EDZ interacts with the local geological structures causing over-break i.e. the failure of rock masses outside the planned perimeter.

1.2 Over-Break

Over-break is seen as an indication of the blasting damage in the form of the failure zone. Over-break from the designed tunnel profile is caused by:

- Drilling geometry; the look-out angle in other to maintain the required tunnel area (practical over-break)
- Drilling inaccuracy; collar placement (cm scale) and drill hole deviation (cm/m scale), (Pre-blast over-break)
- Blasting damage and geological structures; natural rock falls and scaling (Post-blasting over-break)

1.3 Excavation Damage Zone

The EDZ is classified in three separate zones (Van Eldert et al., 2016):

1) Inner Damage Zone (Crush Zone)
2) Transition Zone (Micro and macro fractures)
3) Progressing Zone (Macro fractures and extension of existing fractures)

The micro and macro fractures caused by blasting develop a network within the rock. The micro fractures are caused by the shockwave energy of the blast (Jern, 2001). A clear sign of these fractures is the white wash in the half pipes directly after blasting. These micro fractures can be seen at the surface by visual observation, P-wave measurements and seismic dispersion. The micro fractures connect to each other in transition zone to form macro fractures. These macro fractures are formed radially from the drill hole and can be parallel to the tunnel wall (Olsson et al., 2009). These fractures interconnect with natural fractures and can end on these or jump the fracture (Ittner et al., 2016). The fractures are path ways for the gasses expanding and these can reopen existing fracture or even propagate and develop new fractures (Ouchterlony et al., 2009). In some cases the blasting induced fractures intersected with the pre-excavation joints and fractures in the rock mass, causing massive over-break. The over-break can lead to stability issues and increased cost for the material haulage, machine hours as well as the amount of rock support placed (Van Eldert, 2014).

1.4 Excavation Damage control in Sweden

In Swedish infrastructure tunnels, requirements are set for the theoretical blasting damage zone in the General Material and Work Description (AMA13) (svensk byggtjänst, 2014), as shown in Table 1. It is based on imperial tests, independent of the rock mass and excavation parameters. In practice, this theoretical damage zone is based on the Peak Particle Velocity (PPV). The PPV is determined by the amount of explosive detonated at once and the properties of the rock masses between the detonation point and measurement point. The PPV is to some extent related to the fracture development length into the rock mass, but differs with explosive, blasting and rock mass properties (Olsson et al., 2009). Unfortunately these parameters are not taken into account with this method or the standardized blasting tables.

In practice the charge concentration – fracture length relation is still used in practice as shown in Figure 1.

Table 1. Excavation tolerance and theoretical damage zone, CBC/2 AMA13 (After svensk byggtjänst, 2014).

Rock Excavation Class	Theoretical damage zone (m) Wall/cut	Floor
1	0.2	0.5
2	0.3	0.7
3	0.5	1.1
4	1.1	1.7
5	Excavated rock lays outside theoretical contour	

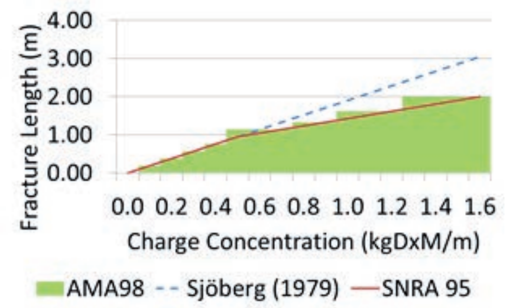

Figure 1. Blast damage table which is used in practice.

2 METHODS FOR EDZ INVESTIGATION

2.1 Indirect method

Peak Particle Velocity (PPV) is a method where the wave amplitude of a pressure wave after blasting is measured (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 grow after blasting (Holmberg and Persson, 1979). The assumption was based on that each rock has a breaking point; a critical particle velocity. When this velocity is exceeded the rock mass will be fractured. An empirical correlation was found consisting of the explosive concentration (Kg) detonated at once and site specific constants. The latter are influenced by many operational parameters as well as the geology of the site. In the 1990s adjustments were made on the PPV theory (Ouchterlony et al., 1991), by the use of a corrective master curve. Olsson and Ouchterlony (2003) presented a new theory incorporation operational parameters (decoupling ratio of the charge), explosive properties (Velocity of Detonation, explosive energy and density of explosive) and rock factures (fracture toughness). They found fracture length has to be corrected for hole spacing, initiation scatter, water in the drill holes and rock types degree of fracturing.

2.2 Semi-direct and direct methods

Half cast factor (HCF) is the ratio of half cast or barrels visible after the blasting in comparison with the number of contour holes drilled. The HCF is commonly applied in hard, competent rock masses. A high HCF indicates a stable, competent rock mass with limited blasting damage and a low number of natural fractures. Some consideration has to be taken into account; the HCF can give false impressions (Lizotte et al., 1996). The HCF is not applicable in soft, weak or heavily fractured rock masses either (Lizotte et al., 1996).

Scaling time is the duration of scaling of the underground construction. Scaling is the operation where the lose rock is broken away by either hand or a mechanical hammer. In Scandinavia mechanical scaling is widely used. Several publications use this parameter to determine the rock conditions of the tunnel. It is seen as a basic indication of the rock mass quality (McKown, 1986; Lizotte et al., 1996; Scoble et al., 1997). The damaged rock volume can be estimated with the scaling time, incorporating the geology and geo-mechanical properties in the rock mass. Although scaler operator depended skills should be taken into account.

Loading tonnage is related to the total rock mass volume that has been taken out. The volume over the practical excavated material can be seen as over-break or the excavation damage. An accurate calculation of the volume mucked includes the rock mass density and the swell factor.

Loading time can give similar results as the loading tonnage, if the muck pile, fragmentation and bucket load is consistent.

Cavity Monitoring Scanning (CMS) is used to measure the volume of an excavated area. Tunnels are scanned for quality control and drift mapping. The drift scanning gives information of the volume of material excavated. This is compared to the practical minimum volume (volume including the look-out angle). This value gives an indication of over-break and can be used in cooperation with geological information for estimating the damage zone.

3D-photogrammetry use photographs to form a three dimensional model of the tunnel. This model can be used in a similar fashion as the CMS.

Surface mapping or *fracture mapping* is done to investigate the geological structure along the tunnel. The most common systems are the Q-system (Barton, 1974) and Rock Mass Rating

(RMR) (Bieniawski, 1974). These systems look at the fracture density (RQD, (Deere and Deere, 1988)) and fracture filling among other parameters. New, un-weathered fractures indicate fractures developed during the excavation. These fractures are related to the excavation damage zone. The intensity of the number of fractures can be used to estimate the relative blasting damage.

Ground Penetrating Radar (GPR) sends high frequency waves (up to 2500 MHz) into the rock masses. Anomalies, for example fractures will reflect the waves and are recorded. Micro fractures will cause a loss of energy as dispersion and can be identified as a damage zone (Silvast and Wiljanen, 2008; Heikkinen et al., 2010). Unfortunately, the GPR cannot be applied in tunnels which use steel fiber shotcrete since the fibers reflect the wave energy (Silvast and Wiljanen, 2008).

Core drilling and *rock slicing* are techniques where rock samples are taken and visually inspected for fractures (Jern, 2001; Ittner et al., 2016; Van Eldert et al., 2016) and type of fractures; natural or blasting induced. This can give the depth of the damage zone as well as the behavior of this zone. In rock slicing a 3D image of the fracture development (Olsson et al., 2009) and the EDZ can be given where in diamond core drilling only information along the hole is extracted, these cores can be logged for RQD, rock and fracture types.

P-wave measurements on drill core samples indicated the extent of micro fractures in the rock mass. Diamond core samples can be measured diametrically to see the change of behavior of the P-wave velocity (Van Eldert et al., 2016). In rock samples with micro fractures the P-wave velocity is reduced (Jern, 2001) has shown a quadratic correlation between the P-wave velocity and the elastic modulus (Young's modulus) of the rock mass. The relative decrease in velocity indicates the EDZ.

3 CASE STUDY

The Stockholm By-Pass improves the North-South connection and will reduce the traffic in the Essingeleden and the Stockholm's inner city (Trafikverket, 2015b). The construction exist of 21 km of new roads, where of 18 km is located in tunnels, as shown in Figure 2. The construction of the first tunnels started in 2016. The By-Pass is scheduled to be open for traffic in 2026.

The different methods discussed in the previous section were applied in two ramp tunnels in the Stockholm By-Pass excavated by Subterra AB, see and Figure 3.

Figure 2. Stockholm by-pass (Trafikverket, 2015b).

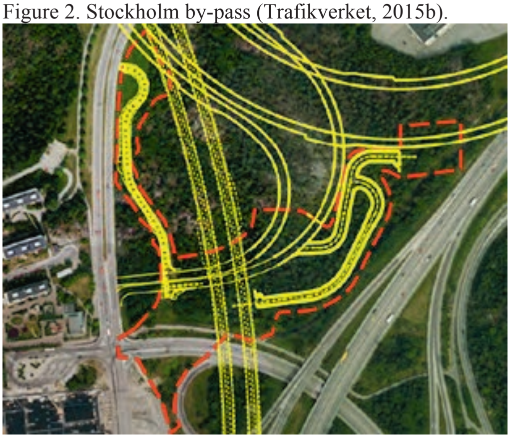

Figure 3. Subterra's consession (Trafikverket, 2015a).

4 RESULTS

During the excavation of the two ramp tunnels several methods for the excavation damage study were performed. The headings below show the results of each investigation technique employed.

4.1 Half Cast Factor

The HCF was not continuously measured in this case study, since the half barrels were not visible along the whole tunnel. Figure 4 and Figure 5 show an example of a blast, where the half cast are visible. The drilling data, Measure While Drilling logs, shows that 49 holes are drilled in the contour. In the blasted contour, 21 half cast were, partly observed, resulting in a HCF of ~40%.

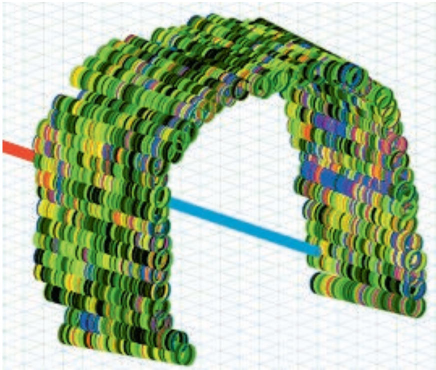

Figure 4. Drill holes in round 0/285 in tunnel 213.

4.2 Cavity Monitoring System

Drift scanning is done on a regular basis; every two to three blasts. The tunnel volume measured with the CMS, Figure 6, is compared with the designed profile, the drilling plan, and the drilling report (hole location, direction and lenght), as shown in Table 2 and in Figure 7. Table 2 shows that the effects of drilling can be significant and have to be taken into account. Besides it has shown that in this case the over-break from the blasting and scaling is low (3.7%). This indicated a limited failure zone.

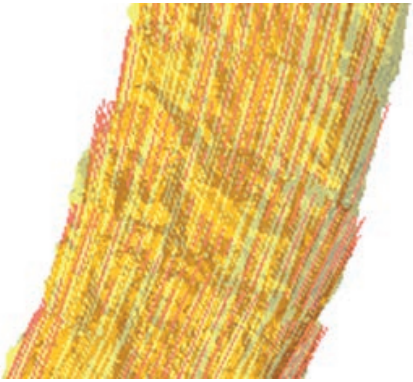

Figure 6. Part of tunnel 214 drilling log and tunnel scan.

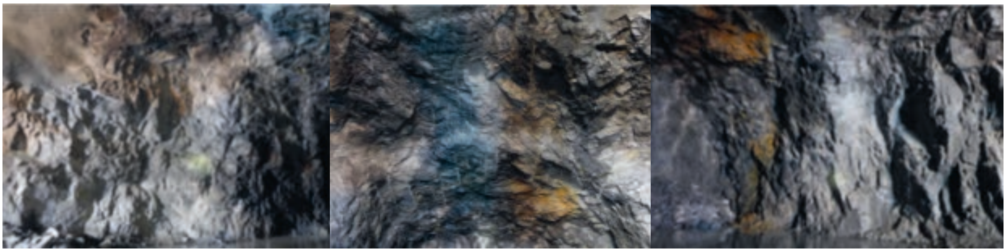

Figure 5. Half casts in in round 0/285 in tunnel 213.

Table 2. Volumetric change of a tunnel.

		Volume from		
	Volume	Design profile	Drill Plan	Pre-Blast
Design profile (Profile & length)	1577 m³	-	-	-
Drill Plan (incl. bottom holes)	1656 m³	79 m³ (5.0 %)	-	-
Pre- Blast (excl. bottom holes)	1633 m³	56 m³ (3.6 %)	-23 m³ (-1.4 %)	-
Post- Blast (excl. bottom holes)	1694 m³	117 m³ (6.6%)	38 m³ (2.2 %)	61 m³ (3.7%)

Figure 7. Over-break compared from the drill log, section 0/845-0/831 in tunnel 214.

4.3 Surface Mapping

Directly after mucking the tunnel surface is mapped for geological structures, displayed in Figure 8. The mapping shows areas of over-break (gray), fracturing (colored lines) as well as geological structures (colored zones). For each section of the tunnel the Q-value is calculated and an example is shown in Table 3. This mapping can be used to record over-break areas and indicate probable causes. The mapping can be utilized to record blasting and blasting indicated fractures. The over-break areas in Figure 8 correspondent well with the over-break measured with the drift scanning, displayed in Figure 7.

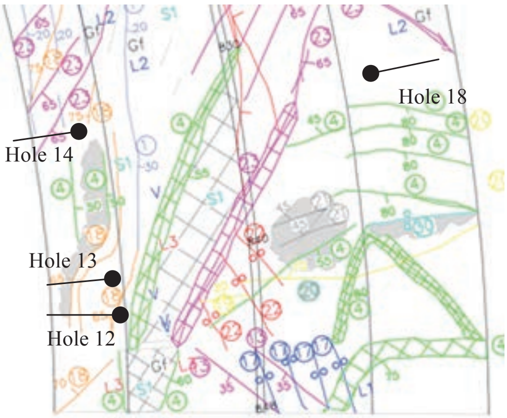

Figure 8. Mapping of a tunnel section, section 0/848-0/827 in tunnel 214.

4.4 Ground penetrating Radar

Ground penetrating Radar (GPR) data was recorded along the tunnel walls in a similar fashion as done by Van Eldert et al. (2016). Figure 9 shows an example of the GPR data collected on the left wall in section 0/848 to 0/827 in tunnel 214. The image shows and high amount of reflection/energy lost in the first 20 cm of the rock mass. This can be indicated as fracturing and blasting damage zone in the form of micro and macro fractures. This zone is marked with the blue line in Figure 9. Deeper in the rock mass other reflections are observed, as marked with the red ellipses in Figure 9. These reflections indicate macro-fractures in the rock mass. This could be related to blasting fractures, but most likely natural factures, most of them can be seen in the tunnel mapping.

4.5 Core Drilling

After the excavation a concrete drill, Hilti DD200, was used to take diamond drill core In total 20 cores were drilled in the concession. In this case study the focus will be on four holes drilled in section 0/848 to 0/827, displayed in Figure 8. The cores were taken to a rock depth up to one meter, Figure 10 and Figure 11. The drill cores were mapped for rock type and fractures, as displayed in Figure 10. They show the quality of the rock mass can vary a lot. The diorite in hole 12 is heavily fractured while the gneiss in drill hole 13 has very little fracturing. The cores are located less than two meters apart, but show differences in rock type and fracturing. In this case the fracturing in drill hole 12 is caused by the geological features. The fractures are filled and clearly not fresh. A natural fracture zone at the tunnel portal has been indicted in the pre-investigation, during drilling with Measurement While Drilling (MWD) data and during the excavation. The rock mass in hole 13 and hole 18 show much less breakage, as displayed in Figure 10 and Figure 11. When looked at the cores closely, fresh, new fractures can be observed in the first 30-40 cm of the rock mas, indicating the excavation damage zone.

Table 3. Mapped and calculated values along the tunnel 214 (Graphite opserved in 848-845).

