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Rock Support by Rock Mass Classification Systems for TBM Tunnelling

In Upper Kon Tum, Vietnam

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Abstract

The purpose of this thesis is to study the difference in the use of rock support measures in tunnels excavated by different methods. The methods examined are excavation by Tunnel Boring Machine (TBM) and excavation by drill and blast. The aim of the study has been to address whether the smooth contour that excavation by TBM provides, leads to reduced amounts of rock support.

Field work has been carried out to examine the possible relation between the method of excavation and rock support requirements. The field work was conducted at Upper Kon Tum Hydroelectric Power Project (HPP) in Vietnam. Here has 650 meters of a headrace tunnel, excavated by TBM, been geologically mapped. At the outlet of the headrace tunnel, a short portal tunnel was excavated by drill and blast. 105 meters of this portal tunnel was also mapped to be able to compare the need for rock support in the two types of tunnels.

The mapped rock mass has been classified according to established rock mass classification systems. The Rock Quality Index (Q-system) and the Rock Mass Rating (RMR-system) has been used for this purpose. Estimations of both type and quantity of rock support based on the classification systems are given, and has been compared to already installed rock support in the tunnels. In addition, estimations regarding costs of rock support has been done.

In total 755 meters of the tunnel at Upper Kon Tum HPP were mapped with respect to geology. The rock mass is characterized as hard and competent, and it is regularly intersected by zones of weakness. The collected data basis consists mainly of rock mass in the classes of fair to extremely good quality, according to the Q-system, and the rock mass classes I and II, according to the RMR-system.

For rock mass of quality between fair and extremely good, it has been found that only rock bolting is necessary as rock support. The need for rock bolting is reduced by a number of 266 rock bolts when the tunnel is excavated by TBM compared to drill and blast. 555 meters of the headrace tunnel, was within these rock mass classes. The reduced number of bolts corresponds to a saving of around 360 NOK per meter tunnel excavated.

The conclusive remarks are that the amount of rock support is reduced in tunnels excavated by tunnel boring machines compared to tunnels excavated by drill and blast in the form of rock bolts. For other kinds of rock support, no relation to the method of excavation has been found. However, the basis of data is not strong enough to make up a final conclusion on this matter. It is, nevertheless, likely that further studies will be able to confirm this indication.

Sammendrag

Hensikten med denne masteroppgaven er å undersøke forskjeller i bruken av bergsikringstiltak i tunneler konstruert med ulike drivemetoder. Drivemetodene som undersøkes er fullprofilboring med tunnelboremaskin (TBM) og konvensjonell boring og sprengnings arbeid. Målet med studiet er å undersøke hvorvidt den jevne konturen man oppnår med TBM fører til redusert omfang av bergsikring.

Feltarbeid har blitt utført for å studere den mulige relasjonen mellom drivemetode og krav som stilles til bergsikringstiltak. Feltarbeidet har blitt utført ved Øvre Kon Tum Vannkraftverk i Vietnam. Her har 650 meter av tilløpstunnelen, som er bygd med TBM, blitt geologisk kartlagt. Ved utløpet av tilløpstunnelen har en kort portaltunnel blitt sprengt ut med konvensjonell boring og sprengning. 105 meter av denne portaltunnelen har også blitt kartlagt for å kunne sammenlikne behovet for bergsikring i de to ulike tunneltypene.

Den kartlagte bergmassen har blitt klassifisert i henhold til etablerte klassifiseringssystemer. The Rock Quality Index (Q-systemet) og Rock Mass Rating (RMR-systemet) har blitt brukt i denne sammenheng. Estimerte mengder og type bergsikring basert på disse klassifiserings-systemene er angitt, og har også blitt sammenliknet med allerede installert bergsikring i tunnelene. I tillegg er det angitt estimerte kostander for de anbefalte bergsikringstiltakene.

Totalt har 755 meter av tunnelen ved Øvre Kon Tum Vannkraftverk blitt kartlagt med hensyn til geologi. Bergmassen karakteriseres av at den er hard og kompetent, og at den jevnlig krysses av svakhetssoner. Den innsamlede informasjonen består i hovedsak av bergmasse i bergmasseklassene middels til ekstremt god, etter Q-systemet, og bergmasseklasse I og II, etter RMR-systemet.