Sec-tion	Section Part	Rock Type	Grain size	Weather-ing	Struc-ture	Compres-sive Strength (MPa)	Q-Values						Q val-ue
							1. RQD	2. Jn	3. Jr	4. Ja	5. Jw	6. SRF	
848–845	1 Left half	Gneiss	fine-large	W1-W3	B4-S1	10-250	80 (50)	9x2	1	8	1	5	0.1
840–845	2 Right half	Gneiss	fine-large	W1-W2	B5	100-250	60	9x2	2	2	1	5	0.6
845–840	1 Back	Gneiss	fine-large	W1-W3	B5-S1	50-100	50	12	1	8	1	5	0.1
840–833	2 Walls	Gneiss	fine-large	W1	B4	100-250	90	9	2	2	1	5	2
833-823	1 Back	Gneiss	fine-large	W1-W2	B5-S1	100-250	60	12	1	13	1	1	0.4
833-823	2 Left wall	Gneiss	fine-large	W1	B5-S1	100-250	60	12	2	2	1	1	5
833-823	3 Right wall	Gneiss	fine-large	W1-W2	B5-S1	100-250	75	6	2	4	1	1	6.25

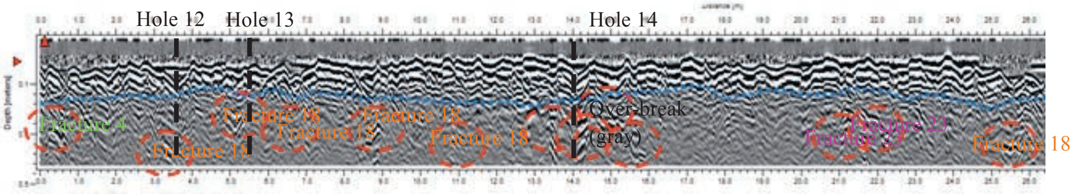

Figure 9. Ground Penetrating Radar on the left wall in section 0/848 to 0/827 in tunnel 214.

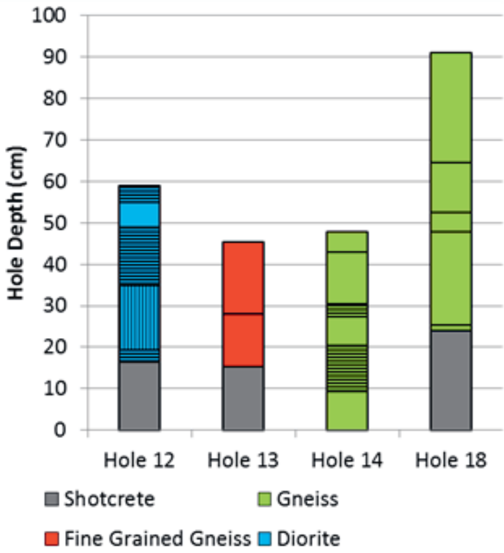

Figure 10. Drill core mapping.

4.6 P-Wave Velocity

The collected drill core can be used for other tests, in this case study P-wave velocity tests were performed. Figure 12 shows the P-wave velocity collected from the drill cores from the drill holes 12, 13, 14 and 18. The P-wave velocity in each of the drill cores is measured diametric at 2 cm intervals at two points under a 90 degree angle as discussed by Eitzenberger and Nordlund (2002). In some cases both or one of the diametrical measurements could not be measured as shown in Figure 11 and Figure 12. The graph in Figure 12 shows that the P-wave velocity stabilizes in the depth zone of 10 to 30 cm. This is in concurrence with the observed cores, in Figure 10 and Figure 11, as well as the depth of the GPR radar, Figure 9.

Hole 12	Hole 13
Hole 14	Empty
Hole 18	Empty

Figure 11. Drill core collected from drill holes 12, 13, 14 and 18.

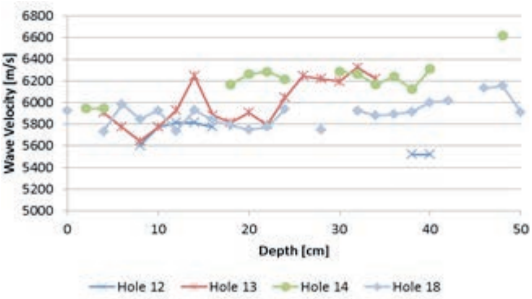

Figure 12 P-wave velocity in diamond drill cores in section 0/848 to 0/827 of tunnel 214.

4.7 *Other Methods*

Other methods were not applied but could have provided additional information. In this case study the scaling time was not measured. In other cases, for example Boliden's Kristinaberg mine, all the activities within the excavation cycle are logged (Van Eldert, 2014). With this information, an index of the amount of scaling and thus the rock conditions can be given. The same posibilities can be performed with loading time and loading tonnage. The loading tonnage requires an installed scale on the loaders.

4.8 *Comparison of Methods*

The techniques described and applied in this paper can determine geological structures blasting damage. Some of the techniques, PPV, scaling time, loading tonnage and time give very rough estimates. Other methods, core drilling, P-wave measurements and cavity scans, give very exact values on the over-break and damage zone. On the other hand, the latter are relatively costly and/or time consuming.

The commonly used techniques (PPV, Half Cast Factor or Standardized Blasting Tables) look only at indication parameters. Since several excavation and rock mass parameters are not taken into account. These methods, HCF and mapping, are not objective and depends on the quality of recording. Other methods use production data directly. This data is collected during the excavation process and does not need additional activities or equipment. This makes the method relatively low cost, but has limited accuracy. Methods with a high accuracy and a direct measurement are often time consuming and/or relatively costly. Table 4 shows the benefits and limitations of the different investigation methods. This table can be used to select and appropriate method for the investigation of over-break and blasting damage. The selection procedure should depend on the requirements, layout of the tunnel and the excavation operation.

5 CONCLUSION AND DISCUSSION

The techniques described above have shown improved methods to measure over-break and the Excavation Damage Zone in underground construction. The methods have a large degree of concurrence and can be seen as an addition to each other.

The paper shows the Excavation Damage Zone can be measured in many different ways. For any excavation a suitable method of investigation can be selected. The results can be used to improve the excavation process and the tunnel quality. By adjusting drilling and charging activities depending on the excavation damaged determined.

Lastly the author wants to emphasize the importance of good quality, control and following up processes in conventional tunneling. In order to minimize over-break and the EDZ accurate drilling and charging is required.

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Table 4 Methods for over-break and Excavation Damage Zone investigation.

Method	Benefits	Limitations
Peak Particle Velocity (PVV)	No production interruption, already recorded data (vibration)	Not a good picture, ignores many parameters, based on arbitrary relationships, site specific
Standardized Blasting Tables	No production interruption, predicting damage based on explosive charge	Based on PPV, based on arbitrary relationships, site specific
Half Cast Factor	Limited interruption, simple, very fast	Only surface data, minimal depth
Scaling time	No production interruption, already recorded data	Indication only, depending on operator quality and skill
Scanning	Accurate method, objective	Needs practical contour and drill hole location to establish over break and EDZ, time consuming (~45 minutes)
3D photogrammetry	Limited interruption, good indication,	Shadowing, needs practical contour and drill hole location to establish over break and EDZ
Loading tonnage	Clear tonnage of rock mass, production based data	Needs practical contour and drill hole location to establish over break and EDZ, needs accurate rock mass density and swell factor
Loading time	Indication on amount of rock mass, production data	Only indication, needs "loading tonnage", highly influenced by fragmentation and loading issues
Tunnel mapping	Clear picture of the surfaces, fracture orientation etc.	Only surface data, interruption of the production
Ground Penetrating Radar	Micro fractures can be detected, limited interruption, 3D picture, penetrated the rock mass	Not able to penetrate shotcrete, no difference in natural and blasting fractures, metal objects interfere, calibration and interpretation is needed
Core drilling	Fracture type and filling, penetrated the rock mass	Time consuming (interruption), sparse data collection
Rock Slicing	Fracture type and filling, penetrated the rock mass, 3D picture	Very expensive and time consuming (interruption)
P-wave velocity	Micro fractures can be detected, standardized method	Need drill cores or hole-to-hole data

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The History and Future of Rock Mass Characterisation by Drilling in Drifting - From sledgehammer to PC-tablet

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The History and Future of Rock Mass Characterisation by Drilling in Drifting
From sledgehammer to PC-tablet

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Abstract— In underground construction projects problematic rock mass conditions are one of the major issues causing cost overruns during the excavation phase. Before a tunneling project starts the rock mass is roughly characterized by a pre-investigation. However, in many cases, these pre-investigation does not portray the rock mass characteristics accurately and do not predict local anomalies in the subsurface. Therefore there is a need for new rock mass characterization methods that can reduce uncertainties and improve the overall tunneling process.

In the end of the 1880s, rock mass characterization based on manual drill data was investigated and rock masses were quantified using drillability. Since then, the technology has significantly changed with the introduction of hydraulic rock drills, computerized drill rigs, and advanced rock mass classification systems based on drill parameters. Nowadays, automatic drill logging systems and drilling data processing software packages are widely available and commonly used in Scandinavian tunneling projects.

This technology uses drilling parameters to characterize the rock mass. However, monitored drill parameters are influenced not only by the variations in the properties of the penetrated rock mass but also by the operator and the rig control system that continuously control the applied forces to optimize drilling and prevent jamming. In order to be useful for geomechanical purposes, the drilling data needs to be filtered, normalized and analyzed to refine the rock related response from responses caused by other influencing factors. If successful the data might be used to determine hardness, fracturing and water indicators.

Even though the technology has shown high potential in laboratory tests and field trials, it is not an obvious choice for all tunneling projects. In this paper, the background of the technology are described and the potential for the future outlined, concluding that the technique probably will be used more extensively in the future.

Keywords: measurement while drilling, MWD, drilling, rock mass characterization

1. Introduction

In underground construction projects, problematic rock mass properties are one of the major issues causing cost overruns during the excavation [1]. Before a tunneling project starts the rock mass is roughly characterized by a pre-investigation. This is normally based on previous experiences, for example, excavated drifts nearby, surface investigations and a limited number of diamond core (DC) drill holes. These estimated rock properties determine the excavation method, excavation advancement, material wear and rock support required. However, in many cases, the pre-investigation does not portray the rock mass characteristics accurately and do not predict local anomalies in the subsurface. The additional costs for this information shortages include extra mucking due to over-break, additional rock support, e.g. increased number of bolts and increased volume of shotcrete and tunnel lining, due to over-break as well as an irregular tunnel contour; and maybe most important, delays resulting in decreased tunneling advancement rates.

2. Measurement While Drilling

Measurement While Drilling (MWD) technology is used to log drilling parameters during drilling [2]. The drilling parameters include operational pressures, penetration rate, flush flow etc. In most Scandinavian tunneling projects MWD data is collected, but not always utilized during excavation [3, 4]. The major benefits of MWD data compared with DC are the high data density and the low data cost. Since the late 1990s, MWD recording options are available on face drill rigs produced by e.g. AMV, Atlas Copco, and Sandvik. Likewise, a similar system is available for (coal) mine roof bolters from J.P. Fletcher [5]. Nowadays, onboard analysis of drilling data can be performed on the latest generation of drill rigs [6, 7].

2.1. Drill Data Logging

The use of drilling data to characterize rock masses dates back to 1888 [8]. The collected parameters contained information on the amount of work per hammer blow and the number of hammer blows applied for removing one cubic centimeter of rock. This parameter was used for categorizing different tunnels and rock types [8]. This method of rock mass characterization came to a hold with the introduction of air powered rock drills. In the 1970s mechanized drill rigs were developed and rudimentary machine parameters, e.g. oil pressures, penetration rates, rotary speed etc. could be recorded [9]. However, with the introduction of these new drill rigs, several issues had to be addressed to collect reliable data. Measures taken included: exact positioning of the drill rig and boom, development of a rig control system, optimization of boom interaction, development of an automated drill bit change and data logging system [9]. The first-generation computerized rock drill rigs recorded basic drilling information, e.g. the number of holes, drilled meters, penetration rate, and drill-hole sequence. Furthermore, this new control system had an automatic optimization of feed pressure and rotation speed to optimize the penetration rate. The system showed its benefits with an improved drilling rate in varying rock mass conditions, reduced bit wear and a better tunnel contour quality. All these resulted in an overall reduction of excavation cost [9]. Besides, this development resulted in improved anti-jamming systems [10]. Conjointly, these drill control systems reduced the drill-hole deviation by adjusting thrust, torque, penetration rate and percussive pressure while optimizing drilling rates. With the gradual improvement of drill control systems, the drill rig operator's work shifted from manual drill control to drilling supervision. Apart from the drill control, drilling parameters started to be used to investigate the existence, location, and aperture of fractures in the rock mass, as well as variations in lithology and estimations of the Uniaxial Compressive Strength of the rock mass [11, 12]. In the 1990s, laboratory tests were undertaken in Sweden [13] and in the USA [14] to predict these rock mass properties. These lab tests were followed by field trials [2, 11, 15-17], during which penetration rate, drilling pressures, and rotary speed were recorded. These measurements were then plotted against hole-length and drilling time. The field tests showed [18], that in homogeneous rock masses the drilling parameters are constant; while in heterogeneous rock masses the drilling parameters show variation in thrust and torque. Based on these and similar experiences, today's MWD system has been developed [2, 17].

2.2. Measurement While Drilling Parameters

MWD data files contain a complete recorded of the drilling operation. Firstly, it contains basic drilling information, e.g. hole ID, hole type, navigated drill rig location, hole collar location, hole depth, time-stamp as well as the drilling settings and recording settings. Secondly, the collected drilling data, i.e. operational values, are recorded at a set sample distance; ranging from minimum 2 cm and upwards. The recorded drill parameters can be separated in independent and dependent parameters [10]. The independent parameters are not influenced by the rock mass, but solely by the drill settings, operator, and control system. These independent parameters are bit thrust or percussive pressure and rotation speed. The dependent parameters are influenced by the drill systems response to varying rock conditions and are typically penetration rate, torque or rotation pressure, damper or stabilization pressure as well as flushing flow and pressure. The thrust is applied to ensure contact between the hole bottom and drill bit throughout the impact and the torque is the energy required to overcome the friction between the drill steel and hole wall and the rotation resistance between the bit and the hole bottom.

2.3. Geological Factors on Percussive Drilling

Rock mass characteristics are known to have a major influence on the drilling performance [19]. These factors are well known and likewise well described:

- Compressive rock strength has a negative correlation with drilling performance [19-21]
- Rock texture has different influences on the drilling performance [19]. The rock texture at a grain or mineral level is a considerable factor in drill fracture initiation and continuation, e.g. grain orientation to the applied stress field, grain size, grain interlocking and grain boundaries roughness. The fracture propagation requires less energy along the boundary, intersecting with grain boundaries is a barrier for fracture propagation. On the contrary, grain boundary roughness has a negative influence on fracture propagation and therefore drilling performance [19]
- Rock mass structure, e.g. joints, intrusions, and foliation, affect the drill performance [19-22]. In the case of anisotropic foliation or sedimentary layers within the rock mass [21], the drillability can differ depending on the angle between the geological structures and drilling direction, i.e. drilling at a 60° angle to the foliation requires the lowest amount of work [20]. In addition to effects on drillability of the rock mass, the foliation and sedimentary layering in the rock mass might cause drill hole deviation [20]
- Mineral composition will affect the drilling performance [8 19 20]; hard minerals (Mohr's scale > 5.5) reduce the drilling rates and increase wear [19]
- Rock cavities cause major increases in penetration rate and water losses, increasing rotary friction along the drill hole and therefore wear [20].
- Porosity and permeability may influence drilling performance to a great extent [20]. High porosity and permeability might drain out the flushing water, increasing rotary friction. Further drilling in sedimentary rock masses only requires the breakage of cemented bonds between the grains, resulting in an increased penetration rate [20]
- Weathered rock masses increase the penetration rate due to their lower hardness, but with a higher risk of drill hole collapses and jamming [5, 20], especially in rock masses with a high clay content [5]

2.4. Drilling Technique Factors on Percussive Drilling

Operational factors are known to have a major influence on the drilling performance. The major factors of influences include:

- Operator skill and experience. During collaring the drilling is performed less aggressive, with reduced percussive pressure and feed pressure, in order to minimize hole deviation and wear, as well as the proficiency to adjust angle of the drill during drilling [18, 20, 22-25]

- The choice of rig, rig capacity, and hammer type determine the optimal penetration rate in varying hard rock masses [20, 25]
- Flushing capacity used for flushing out cuttings and clay. In clay-rich areas a low flush flow may not flush the hole sufficiently, increasing rotary friction in the drill hole [5, 20, 26] Bit type, button versus cross-bit, rod and bit status as well as bit wear and button breakage, impact the drill rate significantly due to friction at the hole bottom and rock breakage capacity [5, 20, 25, 26]

2.5. Normalization of MWD Data

The rock mass, drill rig, and operator influence the recorded machine data [15, 27]. The influences from the rig control and the operator should be removed, in such manner that only features of the rock mass are taken into account [17, 28, 29]:

- In the beginning of the drill hole, when careful collar drilling is applied, i.e. reduced drilling pressures and speed, and the following ramping up the values towards regular drilling settings should be removed from the data set entirely since they do not represent normal drilling conditions [28]
- The drilling process is heavily influenced by the drill hole length. The hammer energy, rotation energy, and hole flushing become less effective at depth [29], due to increased friction and wearing of the drill bit [29] The rotary pressure increases with depth for all drill holes, due to increased friction between drilling rods and hole wall.
- In longer hole drilling, e.g. grout and probe holes, drilling data shows clear drops of feed pressure and penetration rate. This due to energy losses in the couplings of the extension rods during the drilling of these holes [29].