For bergmasse med kvalitet fra middels til ekstremt god, har det funnet at kun bergbolting er nødvendig som sikring. Behovet for bergbolting er redusert med en mengde på 266 bolter, når tunnelen er konstruert med TBM sammenliknet med boring og sprengning. 555 meter av tilløpstunnelen, var innenfor disse bergmasseklassene. Det reduserte antallet bolter tilsvarer en kostnads-reduksjon på rundt 360 NOK per meter tunnel.

Til konklusjon angis følgende: Det er redusert behov for bergsikring i tunneler drevet med tunnelboremaskiner sammenliknet med konvensjonell boring og sprengning i form av bergbolter. Angående behovet for andre typer bergsikring er det ikke blitt funnet noen sammenhenger med drivemetoden. Datagrunnlaget er ikke stort nok til å kunne konkludere på dette punktet, men det er trolig at videre undersøkelser vil kunne bekrefte denne indikasjonen.

Preface

This master thesis is written at the Department of Civil and Environmental Engineering at the Norwegian University of Science and Technology (NTNU). The work reported in this thesis is rewarded by 30 credits which fulfills the requirements of the degree Master of Science (MSc).

The main supervisor for this thesis has been Associate Professor Pål Drevland Jakobsen at the Department of Civil and Environmental Engineering at NTNU.

The thesis will deal with the topic of rock support in tunnels in hard rock mass. The difference in rock support for tunnels excavated by conventional drill and blast and by Tunnel Boring Machine (TBM) is examined. To perform this assessment, field work at a tunneling project in Upper Kon Tum, Vietnam has been done. Established rock mass classification systems has been used for the purpose of deciding quantity and type of rock support.

I would like to express my gratitude to my supervisor, Pål Drevland Jakobsen, for giving me exceptional guidance during the work, to Sindre Log and The Robbins Company for arranging the field work and for providing valuable information, to Nghia Trinh at SINTEF for helping with accommodation in Vietnam, to the crew at Upper Kon Tum Hydroelectric Power Project for their assistance and support during the field work and to Egil Johansen for great cooperation regarding the field work.

The work with this thesis has given me valuable insight within the field. I look forward with excitement to be able to contribute in the field of construction and underground engineering for many years to come.

Trondheim, 7.6.2018

Ola Hobbelstad

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Definitions and abbreviations

ASTM	American Society for Testing and Materials
ESR	Excavation Support Ratio
HPP	Hydroelectric power project
ISRM	International Society for Rock Mechanics
NATM	New Austrian Tunneling Method
NGI	Norwegian Geotechnical Institute
NMT	Norwegian Method of Tunneling
NPRA	Norwegian Public Roads Administration
Q	Rock Mass Quality Index
RMR	Rock Mass Rating
RQD	Rock Quality Designation
RRS	Reinforced Ribs of Sprayed concrete
TBM	Tunnel Boring Machine
UCS	Uniaxial Compressive Strength

1 Introduction

Beneath is the topic introduced, as well as the background for this thesis. Then research questions are raised in the light of a hypothesis which make up the framework of this report. Goals and limitations for the work are stated before a short readers guidance ends the chapter.

1.1 Background

It exists several systems for rock mass classification which can be used to decide type and quantity of rock support. These systems are in general based on empirical data from a large number of tunnel excavations (Hoek, 2006). This way to generalize findings to estimate or recommend an amount of rock support for future underground excavations has been debated throughout the fields of construction and geological engineering.

One opinion is that rock mass classification systems is of great help during the feasibility studies of a project, but one should be careful implementing these systems in the construction phase of a project. In the early stages, it may be used to estimate type and amount of rock support based on information found during the preinvestigation stages. Then estimates about costs and time consumption for the project can be made. Palmstrøm and Broch (2006), Bieniawski (1997) and Hoek and Brown (1980) all agrees that rock mass classification systems is to be used during the preliminary stages of a project.