One way of data normalization is summarized by [28]:

1. Remove the data collected during collaring
2. Eliminate the effects of hole length on the parameters, by using correlations between the drill hole depth and MWD parameters [17].
3. Normalize the variation due to thrust differences
4. Remove influence of penetration rate on torque

3. Analysis of Measurement While Drilling Data

Analysis of the MWD data can be performed in different ways:

- Single parameter analysis, where the independent drilling parameters as far as possible are kept constant and the dependent parameters, i.e. penetration rate, are observed and correlated to rock mass conditions [29]
- Dual parameter analysis, where the interaction between two parameters is taken into account, i.e. feed pressure and penetration rate [20], the interaction will be used to assess the rock mass quality
- Rock mass indices, produce a calculated rock mass parameter based on the drill performance, e.g. Specific Energy for Drilling [30, 31], Alternation Index [24], Rock Quality Index [24], Drillability index [32], and Drilling Energy Index [6]
- Pattern recognition uses a training image or patterns from previous data to identify rock mass characteristics by recognizing these data patterns in the drilling data [33]. The training images contain "standardized" drill behavior for when drilling e.g. fractures, different strata or cavities. The recorded data is then compared with training image
- Multivariate approach incorporates all recorded parameters in order to describe the rock mass characteristics accurately. This method includes higher order statistics and handles drilling parameters intercorrelation [28]. This combined behavior is then related to the rock mass characteristics

In order for the calibration to be accurate, extensive measurement and testing campaigns are necessary [27, 29].

3.1. Analysis of Measurement While Drilling Data

The first rock mass characterization with drilling logs has been performed by Rziha [8] in 1888. He used the destructive work [m*kg] per volume of rock drilled [cm³] to characterize the rock mass. The work per hammer swing was calculated based on the weight of the sledgehammer and drill steel and the end velocity of the swing, multiplied by the number of blows, see Equation 1. The collected data was separated into different rock categories. The data collected showed a good correlation between the rock strength and mechanical work [8]. The same concept was used for the Specific Energy for Drilling (SED) in rotary drilling [kg/m³], equation 2 [30], and refined for percussive drilling, equation 3 [31]. These concepts were further developed into drillability classification methods. Examples of such methods are the Alternation Index based on relative feed pressure and penetration rate [24], the Drillability index based on an empirical study correlating drilling performance with the Brittleness Value and the Sievers' J-Value [32], Rock Quality Index for blastability based on the variation of drilling parameters [24] and Drilling Energy Index [N/mm³] for water driven In-the-Hole hammers by multiplying the water pressure by the number of blows per unit length drilled [6].

$$Work = \frac{Hammer\ weight^2 * swing\ velocity^2}{(Hammer\ weight + Drill\ weigth) * 2 * g} \qquad (1)$$

$$\mathrm{SED} = {}^{\mathrm{W}}/_{\mathrm{V}} = \frac{\mathrm{Thrust}}{\mathrm{Hole\ Area}} + \frac{2\pi * \mathrm{Rotation\ Speed} * \mathrm{Torque}}{\mathrm{Hole\ Area} * \mathrm{Renetration\ Rate}} \qquad (2)$$

$$\mathrm{SED_{perc}} = \frac{4 * \mathrm{Transfer\ coefficient} * \mathrm{Power\ output\ drill}}{\pi * \mathrm{hole\ diameter} * \mathrm{Penetration\ Rate}} \qquad (3)$$

3.2. Hardness Indication

MWD data analysis packages have commonly used a "Hardness" parameter. This parameter portrays the drillability of the rock mass according to the filtered and normalized penetration rate [34], where a higher "Hardness" value indicates soft or fractured rock masses and a lower "Hardness" value indicates solid competent rock masses [29]. For uncalibrated data, a relative ranking is used, but in many projects an absolute value of the hardness is preferred; mostly the Uniaxial Compressive strength. The calibration can be performed based on Schmidt hammer test, Point Load test, Brazilian Tensile Strength test or UCS test [35, 36].

3.3. Fracture Indication

A fracturing parameter is available in the different MWD software packages. This parameter is based on the variation of the normalized penetration rate and normalized rotation pressure or torque [15, 34]. This parameter reflects, in principal, the heterogeneity of the rock mass [37]. Open, clean fractures result in an increase penetration rate, rotation speed and reduced torque, thrust and water pressure; followed by an increase of thrust etc. when the fracture has been crossed. In poor, highly fractured, rock masses the drill holes will partly cave, will result in an increased rotary friction and therefore increased torque. In the worst case, this could cause jamming of the drilling rod [29]. This might result in a lower overall penetration rate. The calibration of the “Fracture Frequency” has been performed by correlating the measured data to rock mass observations, e.g. diamond core holes, borehole TV, Ground Penetrating Radar, and geological mapping of the tunnel. These methods have shown a reasonably good correlation between the “Fracture Frequency” and RQD or observed fractures [15, 29].

3.4. Water Indication

The water indicator is available on the MWD software packages and displays the normalized water flow. The algorithm detects sudden water losses or inflows in the drill hole, by incorporating the water pressure changes. This parameter indicates both water-bearing structures in the rock mass as well as dry fractures [34, 38-40].

4. Discussion

MWD data has been available for face drill rigs since the 1990s [10] and on roof bolters since 2002 [5]. Unfortunately, the data collected on rock masses have not been used to its full potential. The main purpose for MWD data is the recording for the following up process of the tunnel quality, operator performance and saving of data for a future investigation [4]. The MWD data can be used to assess drilling quality, e.g. drill hole collaring a drilling deviation, and improving the drill rigs overall drilling performance [23, 27]. Recent improvements in computing power and the need for increased excavation quality have driven the developments of MWD. The objective and detailed drilling data have a bright future. Although not implemented, the investigation for rock mass characterization, orebody boundaries, grades, blastability and the over-break investigation has been studied, see Table 1. In several studies, the application of MWD in rock mass characterization and rock quality assessment has been used with success [2, 6, 13-15, 24, 28, 31, 35, 41] as well as rock quality assessment during roof bolting [5, 14, 16, 33]. MWD in probing is often used to alter the excavation process, i.e. pilot tunnels, New Austrian Tunnel Method or the excavation method, Drill and Blast instead of Tunnel Boring Machine. Since MWD data has shown to be a good indicator for the rock mass quality and could be incorporated during the rock support design as part of the observational method [42]. Further, MWD data is nowadays used during grouting operations in tunnel construction [3, 7, 38, 40]. The MWD data is used for the decision whether or not to increase the number of grout holes in a single grouting fan. In the near future MWD data could be used as support in blast and fragmentation design [23], with the use of the MWD data the specific charge may be altered to minimize Blasting Damage and over-break [37], as well as optimize fragmentation, depending on fracturing, water inflow and drillability or blastability of the rock mass, see Table 1. Other applications of MWD can be found in mining where the technology is used to detect orebody boundaries and extinguish between grades [17, 29]. These methods are based on mechanical differences between ore and waste as well as an established correlation between drilling parameters, or drillability and ore grade. Unfortunately, these usages of MWD data are not commonly practiced. One major concern of the use of MWD data during excavation is the transfer and process the drilling data directly, the processed information, e.g. Fracture Indication or SED should be feedback to the drilling operator; directly inside the drill rig, on e.g. a tablet PC. Simple, indicative images, such as given by Alas Copco’s Underground Manager in Figure 1, or Sandvik’s iSure, could give the drill operator enough information to support the decision-making process during excavation.

Many underground operations for mining and tunneling are equipped with a WiFi (W-LAN) network; it could be utilized for information data transfer from and to the drill rig in order to give the drilling operator the best information available. These direct-information processes have been tested in Virginia State University, USA [5] and SKB HRL Äspö, Oskarshamn, Sweden. Examples of the utilization of WiFi networks in underground environments can be seen in the mining and tunneling industry. Where the information is transferred to a central database it can be used to optimize other activities of the excavation cycle, e.g. charging, rock support etc., as portrayed in Table 1. This information can be related back to the operators i.e. on the drill and charge crews. The operators could receive this information on e.g. tablet PC and use this to improve the excavation activities, i.e. rock fracturing for charge-ability, rock support or drilling of nearby faces or drill bit wear. Concerning the technological developments today, e.g. W-LAN, “internet of things” and “Big Data”, MWD data can be used to take mining and tunneling productivity to the next level.

Table 1-Status of MWD: P: Practiced, T: Tested, F: Future

Percussive Drilling	**Parameter**	**Application**	
Quality of drilling	Hole location/depth Penetration rate & Drilling accuracy	Drilling [13, 43, 44]	P
	Drilling accuracy	Blasting design [43, 44]	P
Operator quality	All parameters	Quality of drilling [23]	P
Rock mass quality	Fracture frequency (Penetration rate & torque)	Grouting [3, 7, 40, 45, 46]	P
		Blasting – Fragmentation [43, 44, 47]	T
		Blasting – Damage [35, 37]	F
		Excavation [13, 17, 21, 22, 35-37, 43-48]	P
		Support	T
	Hardness (Penetration rate & torque) Weathering	Blasting – Fragmentation [43, 44]	T
		Blasting – Damage [35, 37]	F
		Excavation [17, 21, 35-37, 43-48]	P
		Support	T
	Water inflow (Water flow & pressure)	Grouting [3, 7, 40, 45, 46]	P
		Blasting [7, 38, 49]	F
Ore body boundary	Hardness (Penetration rate & torque) Rock contact	Minimize dilution & ore losses [13, 17, 28, 29]	P
Mechanical rock alteration	Hardness (Penetration rate & torque) Variation in grades	Grade control [13, 28]	T

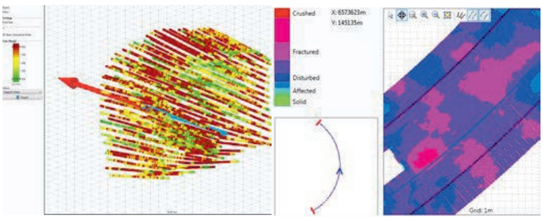

Figure 1. Atlas Copco Underground Manager penetration rate and "Fracture Frequency"

5. Conclusion

Rock mass characterization using drilling parameters has a long history and is driven by a need to have a more detailed characterization of existing geological uncertainty. This paper shows to what extent the drilling is influenced by geological features of the rock mass and operational parameters. With correct normalization and processing of the data, models of the rock masses based on the drilling parameters as well as drilling performance can be made and used during excavation. This will result in an improved tunnel quality at a lower cost, due to the increased knowledge of the rock mass and actions which are taken based on objective data.

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PAPER C

Measurement While Drilling to Predict Rock Mass Quality and Support

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Measurement While Drilling to Predict Rock Mass Quality and Support

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Abstract

A tunnelling project is normally initiated with a site investigation to determine the in-situ rock mass conditions and to generate the basis for the tunnel design and rock support. However, since site investigations often are based on limited information (surface mapping, geophysical profiles, few bore-holes, etc.), the estimation of the rock mass conditions may contain inaccuracies, resulting in underestimating the required rock support. The study hypothesised that these inaccuracies could be reduced by using Measurement While Drilling (MWD) technology to assist in the decision-making process. A case study of two tunnels in the Stockholm bypass found the rock mass quality was severely overestimated by the site investigation; more than 45% of the investigated sections had a lower rock mass quality than expected. MWD data were recorded in 25m grout holes and 6m blast holes. The MWD data were normalised so that the long grout holes with larger hole diameters and the shorter blast holes with smaller hole diameters gave similar results. With normalised MWD data, it was possible to mimic the tunnel contour mapping; results showed good correlation with mapped Q-value and installed rock support. MWD technology can improve the accuracy of forecasting the rock mass ahead of the face. It can bridge the information gap between the early, somewhat uncertain geotechnical site investigation and the geological mapping done after excavation to optimise rock support.

Keywords: Measurement While Drilling (MWD), Rock-mass investigation, Tunnelling, Rock mass quality, Rock support, Drill and Blast technology

1 Introduction

Before a tunnelling project starts, a site investigation is performed to determine the in-situ rock mass conditions. Information generated on the rock mass properties is then used to determine the tunnel design and the required type and volume of rock reinforcement (Barton et al. 1974, Lindfors et al. 2015). Site investigations use rock mass classification systems such as those introduced by Barton et al. (1974), Bieniawski (1973) and Hoek and Brown (1997). However, these systems often have limited information (surface mapping, geophysical profiles, limited bore holes, etc.), the rock mass classification may be inaccurate, leading to increased excavation time and costs (Wahlström, 1964; U.S. National Committee on Tunnelling Technology 1984; Kjellström, 2015). Furthermore, in tunnelling practice, classification is normally done in 5-metre sections (one blast/excavation cycle) along the tunnel, so smaller areas of poorer or better rock conditions may be ignored. Therefore, these systems may over- or underestimate the true rock mass conditions and the required rock support (Edelbro, 2004).

To minimise these problems, a more thorough and detailed pre-investigation could be an alternative, but reaching significantly higher accuracy is expensive. Another alternative could be to use information extracted during construction, but this requires shorter planning horizons and a more flexible organisation.

A more precise and objective method for rock mass characterisation and ultimately rock support design is Measurement While Drilling (MWD). The MWD technology documents the response of the drilling parameters (e.g. penetration rate, operational pressures, rotation speed, flushing flow etc.), while drilling (Schunnesson, 1996). It has been demonstrated to be an objective and reliable method to assess rock mass conditions ahead of the tunnel face (Schunnesson, 1996; 1998, Atlas Copco Rock Drills AB, 2009; Schunnesson et al., 2011; Humstad et al. 2012; Van Eldert et al., 2017).

This paper investigates the quality and usability of MWD data in tunnelling and suggests an approach to incorporate the use of MWD technology into the rock support design process in the production phase of a tunnelling project to improve time and cost efficiency.

The paper is based on a case study from the large Swedish infrastructure project Stockholm bypass.

2 Case study description

The Stockholm bypass is being constructed to improve transport links within the Stockholm metropolitan region. The bypass consists of a new 21km road, of which 18km will be underground (Trafikverket, 2018).

This study focusses on the first section of two access tunnels to the underground portion, Tunnel 213 and Tunnel 214; see Fig. 1. A separate site investigation was performed by Arghe (2013; 2016); the investigation included four diamond core holes (0-120m from the tunnels), two seismic lines, and the mapping of a nearby subway tunnel. The reports describe the rock mass as grey, medium to large grained gneiss with intrusions of lightly foliated granite, pegmatite, green stone, graphite amphibolite and severely weathered mica. They also identify four zones of weakness (Arghe, 2013; 2016).

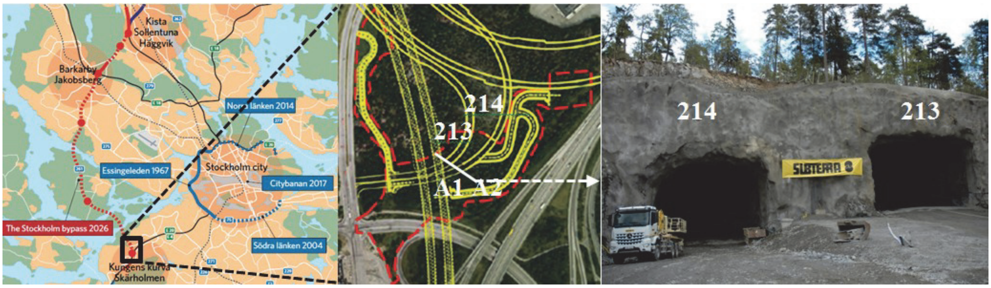

Fig. 1. Tunnel 213 and Tunnel 214 in Stockholm bypass (Illustrations courtesy of Trafikverket).

3 Methodology

The results presented in this paper are based on data collected before and during the tunnel construction.

A number of rock mass classification systems have been developed over the years; of these, Rock Mass Rating (RMR) (Bieniawski (1973) and tunnelling quality index or the Q-system (Barton et al., 1974) are the most commonly used. The Q-system is a tunnelling data-based empirical classification system that categorises the ground into nine rock mass classes and is used to estimate rock support. The quality index ranges from 0.001 to 1000 and is calculated using Equation 1.

$$Q = \frac{RQD}{J_n} \times \frac{J_r}{J_a} \times \frac{J_w}{SRF} \qquad \text{Equation 1}$$

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

The rock mass conditions along Tunnels 213 and 214 were established by Q-values in two phases: (i) “initial Q-values” from the site investigation prior to construction were used to establish the ground support requirements (Arghe, 2013; 2016); (ii) “mapped Q-values” from the tunnel mapping during construction (ÅF, 2016) were the basis for the installed rock support. Drilling for the tunnel construction was performed by an Atlas Copco WE3 3-Boom drill rig using three COP3038 hydraulic percussive rock drills. The rig was equipped with a fully integrated MWD system that recorded penetration rate, percussive pressure, feed pressure, rotation pressure, rotation speed, damping pressure, flushing flow and pressure at defined intervals along the hole. For this test, the sampling interval was set at 2cm, but because of the high penetration rate (Atlas

Copco, 2009), the actual sample interval was between 2cm and 3cm.

The MWD data were applied to calculate drilling indices: Hardness Index (HI) and Fracture Index (FI). In Atlas Copco's Underground Manager MWD V1.6 (UM), the FI is calculated based on the variation from the normalised penetration rate and normalised rotation pressure (Schunnesson, 1990). This variation reflects the heterogeneity of the rock mass (Schunnesson, 1996; 1998; Van Eldert et al., 2016).

During the tunnel excavation, MWD data were collected for grout holes drilled in fans ahead of the face and for blast holes. In Tunnel 213, the first 90m were monitored; in Tunnel 214, the first 220m were monitored. Altogether, MWD data were gathered from 583 grout holes (13 rounds) and 8,525 blast holes (34 rounds).

MWD data normally contain some faulty, biased or unrealistic data (Van Eldert et al., 2017), and these must be removed before analysis can be done to distinguish rock mass conditions. Some data points can easily be defined as incorrect, such as negative rotation speeds (reverse drilling), very high penetration rates (over 48m/min), or 0-values among the parameters. The data set will also have data that are correct but not common, and there will be a sliding transition from faulty data to abnormal drilling behaviour. This causes a problem when filtering data. Fortunately, for this type of high resolution of data set, a more extensive filtering can be done without losing the general pattern of the data. In this case, a conservative statistical approach was used to remove the highest and lowest values. Ultimately, 99% of the data points were accepted, with 0.5% removed at the low end and 0.5% removed at the high end. If one or more of the recorded parameters fell outside the interval, the entire sample was rejected. The filtering limit used by the study is shown in Table 1. The Hardness Index and Fracture Index were calculated using the Underground Manager software for filtered and normalised samples (Schunnesson 1996; 1998; Van Eldert et al., 2017).

Table 1. Filter limits for MWD parameters.

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 (lit/min)	0 to 261	≥60 and ≤210

Monitored MWD responses generally vary significantly with hole diameter and hole length. Penetration rate, for example, is higher for smaller holes and lower for larger ones. Penetration rate always decreases with increasing hole length. In this case, MWD data from two types of holes were available: blast holes with 48 mm diameter and 5.7m length and grout holes with 64 mm diameter and 20 to 25m length.

To compare the MWD response for the blast holes and grout holes, the data response must be normalised. The initial normalisation procedure used in this case follows Ghosh (2017):

1. Calculate the average or median (for symmetrically distributed data) or mode (for unsymmetrically distributed data) (Mo) and standard deviation (σ) of the selected parameter.
2. Normalise the residual of each data point using the standard deviation of the data, Equation 2.

$$\text{Normalise data} = \frac{\text{values} - \text{Mo}}{\sigma^2} \qquad \text{Equation 2}$$

Fig. 2 shows the normalisation of the Fracture Index for the grout holes and blast holes. The Fracture Index was normalised using the standard deviation. The Residual Index was not calculated, as the data set has a very similar response.

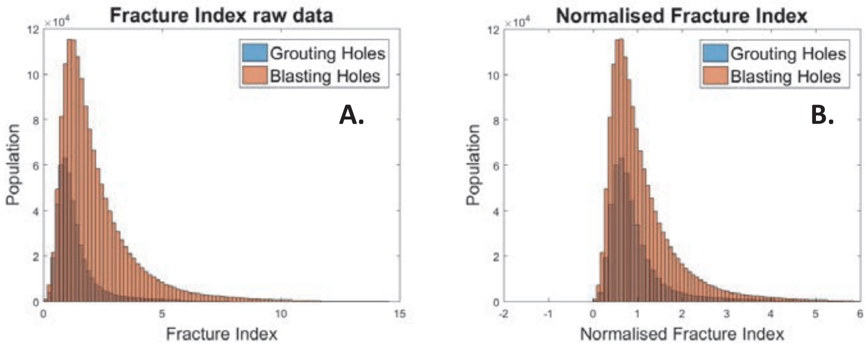

Fig. 2. Normalisation of the Fracture Index of the grout and blast holes; (A) raw data and (B) normalised data.

4 Results

The rock mass characterisation described by the Q-values was defined by the site investigation before excavation started and by tunnel mapping during construction. These data sets are compared in Fig. 3A for the first 90m of Tunnel 213 and in Fig. 3B for the first 220m of Tunnel 214.

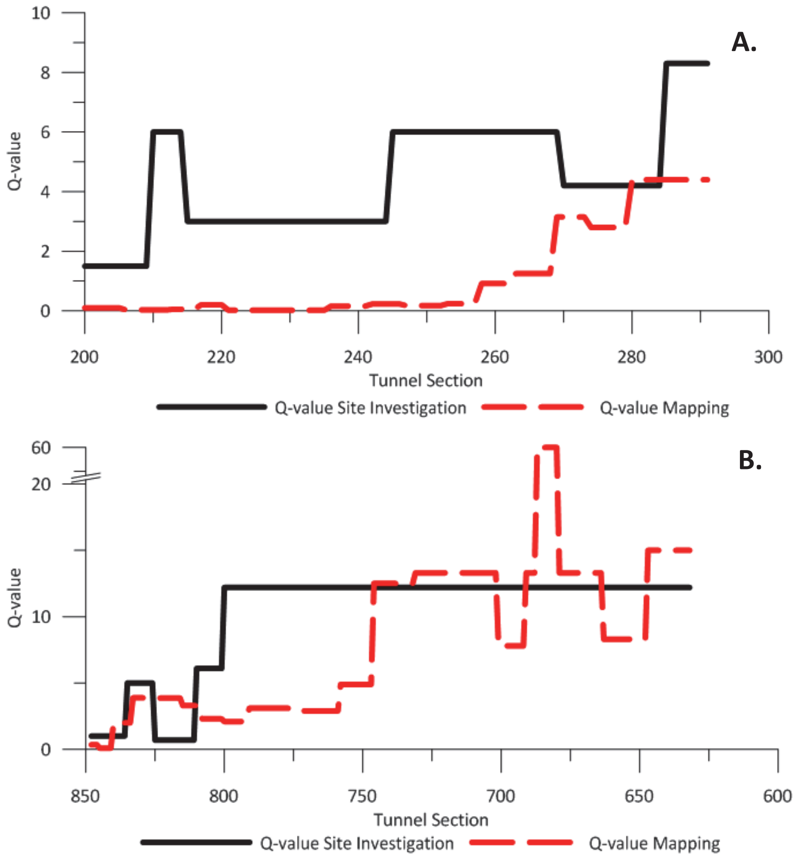

Fig. 3. Q-values established during site investigation (Arghe, 2013; Arghe, 2016) and from tunnel mapping (ÅF, 2016) for Tunnel 213 (A) and Tunnel 214 (B)

As the figure shows, in Tunnel 213, the first 80m of the tunnel had a significant lower mapped Q-value than estimated in the site investigation, indicating that the real rock conditions initially were much poorer than expected. In most of the section (63%), the mapped Q-value was 10 times lower than estimated. The conditions in Tunnel 214 followed a similar pattern, in that the first 100 m had a very low Q-value. In this case, however, the site investigation indicated low values for the first 50m, so the estimation was reasonably accurate. But there was a big mismatch between estimated and mapped Q-values for the following 50m. Furthermore, several fracture zones at the tunnel entrances were not noticed during the site investigation.

As a consequence of the large difference between the Q-value estimated by the site investigation and the mapped Q-value, the rock mass support was increased during construction. Fig. 4A-C compares the estimated bolt spacing, bolt length and shotcrete thickness, based on the site investigation, with the installed rock support during construction.

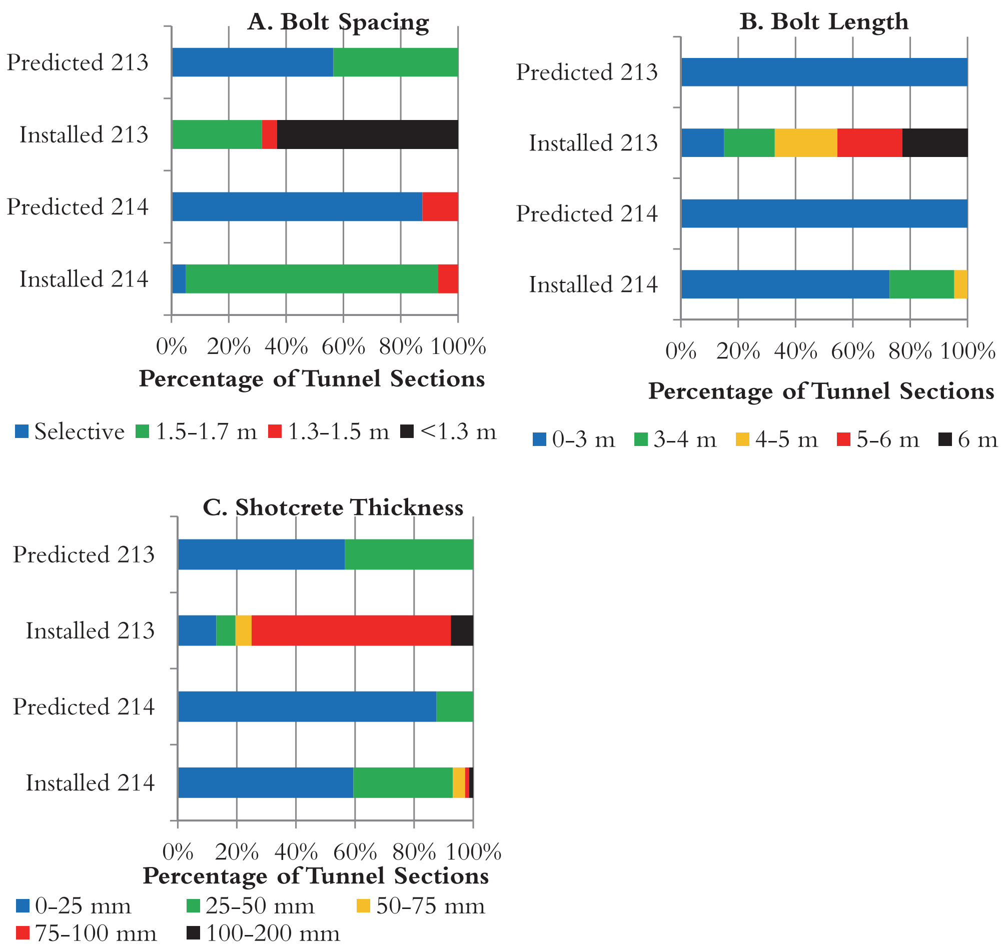

Fig. 4. Predicted and Installed Bolt Spacing (A), Bolt Length (B) and Shotcrete Thickness (C) in Tunnel 213 and Tunnel 214. Length is defined as the percentage of tunnel section, i.e., 90m for Tunnel 213 and 220m for Tunnel 214.

Because the rock conditions were worse than estimated by the site investigation, the bolt spacing was decreased from selective bolting to 1.7-1.3m in the major parts of both tunnels (Fig. 4A). The bolt length was increased from 3m to 6m in the least favourable parts of the tunnel, especially in Tunnel 213 (Fig. 4B).

An additional amount of shotcrete was installed in both tunnels. In Tunnel 213, the shotcrete thickness increased significantly from the amount estimated by the site investigation (Fig. 4C); overall, more than twice the amount of shotcrete was required. In addition, 200mm thick shotcrete arches were installed at the tunnel entrances.

The use of MWD may be able to minimise the consequences of the knowledge gap between the initial site investigation and the experienced rock mass conditions during excavation. It may increase the knowledge of the rock mass conditions and improve the accuracy of the rock characterisation ahead of the face. In the test site, MWD was used for both the blast holes and the grout holes in both ramp tunnels. Figures 5 and 6 compare the geotechnical mapping of the tunnels with the interpolated Fracture Index based on MWD data. In Tunnel 213, the Fracture Index shows highly fractured areas at the tunnel portal (i.e., section 200 to section 265) in section 230 to 232 and in section 238 to 260; this corresponds well with the geotechnical mapping of the tunnel. The fracture zones cross the tunnel at a 30° angle to the tunnel centre line. The location and the orientation of these areas are shown by the black lines in Fig. 5. In Tunnel 214, fracture zones are observed at the tunnel portal (the first 10 m) and in section 850 to 835 in the MWD data for the grout holes and the blast holes and in the geotechnical mapping. The location and the orientation of these areas are denoted by the black lines in Fig. 6.

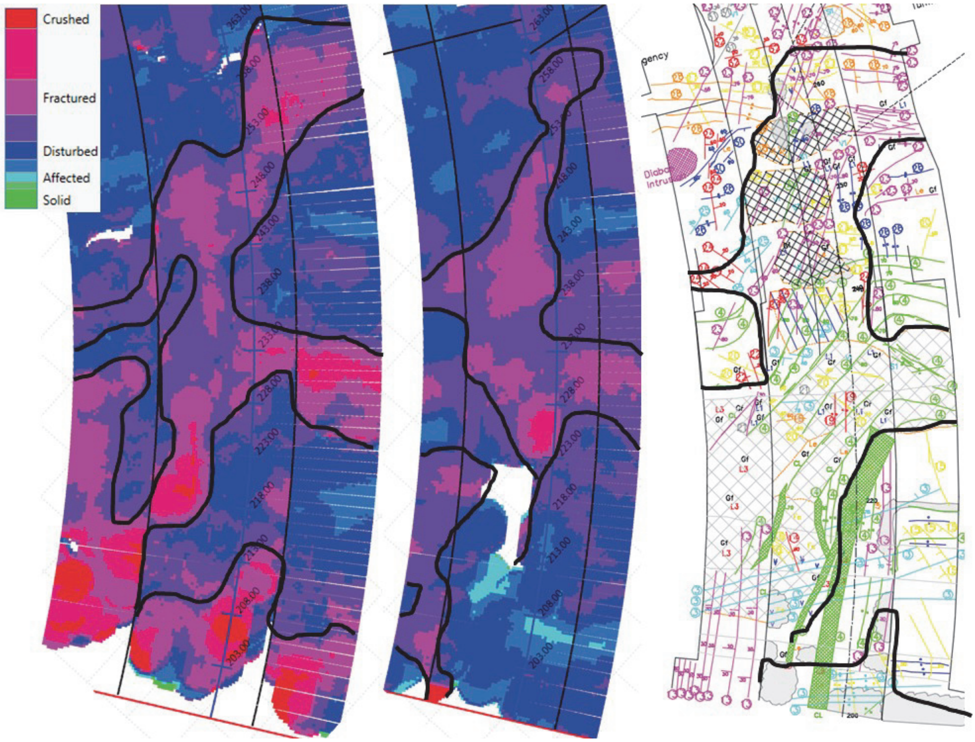

Fig. 5. Fracture Index from MWD data and geotechnical mapping for sections 200 to 263 in Tunnel 213 calculated in Underground Manager™ (left to right grout holes, blast holes, geotechnical mapping).