Palmstrøm and Broch (2006) emphasizes that rock mass classification systems should not be used to decide amount and type of rock support. It should not replace the judgement of an experienced tunneling crew. However, they realize that such systems are a good indicator for the necessary amounts of rock support, and that it is also a good way of documenting rock mass characteristics and the installed rock support in a tunneling project.

On the other hand, in an report complied at Norwegian Geotechnical Institute (NGI), Løset (1997) states that the Rock Mass Quality Index (the Q-system) is based on more than 1000 project records regarding permanent rock support, and that it is made to be used for choosing rock support during construction as well as to provide estimates in the preliminary stages.

This difference of opinions seemed interesting to examine more closely.

When it comes to excavation of tunnels, it is two main methods to do so. Either it can be done by the conventional drill and blast method, or it can be done by a tunnel boring machine (TBM). Jakobsen et al. (2015) argues that the need for rock support is significantly less in tunnel excavated by TBM than in tunnels excavated by drill and blast. Based on a series of case records from Norwegian tunneling projects it was found that the reduction in support material is between 40 and 90 % when the tunnel is excavated by TBM compared to drill and blast. There is however a lot more considerations to take than just rock support when one is to choose the method of excavation for a project. Macias and Bruland (2014) conducted a study on this topic and concluded on 14 factors which are crucial for the choice of tunneling method. Rock support influences several of these factors.

The remarkable difference in consumption of rock support material between the two tunneling methods will also be a topic for further examination in this maters thesis.

To study these topics a tunneling project in Upper Kon Tum, Vietnam will be examined. The tunnel in question is a headrace tunnel under construction for a hydroelectric power plant. The project is part of the large ongoing development of hydroelectric power plants in Vietnam the recent decades (Trinh et al., 2015). The main tunnel is excavated by TBM, while a portal tunnel, which is about 200 meter long, is excavated by drill and blast.

Among the rock mass classification systems present today, the Q-system (Barton et al., 1974) and the RMR-system (Bieniawski, 1989) are the two most widely used according to Evert Hoek (2006). These two classification systems are also picked out by the owner of the Hydroelectric Power Project (HPP) in Upper Kon Tum, Vietnam to apply when classifying the present rock mass. Based on this the two classification systems which will be used to classify rock mass and to detect the need of rock support measures for this thesis are the Q-system and the RMR-system. More information about rock mass classification and the said systems are given in chapter 2.3.

1.2 Research questions and hypothesis

The thesis is based on one main hypothesis. The whole thesis is a basis for concluding upon this hypothesis in the best way possible. The hypothesis is stated as below:

"The amount of rock support is reduced in tunnels excavated by tunnel boring machines compared to tunnels excavated by drill and blast"

This means that the topic of rock support needs to be addressed, as well as existing methods of deciding the need for and the amount of rock support. Information about these topics is given in chapter 2.2 and 2.3. In addition, a basis of data is needed to examine how the method of excavation affects the need for rock support. This will be provided through geological mapping at Upper Kon Tum HPP.

In the light of the hypothesis several research questions arise. In this thesis, the aim will be to answer the two following research questions:

1. Can the effect of a better rock contour achieved with mechanized tunneling be related to reduced rock support?
2. How much can the amount of rock support and the costs be reduced in fair to extremely good rock mass conditions?

1.3 Goals and limitations

To be able to answer the research questions stated above, it will be necessary to master suitable rock mass classification systems.

The goal is to become comfortable with these classification systems while conducting the geological mapping. Data collected in the geological mapping will be displayed through the implementation of the classification systems. This means that it is crucial that the systems are used in a most possible correct fashion.

This thesis is limited to the use of two rock mass classification systems. These are the Q-system and the RMR-system. The reason for this limitation is to be able to gather in depth skills in the use of these systems. A great deal of literature exists about the two systems and about the relation between them. This is used as a control to assure that the results produced are as reliable as they can be.

The second research question listed above indicates that the thesis will be limited to the subject of rock mass in the range of fair to extremely good quality. This is only partially the case. Some sections or parts of the rock mass may be considered to be of a poorer quality than fair. These sections are denoted as zones of weakness. The general picture of the rock mass conditions is that it is characterized as hard rock, with quality mainly between fair and extremely good.

1.4 Readers guidance

The structure of this thesis, with its six main chapters, is presented in figure 1 beneath.

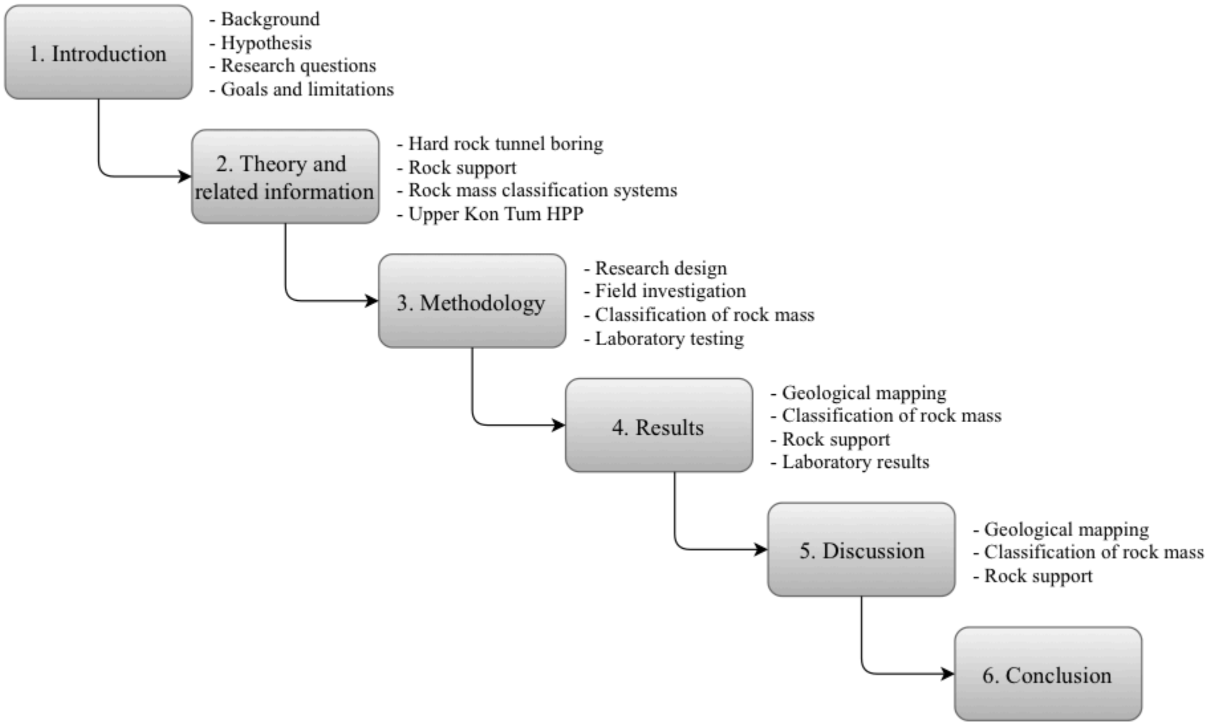

Figure 1: The structure of the thesis

After these main chapters, some thoughts about further work within this topic is presented before references and appendixes is listed up.

Figures and tables are numbered chronologically throughout the report. Equations, on the other hand, is referred to with two numbers separated by a punctuation mark. The first number refers to the main chapter, and the second number gives the chronological order within this chapter.

2 Theory and related information

This chapter contains relevant theory for this thesis. Firstly, a short presentation of the principle behind tunnel boring is given. Then a more thorough review of rock support and rock mass classification systems are done. At last a presentation of the Hydroelectric Power Project in Upper Kon Tum, Vietnam is given.