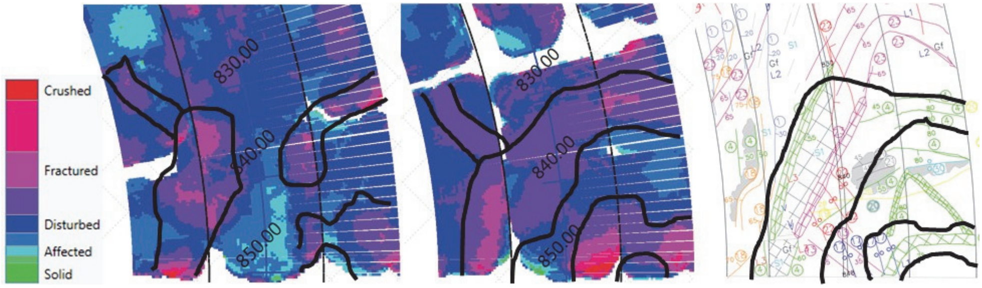

Fig. 6. Fracture Index from MWD data and geotechnical mapping for sections 850 to 820 in Tunnel 214 calculated in Underground Manager™ (left to right grout holes, blast holes, geotechnical mapping).

The calculated Fracture Index is based on the drill rig response when penetrating fractured rock. It is independently recorded and is measured at high resolution along every hole. This high-resolution data set facilitates identification of smaller zones in the tunnel and rock mass characteristics for the remaining rock mass (up to 5m from the rock surface), as the grout holes are inclined.

The Q-value is based on 6 parameters (see Equation 1); it is often estimated for entire tunnel sections, such as 5m blasts. Each parameter must be manually estimated. As the estimation of parameters requires skills and experience, the Q-value will be biased and influenced by personal differences (Edelbro 2004). Therefore, even if the Q-value theoretically includes a better and wider description of the rock mass, it is highly dependent on the person performing the mapping and is limited to visible parts of the tunnel. In contrast, MWD only records fracturing of the rock mass with very high resolution, including the rock mass outside the tunnel contour.

To test how well the Fracture Index correlates with the Q-value in the case study, the mapped Q-values were compared with the calculated Fracture Index from 24 different locations in both tunnels; see Fig. 7.

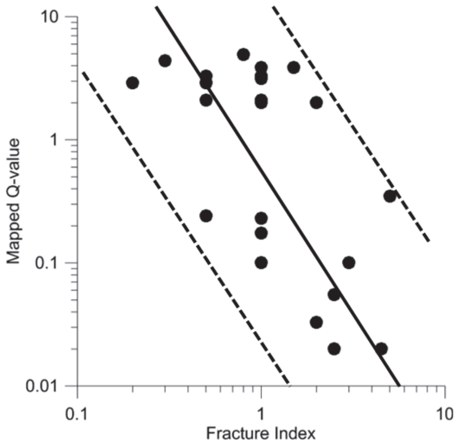

Fig. 7. Relations between mapped Q-values and MWD Fracture Index on a log-log scale.

The figure shows a clear negative correlation between Q-value and Fracture Index, where more fractured rock corresponds to a lower Q-value. In a construction like this, the Q-value is basically used to estimate required rock support, and with the correlation presented in Fig. 7, it may be possible to use MWD.

The rock mass conditions along the tunnel paths varied considerably (see Fig. 3), resulting in a large variation in installed rock support. The support options for the tunnels include bolts with different lengths (from no bolts to 6m bolts), different bolt spacing (from selective bolting to 0.8m spacing) and different shotcrete thickness (from no shotcrete to 200mm thickness).

Fig. 8 shows the correlation between MWD fracturing response and the installed rock support for 24 locations along the tunnels.

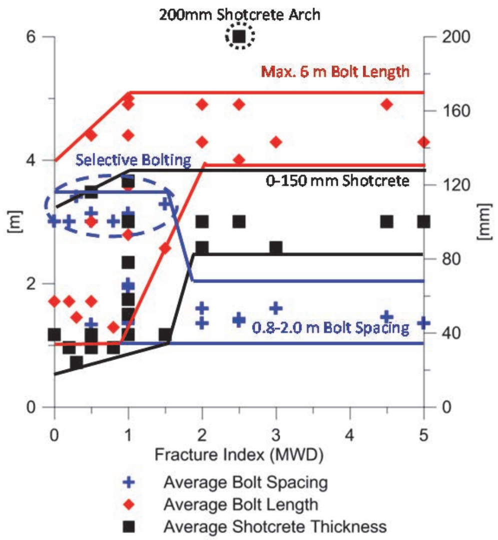

Fig. 8. Correlations between MWD Fracture Index and installed rock support in 24 sections of 1m along the two tunnels.

In general, the bolt spacing decreases and bolt length and shotcrete thickness increase in “poorer rock” with a higher Fracture Index. For a Fracture Index from 2 to 5, the installed rock supports are quite consistent for bolt length, bolt spacing, and shotcrete thickness. For a low Fracture Index, i.e., below 2, indicating a more solid rock mass, the variation is much larger. Some sections have shorter bolts, larger bolt spacing, and thinner shotcrete layer, and this is logical. However, other sections are similar to those with a high Fracture Index; some even have a thicker shotcrete layer. This phenomenon suggests the rock mass in some sections may be over supported.

5 Discussion and conclusions

The results from the study show the potential for the use of MWD to predict rock mass conditions ahead of the face. Fig. 5 and Fig. 6 show a clear correspondence between the MWD findings and manually mapped rock conditions. There is also a correlation between the Fracture Index, based on MWD, and the mapped Q-index.

The installed rock support correlates to the Fracture Index, except for low Fracture Index values (solid rock), where there is a large variation in applied rock support. There may be several reasons for this, but a discrepancy between the rock condition predicted by the site's pre-investigation and the rock condition experienced during excavation may result in more installed support than is actually required.

The traditional procedure for infrastructure projects based on pre-investigation and mapping during excavation may benefit from additional on-line rock mass information, even if the information comes at a late stage in the planning process. The MWD data are independently recorded, have high resolution and include information on the rock mass outside the tunnel profile.

To test the accuracy of MWD, the paper normalised and analysed MWD data for short blast holes and long grout holes. For the blast holes, the time interval between drilling and support installation is often very short, and the time available to make decisions on changes or modifications in the required support may be inadequate. However, the grout holes are drilled 20 to 25m ahead of the face, and the time interval before the face catches up can be a week or longer. This time interval may be long enough to re-characterise the rock mass and adjust the preliminary rock support design based on the seen rock characteristics. The following blast hole drilling can then be used to verify the expected rock mass conditions.

This procedure can reduce risk and may provide the opportunity to prepare the excavation work for unexpected rock mass conditions, hence reducing delays. The use of MWD data in the rock support design may also result in a more efficient workflow, as these data cover the information gap between the somewhat uncertain geotechnical site investigation and the post-excavation geological mapping.

The specific conclusions from this study are the following:

- The Q-value determined from the initial site investigation was inaccurate and significantly overestimated the rock mass conditions.
- The MWD data from grout and blast hole can be normalised to overcome differences in geometry of the drill holes, bit size and hole length to provide identical rock mass characterisation.
- The Fracture Index shows good correlation to the mapped Q-value, suggesting its potential as an additional information source for rock

support requirements.
- The use of MWD technology can bridge the information gap between the early, somewhat uncertain geotechnical site investigation and the geological mapping done after excavation to optimise rock support.

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Evaluation of Alternative Techniques for Excavation Damage Characterization

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Evaluation of Alternative Techniques for Excavation Damage Characterization

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ABSTRACT

Numerous aspects of underground construction, from structural stability to construction costs, depended on the tunnel quality, including blast damage and the Excavation Damage Zone. Accurately quantifying the extent and severity of damaged rock is a problem. Recent technical developments in the field of Measurement While Drilling (MWD), including software for on-board logging and on-site analysis, have shown potential for rock-mass characterization. Ground Penetrating Radar (GPR) and P-wave velocity measurement have also improved and show similar potential. This paper explores the use of MWD, GPR and P-wave velocity measurements and uses them in techniques for excavation damage characterization and prediction. The paper is based on data collected from a small underground waste-collection site in central Stockholm, Sweden. The data is correlated against rock-mass characteristics and their responses are evaluated. Results indicate potential for excavation damage characterization for all tested techniques, which could minimize blasting damage and improve the over-all tunnel quality.

1 INTRODUCTION

During underground excavation the rock mass is influenced in many ways. The Excavation Damage Zone (EDZ) is a result of these influences. The EDZ can be defined by an irreversible change in rock mass properties, for example increased permeability. EDZ is influenced by the excavation method, the rock mass properties and the in-situ stresses in the area of the tunneling. Since the 1990s research has been done on the EDZ by Emsley et al. (1997), Christiansson et al. (2005), Olsson et al. (2009), Ouchterlony et al. (2009), Ittner et al. (2014), and Siren et al. (2015).

Based on past literature the EDZ have in this paper been separated in the following subzones:

1. Failure Zone or over-break is the area outside the planned tunnel profile, including the Look-Out and hole deviation. This zone consists of fracture networks (both natural and blasting-induced), that caused rock fall-outs. The extent of the over-break can be determined by volumetric scanning and photogrammetry and the half cast factor can be used as a damage indicator.
2. The Damaged Zone is split into three parts and but the quantity and depth of damage is difficult to measure. Although it can be visualized by sawing of rock slices in the tunnel wall (Olsson et al. 2009), diamond coring (DC) or P-wave velocity reduction (Jern 2001):
    a. Inner Damage Zone (Crush Zone): It is located directly around the drill hole, and is caused by the shock-wave energy of the detonation. The micro fractures in the Crush Zone produce a white-wash (rock dust) in the half cast.
    b. Transition Zone: This zone consists of micro fractures that connect and form macro fractures, both radially and parallel to the tunnel wall. The macro fractures are caused by the gas expansion during the detonation of the explosive; it increases the pressure in the (micro) fractures and creates the macro fractures. Seismic reflection, rock slicing and core drilling can be used for the examination.

c. Progressing Zone: In this zone the existing radial fractures propagate due to the increase gas pressure inside these macro fractures. Examination methods are seismic reflection, rock slicing and core drilling. In addition, the Peak Particle Velocity (vibrations) could be used to estimate the depth (Tesarik et al. 2011).

3. Stress Damage Zone consists of rock damage caused by the redistribution of stresses. Fractures occur when the local stress exceeds the rock strength. These fractures are caused by slip-slide or tear events. The events are identified by acoustic emission (AE) and are located deeper in the rock mass.

For Swedish rock construction works, the blasting damage of the rock mass is estimated solely by the charge concentration and provides an estimation of the length of the longest fracture (AMA 13 2014). As a result of this the influence of the geology and other rock conditions are not used to alter blasting plans in order to reduce blasting damage to the rock mass.

Direct measurement of blasting damage is often difficult, slow and costly. That being said, for a construction site, methods with limited disturbance to production activities are favorable and sometimes a prerequisite for any field measurement. Ground Penetrating Radar (GPR) is a Non Destructive Testing (NDT) technique that causes limited disturbances for tunneling production. GPR technology uses high frequency waves (up to 2500 MHz) that are transmitted into rock masses or sediments. Based on the reflection of waves in the rock mass, it can indicate anomalies at a certain depth. It can be used to find macro fractures, rock contacts or fracture zones. Large fractures will be located by reflection, while micro fractures will be estimated based on the loss of energy (dispersion) (Silvast and Wiljanen 2008; Heikkinen et al. 2010). Diamond Core (DC) drilling technology is the use of hollow drill bits to extract cores from the rock mass. The DC technique is widely used in exploration in the mining industry and site pre-investigation in construction. The method normally uses a large rig to drill the core, where in this case a small flexible drill was mounted on the wall as shown in Figure 2. The intention with diamond coring was to visually identify fractures and with P-wave measurements to identify micro fractures (Jern 2001). Jern's (2001) study shows a quadratic correlation of the elastic modulus of a rock mass and the P-wave velocity reduction. The method itself will give partial observations of the fracture patterns compared with cutting slots.

Measure While Drilling (MWD) is a technique that records drill parameters during production drilling. The data consists of machine parameters that have to be analyzed to provide rock properties. The drill parameters include operational pressures (such as feed force and rotation speed) and response parameters (such as penetration rate and rotary torque). In the 1970s and 1980s the information extraction and application of MWD was low. In the second half of the 1990s MWD recording as used today were introduced and tested (Schunnesson 1996, 1998). At the same time computer programs were introduced in order to process and visualize the data. In the 2000s MWD logging has made tremendous steps forward and was used for numerous construction projects in Scandinavia (Hjelme 2010; Martinsson 2010; Rødseth 2013; Valli et al. 2010; Schunnesson et al. 2012; Høien and Nilsen 2014). The high data resolution (2-10cm intervals), low data cost, low data risk and undisturbed production are the benefits of MWD. The drawbacks are the need for data processing and the manual interpretation of the drill logs (Schunnesson 1996, 1998). In general, the data processing removes operator and machine influences from the logged information. This is done by normalization, validation and calibration to the rock properties of the MWD data. The measured parameters can be used to calculate indirect rock parameters as "hardness" and "fracture frequency" (Martinson 2010). The calculated "hardness" is based on the normalized penetration rate and machine pressures, and does not display the actual rock mass hardness but merrily drillability (Thuro 1997; Tamrock 1999; Zara and Bruland 2013). The "fracture frequency" is calculated based on the fluctuation of the penetration rate, water flow and pressure during the drilling process and is used to identify fractured rock. In some cases heterogeneous rock masses can be faulty identified as fractured for example banded rock sediments or cemented veins (Schunnesson 1996, 1998).

The purpose of this paper is to evaluate alternative techniques for excavation damage characterization with blasting. The methods used are focused on being relatively effective and minimizing production stoppage. The selected techniques are tested on a small tunneling site in Stockholm, Sweden.

2 METHOLOGY

2.1 Site description

In Norra Djurgården, central Stockholm, Sweden, several thousand new apartments are currently under construction. To support these new apartments a garbage collecting system with vacuum pipes from the apartments and to a central collection depot is being constructed in the bedrock below. It consists of a 50 meter access tunnel (8m x 6.5m) followed by 50m x 20m x 12.5m underground gallery as shown in Figure 1. The excavation work was carried out during the spring and summer of 2015. Constraints of the construction were a nearby located (<20m) naphtha storage and also low rock cover in the beginning of the access tunnel. To reduce vibration levels in the urban area and minimize the blast damage, electronic detonators were used.

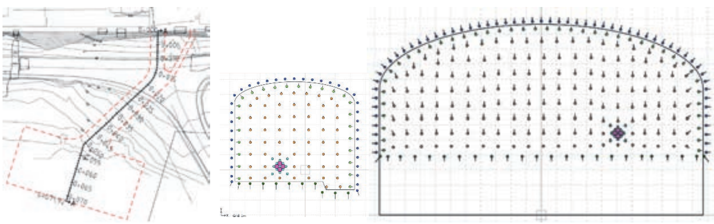

Figure 1 Garbage collection lay-out and drill plans for the tunnel and the gallery (Veidekke2015)

From the pre-investigation, the main rock mass was fine grained granite with locally large grains. In the eastern part of the excavation, gneiss with an E-W oriented foliation was dominating. The two main joint sets strike N-S and NW-SE with 85-90° dip. A flat lying fracture zone also exists in the area. The Rock Mass Rating (RMR) was estimated to be between 60 and 80 (Karlsson 2014).

2.2 Measure While Drilling

The Measure While Drilling (MWD) data collected included the drilling parameters: hole depth [mm], penetration rate [cm/min], feed pressure [bar], percussive pressure [bar], damping pressure [bar], rotation pressure [bar], rotation speed [rpm], water pressure [bar] and water flow [L/min]. The logging interval was set to 2cm for blast holes, probing and grouting holes. The MWD recordings were processed in Atlas Copco's Underground Manager, which calculate the additional parameters relative "hardness" and the relative "fracturing".

2.3 Ground Penetrating Radar

The Ground Penetrating Radar (GPR) equipment consisted of a hand-held sender-receiver unit (1.6 GHz), data recording device and display. The measurements were carried out in an un-shotcreted wall in the gallery, since the shotcrete will reflect the radar signals (Silvast and Wiljanen 2008).

2.4 Core Drilling and Core Analysis

Diamond core (DC) drilling was done with a concrete coring machine placed on a 2 m support beam as shown in Figure 2. The drilling reached a maximal depth of 1.88 m into the walls. The drill cores were geologically and geotechnical logged (Rock Quality Index) (Deere and Deere 1988). The P-wave velocities (primary or pressure wave) were later measured in a laboratory. The dry cores (ø51mm) were analyzed diametrically with water as transfer medium as done by Eitzenberger (2012).

Figure 2 Diamond coring technique employed

3 RESULT AND ANALYSIS

3.1 Measure While Drilling and Diamond Coring

A total number of eight holes for diamond coring were drilled; six were located in the access tunnel and two in the gallery as shown in Figure 3. These locations were selected based on the MWD-data, which was only available in the tunnel and the northern wall of the gallery. The holes targeted different degrees of “hardness” and “fracturing”; from relatively solid (un-fractured) to relatively high-fractured in the MWD interpretation in Atlas Copco’s underground manager. GPR-profile was located in the gallery in the vicinity of the core holes.

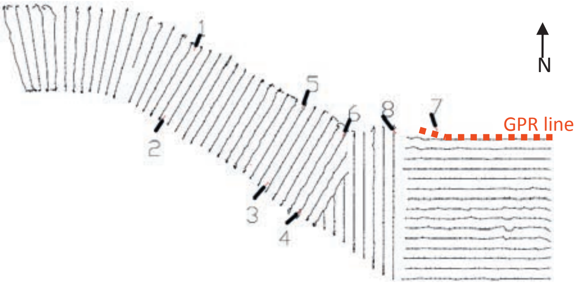

Figure 3 The drill holes, marked 1-8, and GPR line, red and dashed, along the tunnel and in the gallery

In Figure 4 the results from the core holes is summarized. In contrast to the performed pre-investigation the cores show a larger variety of rock types: schist, granite, pegmatite and tonalite. From MWD data the expected rock strength is, medium-hard rock in holes 1-6 and a relatively softer rock in holes 7 and 8. These results however, cannot be conclusively explained by the geological variation. Regarding fracturing, both the MWD based estimation and the core based calculation are presented in the figure.

The calculated parameters “hardness” and “fracturing” are based on MWD data. These are presented for holes 1-6, in Figure 5 and in Figure 6 for hole 7 and hole 8. Comparing the MWD parameter “hardness” with the Drillability index (DRi) for the rock types found on the construction site, show that the values are within the same range, as displayed in Table 1. For example hole 8 contains three different rock types (pegmatite, tonalite and schist) these are not distinguished by the MWD data. However, some differences of the rock types are noted during the tunnel excavation.

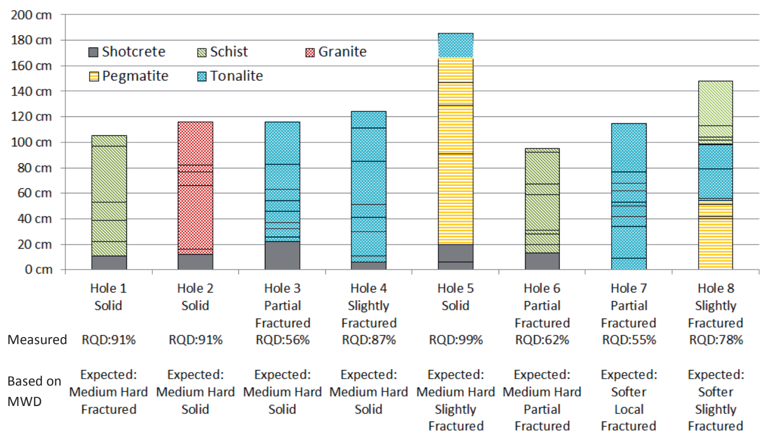

Figure 4 Core samples logged geological and geotechnical. The black horizontal lines are fractures both natural as well as blasting induced

Table 1 Mineralogy of the cores and the Drilling Rate index (DRi) (after Tamrock 1999)

						DRi
Altered Biotite Chlorite Schist	60% Biotite	30% Feldspar	5% Chlorite	2% Serpentine	1% Pyrite	40-70
Tonalite (Granite)	50% Quartz	15% Plagioclase	15% K-feldspar	10% Biotite	10% Mafic vein	30-70 (30-80)
Pegmatite	70% Quartz	15% Plagioclase	10% K-feldspar	0-5% Biotite	0-5% Chlorite	40-80
Veins	50-90% Quartz	10-50% Chlorite	-	-	-	-

Schist is present in hole 1, hole 6 and hole 8, and these locations are distinguished by medium “hardness” in the MWD data and shown in figure 5 as local increased “hardness”. This value is generated by the predominance of quartz, biotite and feldspar in the rock mass, as noted in Table 1. Biotite and feldspar are relatively-soft minerals, where the quartz is relatively hard, when compared to the other minerals found in the area. The pegmatite, encountered in holes 5 and 8, is characterized by MWD data as medium “hardness”, similar to the schist. Although pegmatite has a higher content of quartz than schist does (Table 1), and is therefore theoretically more difficult to drill, this is not shown in the MWD data. The reason for the relatively-high drillability of the pegmatite can be found in the grain size; larger crystals (grains) have a lower yield stress and break easily (Howard and Rowlands 1987). Granite and tonalite are found in holes 2-4 and hole 8. These rock types have a medium “hardness” according to the MWD data; within the same range as the schist and pegmatite.

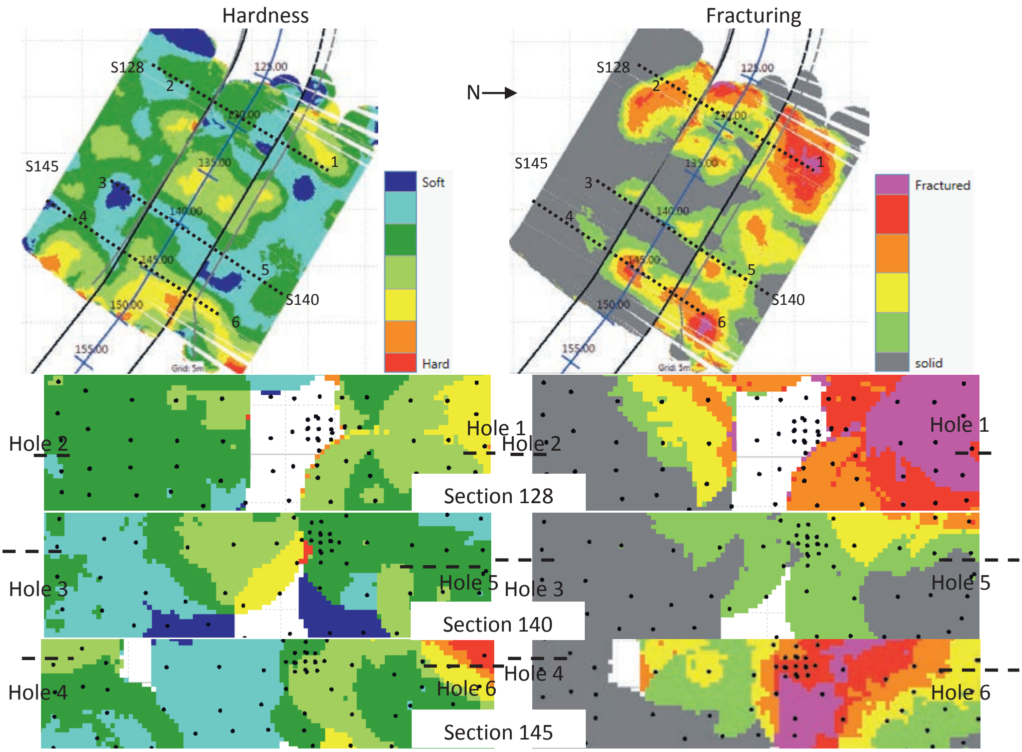

Figure 5 Calculated MWD data from the access tunnel “Hardness” and “Fracturing” (top & side view)

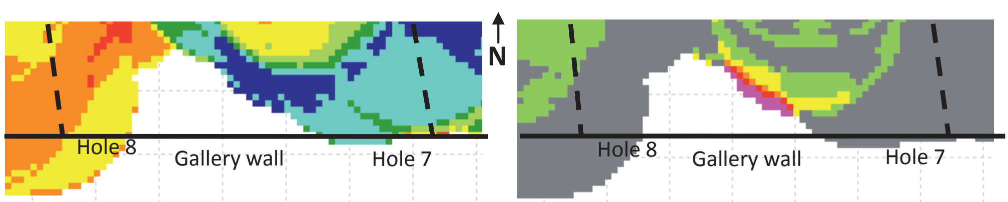

Figure 6 The calculated "hardness" and "fracturing" around hole 7 and hole 8 in the gallery (top view)

The “fracture frequency” calculated from the MWD data in Figure 5 and Figure 6 shows reasonably good correlation with the calculated RQD for hole 2, hole 4 and holes 6-8 as shown in Figure 4. The calculated “fracture frequency” in hole 1, hole 5 and partially hole 8 turned out to be higher (lower RQD) based on the MWD data than core log data. The lower RQD value is presumably affected by the rock structure and texture. The alter Biotite-Chlorite schist in hole 1 and hole 8 is heterogeneous due to many quartz filled veins. The pegmatite in hole 5 and hole 8 has a texture of large phenocrysts. These rock mass properties result in high variation in the drilling parameters. The penetration rate varies caused by the higher hardness of the quartz veins in the schist and phenocrysts in the pegmatite. Due to the nature of MWD data this variation can be perceived as fractured rock mass. The tonalite in hole 3-4 and hole 7 shows a lower RQD value than expected. This may be caused by the blasting of the rock or the sensibility to the drilling technique, rock mass structure, texture or unloading of the rock mass.

It is therefore important to stress the fact that MWD fracturing is based on a statistical calculation over specified length intervals (Schunnesson 1996) that emphasizes continuous physical properties (e.g. schisting) or fracture zones while RQD can be significantly affected by single fractures located at strategic places. Therefore correlation should not always be expected but instead distinguishes the information difference between the methods.

3.2 Measure While Drilling, Ground Penetrating Radar and Diamond Core Drilling

GPR measurements were taken at the northern wall of the gallery as shown in Figure 3. Later, hole 7 and hole 8 were drilled in the vicinity of the GPR profile line. The data was processed in MALÅ GroundVision. In this software several filters were applied to enhance the contrast between the received signal and the background noise. A zone with high dispersion of wave energy has been observed within the top 10cm of the GPR recording, see Figure 7. The dispersion is observed to be caused by blasting induced micro fractures. The dispersion corresponds to the heavily-broken tonalite observed in the first 9 cm of the core sample from hole 7. This observation is not made in hole 8, but a major decrease of P-wave velocity was recorded. Underneath this zone the GPR data shows several reflections in the wall of the gallery, marked with circles in Figure 7. The blue circles are interpreted as major reflections caused by blast-induced fractures. These reflections are observed up to 30 cm into the wall. Local “fracturing” was expected from the MWD and GPR data in hole 7 and hole 8. This interpretation was confirmed by the core sampling, see Figure 4. The deep (>50 cm) and vague reflections in the GPR recordings appear a combination of the natural fractures and rock contacts. For example the green circle in Figure 7 shows a reflection at 70 cm depth, seemingly a single fracture or rock contact. It is interpreted as pre-existing anomaly to the excavation and outside the EDZ, and is not considered to be blasting damage.

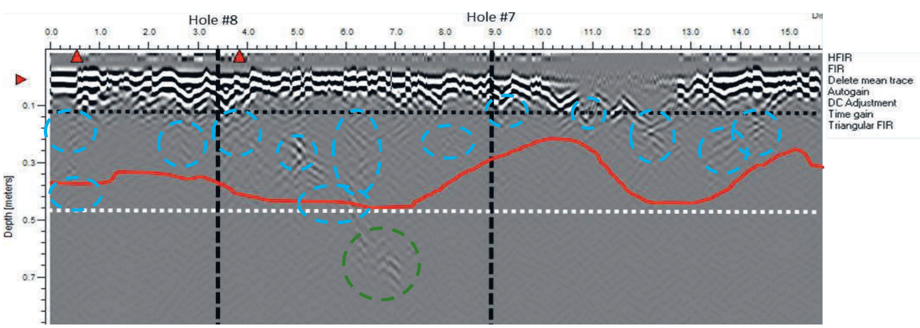

Figure 7 GPR data of the gallery wall and its interpretation

3.3 P-Wave Measurements and Blasting Damage

All cores from the test site were geologically mapped and analyzed in a laboratory environment. The diametric P-wave velocity measurements on the core samples show a velocity range of 5600 to 7300m/s. These wave velocities (<6700m/s) are in compliance with previous tests done by Eitzenberger and Nordlund (2002). Hole 6 and hole 8 show higher velocities, probably caused by the high quartz content.

The P-wave velocities were found to be lower close to the blast surface. Deeper in the rock mass the velocity increases, which is in accordance with observations by Jern (2001). The P-wave velocity reduces at micro fractures; the porosity increase and the wave velocity decreases (IAEA 2002; Jern 2001).

The relative P-wave velocity along the drill holes is compared with the virgin rock conditions in Figure 8. The trend-lines in figure 8 are indicators based on the average P-wave velocities for the virgin rock conditions. Hole 5 and hole 8 are removed from the figure due to their locally increased velocity at 65-100cm depth, Figure 4, caused by the large crystal sizes in the rock types. In the first 50cm the normalized P-wave velocity varies greatly, at depth the velocity curve over all the drill holes stabilizes to virgin rock conditions. Variation at depth (>50cm) are seemly caused by varying lithology and natural fractures. The zoning in figure 8 is determent based on the damage zone limitations and the blast tables.

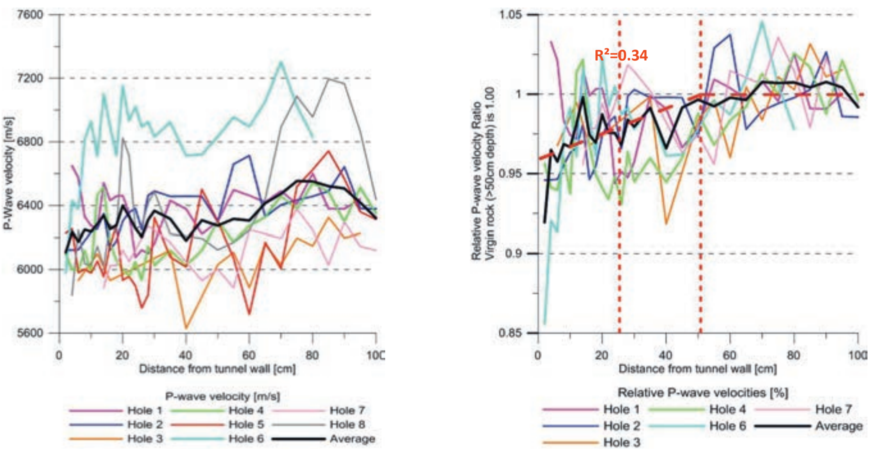

Figure 8 The P-wave velocity and the relative P-wave velocity along the drill holes

4 DISCUSSION AND CONCLUSIONS

From this study it can be concluded that the variation in the pre-investigation data is comparably small in relation to the detailed investigation done in this test based on core drilling, MWD data etc. This raises the question of how much detailed knowledge that is required. However if the intention is to optimize production, round by round, to minimize blast damage by more accurately adapting charging to local geology, the need for a rock mass description with higher resolution, than the existing pre investigation, is obvious.