2.1 Hard rock tunnel boring

Today, tunnel boring is a well-known technology, and it has been developing ever since the 19th century (Jakobsen et al., 2015). Because of the circular cross-section one gets with tunnel boring it is especially suited for water tunnels. The circular profile minimizes the friction between flowing water and the rock mass. Bruland (2013) has examined bored and blasted water tunnels and developed a relation between the different cross sections that transport the same amount of water. It has also proven to be a suitable cross section for railway tunnels.

The principle of tunnel boring is that cutters shaped like round disks is pressed against the rock mass with high axial thrust force while the cutterhead is rotating. In the zone of contact between the rock mass and the cutters, the rock will be grinded into powder. From this zone radial cracks propagate into the rock mass. When these cracks meet other cracks chips of rock will break loose (Macias, 2016). These other cracks may be formed by neighboring cutters or cracks that initially were present in the rock mass. This principle is shown in figure 2 under.

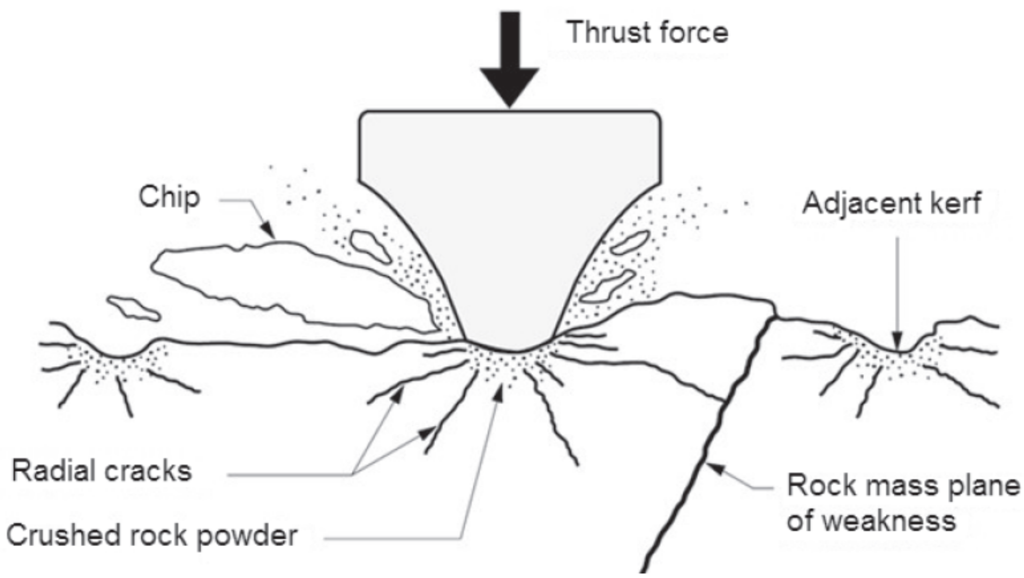

Figure 2: The principle of rock breaking and chipping in tunnel boring (Macias, 2016)

Figure 2 shows the three steps in the process of boring and breaking of rock. These are: the formation of a crack zone, propagation of radial cracks and the formation and loose breaking of rock chips (Liu et al., 2017).

The efficiency of tunnel boring is highly dependent on the quality of the rock mass. Considering tunnel boring the rock mass is denoted by its drillability. Several parameters affect the drillability. Macias (2016) divides these parameters into five categories:

- Strength and deformability
- Surface hardness
- Brittleness
- Toughness
- Abrasivity

In addition to the parameters mentioned above the jointing of the rock mass will affect the penetration rate. However, the jointing affects the rate of boring at a larger scale.

Research shows that the arrangement of cutters and the distance between neighboring cutters have big influence on the formation of chips and then also the rate of boring (Liu et al., 2017). The ideal distance between cutters are dependent on the geology and the condition of the rock mass. These conditions need to be addressed beforehand, such that the machine may be designed in a best possible way. Liu et al. (2017) concludes that the best arrangement of cutters is in a sequential pattern. This way of arranging cutters makes the radial cracks propagate longer into the rock mass than if the cutters had been placed right next to one another.