One such method is MWD, which measures rock drilling properties in every single drill hole. In this case, the “fracturing” parameter clearly indicates anomalies along the tunnel. In this test some of those anomalies are related to hetrogenous rock masses rather than fracturing. Therefore it is important to remember that MWD fracturing is based on a significantly different principle than RQD, and correlation should not always be expected but instead can distinguish the information difference between the methods. In this project the “hardness” parameter shows limited variation, which agrees with the small geological variations and the corresponding rock strength differences. However, it is also here important to stress the fact that MWD provide “hardness” which is related to drillability, similar to the DRi (Tamrock 1999). Therefore any correlation to geology is strictly based on mechanical differences and not geological ones.

Consequently this paper shows the difficulty in identifying either rock type or rock properties based on MWD data. This is problematic when trying to correlate EDZ with the characteristics of the rock mass, mechanically or geologically. In this case study for example the tonalite, pegmatite and schist could not be separated based on MWD data. Therefore, to quantify the MWD-EDZ correlation more research is needed, especially in highly varying rock conditions.

The Diamond Core drilling with wall-mounted rig has shown to be an effective method to extract drill core from the tunnel wall, without interruption of production. Unfortunately the technique is relatively time consuming, roughly 1 hour per meter drilled. The drill cores gave detailed information on the lithology (rock type), the rock mechanical parameters (RQD) and P-wave velocity. In this initial case study the extent of the EDZ could be identified. However, the method does not provide continuous data and therefore may not give the complete picture of the EDZ as in cutting slots. This indicates that more detailed studies are needed for the use of this investigation method for the EDZ.

Ground Penetrating Radar can be used for the investigation of micro and macro fractures in the rock mass without interrupting production. The macro fractures give a refection whereas the micro cracks can be identified through dispersion. This technique can be used in the future for more extensive rock-damage investigations, although the type of reflection and its location need to be calibrated and validated by other methods, for example core drilling.

The P-wave velocity measurements show a damage zone by the reduction of velocity. It can be concluded that this type of measurement can be used to quantify the depth of the EDZ. The method requires intact drill cores and the diametrical wave guides used may affect the P-wave velocity. The use of geological, geophysical and geotechnical methods could provide a frame-work for interpretation of the MWD data on rock-mass properties and conditions as well as create an understanding of the EDZ. The combination of the investigation techniques has a great potential to create a correlation which can be used for the prediction of the extent of the EDZ.

5 ACKNOWLEDGEMENT

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Measurement While Drilling (MWD) technology for blasting damage calculation

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Measurement While Drilling (MWD) technology for blasting damage calculation

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ABSTRACT

For all underground excavations it is important to reduce both blasting induced damage and the blast-induced Excavation Damage Zone (EDZ). Except for the operational parameters, the geomechanical conditions of the rock mass have a significant impact on the amount of over-break in tunnel constructions. Today, direct measurements of the EDZ are difficult to perform and are therefore not commonly done at construction sites. This paper investigates the application of Measurement While Drilling (MWD) technology to predict the extent of the EDZ, using data from four Swedish tunnel excavations. The depth of this zone is determined by Ground Penetration Radar (GPR), P-wave velocity measurements and Rock Quality Designation (RQD) for drill cores. The correlation between MWD, operational parameters and EDZ was evaluated using multiple linear regression. The study shows that the EDZ is heavily influenced by rock mass conditions but also by operational parameters. Furthermore, the EDZ depth based on GPR measurements, can be reasonable well predicted using MWD data.

1 INTRODUCTION

In Scandinavia, tunnels are mainly excavated by drilling and blasting, which is an effective excavations method in hard rock. The blasting however, induces damage to the rock, that can be divided in over-break and the Excavation Damage Zone (EDZ) (Siren, et al. 2015) that both are influenced by geological, design and excavation parameters (Ibarra et al. 1996; Olsson and Ouchterlony 2003; Olsson et al. 2009; Mohammadi et al. 2017). Despite the efforts to reduce the blast-induced damage through optimization of the design and excavation parameters, the EDZ is still inevitable (Ibarra et al. 1996; Ericsson et al. 2015).

A number of observational methods are available to estimate over-break and the EDZ, e.g. theoretical damage tables in project requirements (AMA anläggning 17: allmän material- och arbetsbeskrivning för anläggningsarbeten), Peak Particle Velocity (PPV), operation cycle times, muck tonnages, and Cavity Monitoring Systems (CMS) measurements (Ibarra et al. 1996; Lizotte et al. 1996; Van Eldert 2017). Furthermore, the EDZ macro fractures can be measured in rock mass slices (Olsson et al. 2009), through drill cores or with borehole cameras (Van Eldert et al. 2016). Micro fractures can be determined with the dispersion of GPR wave energy (Ericsson et al. 2015) and by measuring the P-wave velocity decrease (Eitzenberger 2012). A method to estimate the damage zone, defined by the longest fracture, has been presented by (Olsson and Ouchterlony 2003, and Olsson et al. 2009). This method takes into account the explosive properties, operational factors, e.g. explosive type and diameter, hole diameter, initiation system, hole spacing and water in the drill holes and to some extent the rock mass properties, e.g. P-wave velocity, fracture toughness and natural fracturing.

In recent times, data-driven techniques have been used in tunnelling projects, to estimate rock mass quality and altering the design and excavation parameters depending on the rock mass conditions. These techniques include charge logging (Ericsson et al. 2015) and Measurement While Drilling (MWD) technology (Schunnesson et al. 2011, Van

Eldert et al. 2016). MWD technology means that a number of parameters (hole depth, penetration rate, feed pressure, percussion pressure, rotation pressure, flushing pressure, rotation speed and flushing flow) are measured at defined sampling intervals along each production bore hole. Each data point represents a finger print of the rock mass and can be sampled at intervals down to 2 cm (Van Eldert et al. 2017). The measured parameters can also be used to calculate parameters such as Hardness or Fracturing, or "Rock Quality" (Schunnesson 1996, 1998; Van Eldert et al. 2018).

In this paper the possibility to used MWD to characterize the rock mass conditions and to provide an indication of the extent of blasting damage, are presented. The study is based on three different excavation sites in central Sweden.

2 METHODOLOGY

The sites used in this study were: an access tunnel to an underground garbage collection depot in central Stockholm; two ramp tunnels to the Stockholm bypass; and an experimental tunnel at Äspö Hard Rock Laboratory (HRL). At these sites, MWD data, GPR data and Drill Cores (DC) were collected. The latter were analysed for RQD and the diametrical P-wave velocity to estimate the extent of the EDZ. Multiple linear regressions were used to try to correlate the MWD parameters with the acquired EDZ depth.

2.1 Measurement While Drilling

The MWD data were filtered and normalized, as discussed by Schunnesson (1996, 1998), using the supplier's software packages (Sandvik's iSure V7.0 and Atlas Copco's Underground Manager (UM) V1.6) and Matlab code. Variations in the Hardness and Fracturing indicators were used to select the locations from where core drilling was done, in the Access Tunnel and the two ramp tunnels in the Stockholm bypass. In the TAS04 tunnel, the drill cores were drilled at approximately 3 m spacing along the tunnel walls. Based on the known location of the DC, the MWD values were determined.

2.2 Field Data Collection

All tunnel walls where manually mapped for rock types and fractures.

Drilling of ø48-51 mm, horizontal core holes into the tunnel walls were done to characterise the blast damage. Figure 1 show the collaring locations of all 49 drill cores in all four tunnels. Eight drill cores were drilled at the garbage collection depot (Figure 1A), six in the access tunnel and two in the cavern, eight in ramp tunnel 214 (Figure 1B) and thirteen in ramp tunnel 213 (Figure 1C) at the Stockholm bypass and twenty in the Äspö HRL TAS04 tunnel (Figure 1D). All drill cores were logged for rock type and RQD.

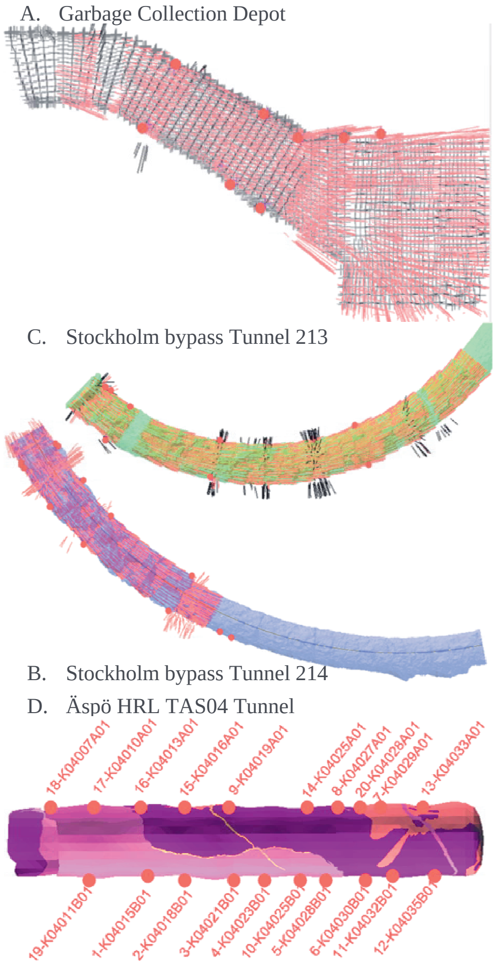

Figure 1 Location of diamond core holes (red dots) in the four tunnels.

The P-wave velocity was measured every 2 cm along all drill core and then plotted versus the core length. Close to the tunnel surface the velocity is lower but increases with increasing distance from the tunnel surface. The distance, at which the velocity reaches the rock mass natural (in-situ) P-wave velocity (threshold), is interpreted as the extent of the EDZ.

GPR was measured, with Malå GS equipment and 1.6 GHz antennas, along 34 lines on the accessible tunnel walls at 2 cm measurement intervals.

During the tunnel mapping attempt was made to differentiate between natural fractures and blast induced ones. By comparing mapped fractures or fracture zones with GPR data, locations with both mapped fractures on the tunnel wall and clear GPR fracture response was considered as natural fractures. If the only appear in the GPR data they were considered to be blast induced (Karlsson 2015, ÅF 2016, Van Eldert 2017). The extent and depth of these blast-induced fractures was used as an indication of the depth of the EDZ.

2.3 *Multiple Linear Regression*

Multiple Linear Regressions were used to investigate the correlation between the collected field data and the selected MWD parameters. The aim was to predict the EDZ depth based on charge concentration, penetration rate, feed pressure, rotation speed, water flow rotation pressure, rock cover (tunnel depth), tunnel area and contour hole spacing, as well as the Atlas Copco Hardness and Fracturing indicators. F-statistics were used to quantify the correlation between the used parameters. The impact of the blast initiation system, electronic or pyrotechnical, was considered but could not be numerically quantified.

3 TEST SITES

3.1 *Veidekke Garbage Collection Depot Access Tunnel*

The Veidekke Access Tunnel is a 50 m long, 60 to 76 m², tunnel connected to a cavern for underground garbage collection for household waste in Norra Djurgården in Stockholm, Sweden. Figure 2 shows the layout of the construction (Karlsson 2014). The rock mass mainly consists of fine-grained granite and gneiss. An Atlas Copco XE3 rig was used for the excavation, drilling ø48mm drill holes at an average specific drilling of 1.60 m/m³. The contour holes were spaced 45-50 cm apart, and were charged with emulsion 0.350 kg/m string charge with 0.4 kg bottom charge (Orica Civec). The rounds were initiated with an electronic blasting system (Orica eDev2).

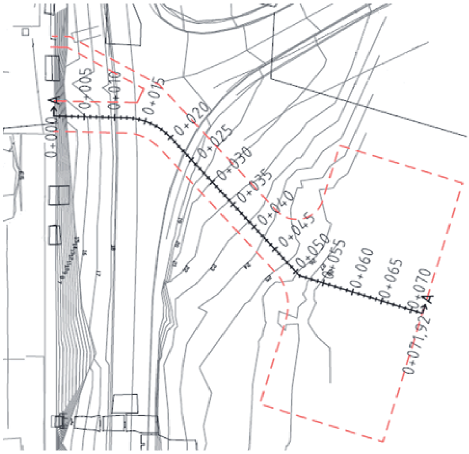

Figure 2 Access Tunnel and underground garbage collection depot (Karlsson 2014).

3.2 *Subterra Stockholm Bypass Tunnels*

The Stockholm Bypass consists of 21 km of new roads, of which 18 km will be located underground as tunnels (Norberg, Markstedt & Thörnqvist2005). The construction of the first access and ramp tunnels started in 2015 in Skärholmen in Stockholm, see Figure 3.

The rock mass is mainly a gray, medium to large-grained gneiss with intrusions of granite and pegmatite. Some zones of weaker material, e.g. graphite and weathered rock, were also observed in the site investigation (Arghe 2016). The 97-119 m² tunnels was excavated with an Atlas Copco WE3 rig, drilling ø48 mm drill holes with a specific drilling of 1.44 m/m³. The contour holes were spaced 50-90 cm apart and charged with 0.350 kg/m string emulsion and 0.4 kg bottom charge (Forcit Kemiiti 810). In this case, pyrotechnical detonators were used (Austin Powder).

Figure 3 Stockholm bypass and Tunnel 213 and Tunnel 214 (Illustrations courtesy of Trafikverket.

3.3 SKB TAS04 at the Äspö HRL

In 2012 several new tunnels were excavated at the 410 meter level at Äspö Hard Rock Laboratory (HRL), the underground research facility of the Swedish Nuclear Fuel and Waste Management Co. (SKB) close to Oskarshamn, see Figure 4. The excavation was performed as a showcase for best-practices in Drill and Blast tunnelling. Therefore, it took place with great care, quality assurance and quality control (Ericsson et al. 2015).

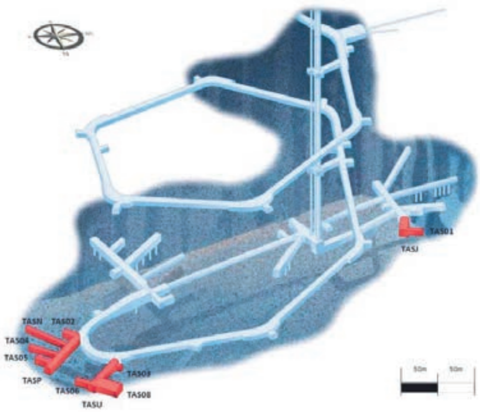

Figure 4 Äspö HRL (Courtesy of SKB).

A new Sandvik DT920i rig drilled the rounds with ø48 mm drill holes. The average specific drilling in the eight rounds was 4.04 m/m³. This 36 m long and 19.7 m² tunnel has mainly bin excavated in fine-grained granite, diorite and granodiorite (Ericsson et al. 2015). The contour holes were spaced 40-50 cm apart and were charged with a 0.350 kg/m string emulsion and 0.5 kg bottom charge (Forcit Kemiiti 810). The blast initiation was performed with an electronic blasting system (Orica i-kon VS).

4 RESULTS

4.1 Field Data

When drilling the rounds in the tunnels the used rigs are navigated and all recorded MWD data are aligned with the tunnel line coordinates. The GPR measurements on the other hand where recorded along profiles on the tunnel walls. In order to calibrate the GPR data with the detailed located MWD data, the bottom charge was used as an indicator. This was possible, since the bottom charge had a significant higher specific charge and was clearly visible in the GPR measurements (see Figure 5).

The dotted line in Figure 5 shows the estimated blast damage at the first part of the left wall in the TAS04 tunnel. The depth of the damage zone is at the string charge (blast hole pipe) between 8 and 12 cm. At the bottom charge, at the end of the drill holes, the GPR reflections goes much deeper into the rock mass (30 to 40 cm), indicating a more extensive damage zone.

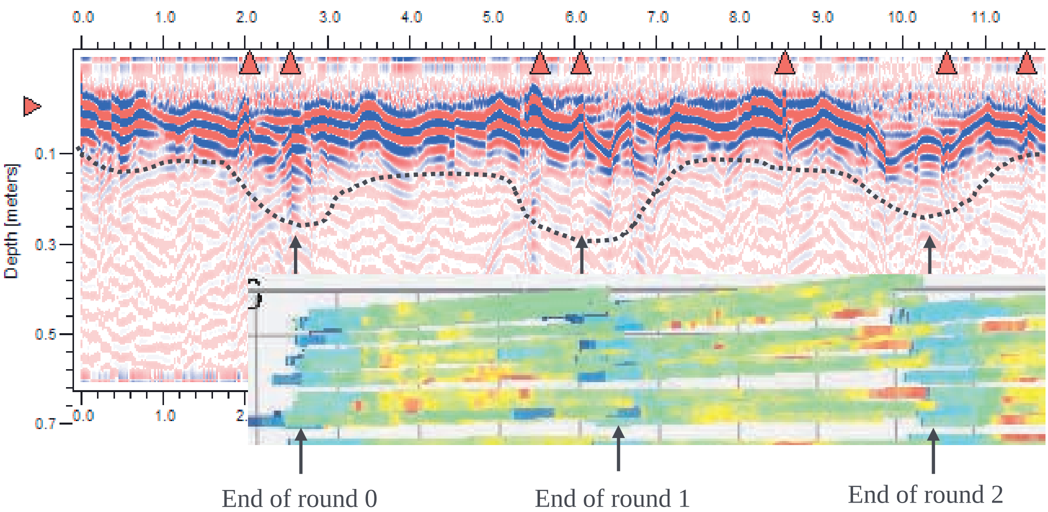

Figure 5 GPR- and drilling data at the left wall of TAS04 tunnel

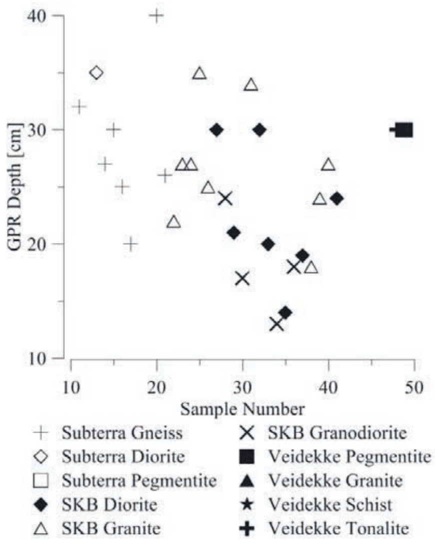

Figure 6 GPR depth measured at the collaring location of all 49 core holes in all three sites. The different rock types for all responses are also presented.

In Figure 6 the depth of the GPR reflection at the location of all 49 drill cores, are presented. The depth varies from 12 cm up to over 40 cm into the tunnel wall.

The extracted drill cores were drilled in rock types ranging from large phenocryst pegmatite to fine-grained granite and foliated gneiss, see Figure 7. The length of the drill cores was from 40 cm up to a maximum length of 168 cm. RQD was determined for all cores, and the RQD ranges from 0% (natural crushed rock) to 98%, see Figure 8. The black separation lines with in the bars in Figure 7 display fractures observed during the core logging. In general a decreasing fracture density along the core was registered. The closer to the tunnel wall the more fractured the rock, see Figure 7.

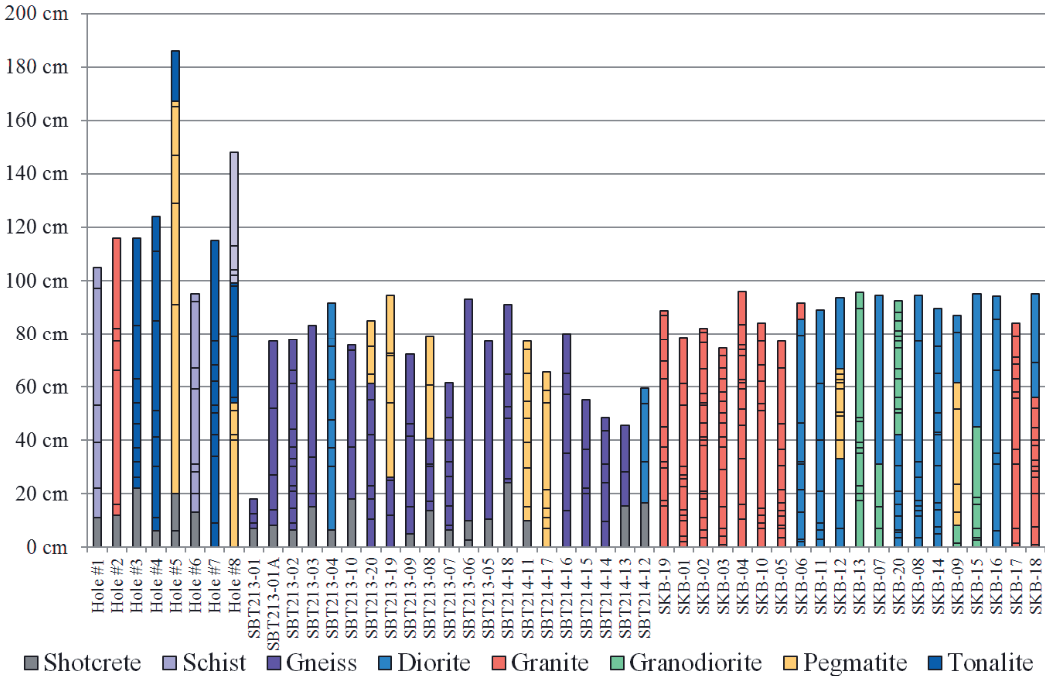

Figure 7 **The collected diamond core samples at the garbage collection depot (8), Tunnel 213 (13), Tunnel 214 (8) and TAS04 tunnel (20). The black separation lines with in the bars indicated observed fractures.**

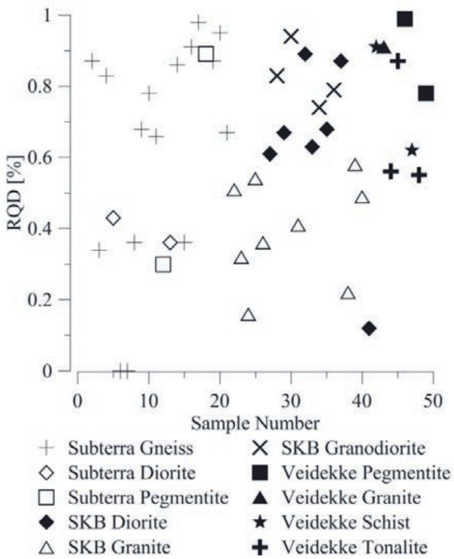

Figure 8 **RQD of different rock types at the three sites measured in the 49 drill cores.**

4.2 P-wave measurements

Diametric P-wave measurements were done for all cores with enough quality. Some parts of the cores were heavily fractured and could not be measured. According to the theory the P-wave will be lower close to the tunnel wall where the rock has been affected by blasting. With increased distance from the tunnel wall the rock mass conditions will be more and more undisturbed resulting in a higher P-wave velocity. The distance from the tunnel wall to the transition point where the P-wave velocity is no longer affected by the excavation, is defined as the estimated EDZ.

In Figure 9 the average P-wave velocity for each excavation site is presented. For the Stockholm by-pass tunnels the trend above is clearly seen, and the distance from the tunnel wall where undisturbed rock is reached is around 20 cm. For the garbage collection despot, the same EDZ depth is seen even through the trend is not as clear as for the Stockholm by-pass. For the Äspö TAS04 tunnel the trend is not seen, which may depend on the exceptional care taken during excavation.

In Figure 10 the P-wave velocity based EDZ depth from all 49 investigated core are presented. The individual values have a significant variation from 2 cm up to 46 cm, which indicates that the influencing background condition vary significantly.

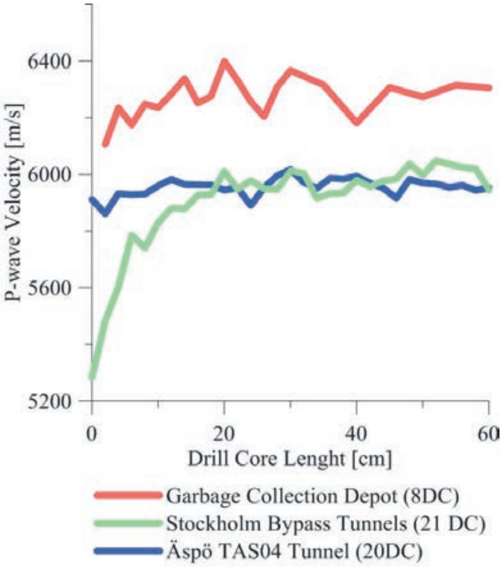

Figure 9 Average P-wave velocities in the drill cores.

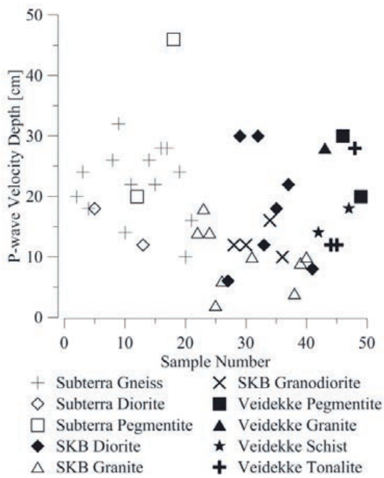

Figure 10 Depth of the EDZ based on the P-wave velocities of different rock types at the three sites measured in the 49 DC.

4.3 Multiple Linear Regression

The correlation between the recorded MWD data and measured extent of the blasting damage was investigated with Multiple Linear Regression. The aim was to predict the extent of the damage zone based on the MWD parameters. In addition to the MWD parameter, design parameters, i.e. planned contour charges for the collar (0.0 kg/m), pipe (0.35 kg/m) and bottom (1.2 kg/m), rock cover, cross section area and the contour spacing of each tunnel section was also included, see Table 1.

The GPR blasting damage depth shows a relative good correlation with both raw MWD data and the design parameters (R^2: 0.673), displayed in Table 1. The most significant parameters based on the p-Value (<5%) are 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 2, shows also a good correlation between the GPR blasting depth, the MWD and design parameters (R^2: 0.578), although the significance of the input parameters is rather low (p-Value>5%).

The P-wave velocity damage depth shows a significantly lower correlation with the input parameters than for the GPR based damage depth, for both the raw MWD data (R^2: 0.363, Table 1) and the calculated MWD parameters (R^2: 0.107, Table 2). The statistical model also failed to identify significant input parameters for the correlation with the P-wave velocity damage depth.

The RQD along the drill hole displays a medium correlation for both the raw MWD data (R^2: 0.338, Table 1) and the calculated MWD parameters (R^2: 0.359, Table 2). The p-values show a high significance of the design parameters, i.e. tunnel cross section (1.4% and 4.0%), rock cover (1.5% and 4.7%) and charge concentration (4.8% and 3.0%), and for the feed pressure for the raw MWD parameters (4.8%), as is displayed in Table 1 and Table 2.

Table 1 Multiple Linear Regression, the factors, and significance of the input parameters (Raw MWD parameters).

	GPR	R^2: 0.673	P-wave decrease	R^2: 0.363	RQD	R^2: 0.338
Raw MWD	Factor	p-Value	Factor	p-Value	Factor	p-Value
(Intercept)	-15.022	0.427	-13.476	0.543	-0.450	0.465
Charge Concentration	7.111	**0.007**	-0.584	0.870	-0.212	**0.048**
Penetration rate	-0.044	0.063	0.020	0.498	-0.001	0.180
Feed Pressure	0.220	0.057	-0.054	0.721	0.008	**0.048**
Rotation speed	0.135	**0.025**	0.064	0.300	0.001	0.625
Water Flow	0.155	**0.004**	-0.019	0.792	0.003	0.112
Rotation Pressure	-0.091	0.425	-0.029	0.871	-0.001	0.908
Rock Cover	-0.021	**0.016**	0.019	0.160	0.001	**0.015**
Tunnel Cross section	-0.070	**0.031**	0.089	0.071	0.004	**0.014**
Contour Spacing	-2.932	0.836	11.843	0.446	-0.177	0.706

Table 2 Multiple Linear Regression, the factors, and significance of the input parameters (Calculated MWD parameters).

	GPR	R^2: 0.578	P-wave decrease	R^2: 0.107	RQD	R^2: 0.359
Calculated MWD	Factor	p-Value	Factor	p-Value	Factor	p-Value
(Intercept)	82.491	0.105	-10.229	0.714	-0.429	0.605
Charge Concentration	16.792	0.286	-3.143	0.681	-0.440	**0.030**
Hardness Indication	-0.804	0.442	0.177	0.850	-0.011	0.685
Fracturing Indication	-0.018	0.997	1.877	0.595	0.018	0.863
Rock Cover	-0.095	0.262	0.049	0.315	0.003	**0.047**
Tunnel Cross section	-0.348	0.256	0.169	0.294	0.010	**0.040**
Contour Spacing	-13.288	0.643	13.674	0.532	0.044	0.946

5 DISCUSSION

The measured blast damage zone based on GPR data, P-wave velocity and the RQD show a large variation. However, the data still show that the blast damage are significantly influenced by the charge concentration (e.g. Figure 5, more extensive damage at the bottom charge) and the contour hole spacing see Table 1 and Table 2. This is in agreement with studies previously conducted by Olsson and Ouchterlony (2003) and Ouchterlony et al. (2009). The study further indicates an effect of the blast damage zone by the initiation system and the contour hole spacing, in particular the P-wave velocity (Figure 9, Table 1 and Table 2). The extended damage zone based on the P-wave velocity for the Stockholm bypass tunnels (Figure 9) may be explained by the use of pyrotechnical detonators instead of electronical detonators that was used in the contour holes at the other tunnel sites, giving less extensive blast damage. This is also in line with observations made by Olsson and Ouchterlony (2003) and Ouchterlony et al. (2009).

The collected blast damage data may also indicate an effects of rock mass texture (grain size) on the extent of the blast damage. The large grained pegmatite shows less macro fractural damage (RQD) but more extensive micro fracture damage (P-wave velocity decrease). The fine grain granite on the other hand sustained more macro fractural damage (lower RQD) but less micro fracture damage (P-wave velocity decrease), see Figure 8 and Figure 10. These differences might be explained by the required fracturing force in grains and on grain boundaries (Howarth and Rowlands 1987).

The statistical analysis showed that there are correlations between MWD and the measured excavation damage, especially indicated with the GPR. Here the coefficient of determination was 67.3% for the raw MWD parameters and 57.8% for the calculated MWD parameters. For the other methods, medium correlations were found (Table 1 and Table 2).

The variation in the measurements of the excavation damage could be explained by the design parameters (i.e. charge concentration, contour hole spacing, rock cover and tunnel cross section), see Table 1 and Table 2. These findings are similar with earlier studies conducted on over-break (Mohammadi et al. 2017) and the EDZ (Olsson and Ouchterlony 2003, Ouchterlony et al. 2009).

The suggested approach to use MWD data and design parameters to predict blast damage in underground excavations has an identified potential. The effects of other parameters such as initiation (Olsson and Ouchterlony 2003, Ouchterlony et al. 2009), fracture toughness and rock mass texture (Howarth and Rowlands 1987) has not been studied within the project and is still not included in the current prediction model. Further studies to include these effects are suggested.

6 CONCLUSION

The study shows that the extent of EDZ, measured by GPR, P-wave velocity and RQD, is influenced by the properties of the rock mass and excavation parameters. The study also shows the effects of initiation on the measured blast damage. Furthermore, multiple linear regression show that the combination of MWD and design parameters can describe the extent of the GPR based blast damage quite well, and to a lower extend for the blast damage based on P-wave velocity and RQD.

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