2.2 Rock support

In competent rock the support philosophy is to reinforce the rock to stand on its own. The goal is to take advantage of the rock mass' capability to carry itself, and to secure as much interaction between the rock and the support material as possible (Nilsen, 2017). In weaker rock where this is not possible one have to use more elaborate methods and more heavy equipment to support the rock mass (Bollingmo et al., 2010). Light support to reinforce the rock mass are mainly done with rock bolts, shotcrete and also grouting. Often a combination of these measures is done in tunnels for civil engineering purposes today.

When it comes to heavy rock support methods, casted concrete linings have historically been a popular solution. Recently reinforced ribs of sprayed concrete have more or less replaced these casted in-place concrete structures (NGI, 2015).

Other conditions such as squeezing or swelling rock may call for other types of rock support. This could be support systems that allows a bit of deformation before they start acting, or systems with very high bearing capacity such as lattice girders in combination with shotcrete (Bollingmo et al., 2010).

Some methods or techniques for tunneling have been developed over the years. Two of the most famous ones are the New Austrian Tunneling Method (NATM) and Norwegian Method of Tunneling (NMT) (Barton et al., 1995). The methods consist of a complete guidance from the preinvestigation stage to excavation, and they give recommendations regarding rock support. They are based on decades of experience from tunneling practice.

NATM originates from the Alpine region and was developed for tunneling in weak or squeezing rock (Barton et al., 1995). Such conditions often require heavy support systems, and the rock cannot be trusted to bear itself.

NMT is the result of the extensive tunneling activity done in Norway over many decades (Barton and Grimstad, 1994). Because of the hard rock region that Norway is (Grimstad et al., 2007), this method is applicable firstly in hard and jointed rock. NMT has gained a lot of its reputation because of technical advances on the fields of rock bolting, shotcrete and grouting.

2.2.1 Rock bolts

Rock bolts are a widely used measure in supporting rock mass underground. Rock bolts can either be used in a systematically pattern to give general support to the rock mass, or in a more irregular pattern supporting only given places which are assumed to represent a danger of outfall (NGI, 2015). There are several types of rock rock bolts, and they each have their advantages and disadvantages. In the following, some of the most important ones are explained in more detail, based on (Rostad et al., 2014) and (Stefanussen, 2017).

End anchored rock bolts

End anchored rock bolts are often used when there is a need for immediate support during excavation. They are fast to install and increases the friction between joints which enlarges the strength of the rock mass right after installation. Mechanically anchored rock bolts are the most common type of the end anchored rock bolts. They use an expansion sleeve at the end of the bolt, which give support against outfall of blocks. This rock bolt type is only allowed as temporary support. End anchored rock bolts using polyester for anchoring are an alternative which are allowed also for permanent support. A patron of polyester is placed at the end of the hole. The supporting effect is gained when the bolt is screwed trough the patron which blends the polyester with a hardening agency.

Fully embedded rock bolts

Fully embedded rock bolts provide a passive support of the rock mass. That means they are working first when the rock mass is starting to deform. Rock bolts of this kind are widely used as permanent support. These rock bolts are installed in a hole filled with an embedment agency, either cement based or polyester, and left there until its hardened. Rebar rods are often used as bolts of this kind. Full embedment is also good protection against corrosion.

Frictional rock bolts

Frictional blots create an outgoing pressure from the drill hole against the rock mass. This gives support to the rock mass along the hole length of the bolt, and the support effect is achieved immediately after installation. Protection against corrosion is a challenge with this kind of rock bolts.

Combinational rock bolts

Combinational rock bolts are end anchored rock bolts that can also by casted later on. The purpose is to be able to use the same bolt for both temporary support and permanent support. The bolt is pre-tensioned and cement grout or polyester is injected into the hole through a specially produced disc.

2.2.2 Shotcrete

Shotcrete, or sprayed concrete, is a type of concrete consisting of cement, sand, fine aggregates and admixtures (Martin et al., 2015). It is pneumatically applied and adhesively clings to the object it is applied to. In the area of civil engineering shotcrete has been used for as long as nearly 100 years.

Admixtures used in shotcrete includes accelerators, retarders, micro silica, plasticizers and reinforcing components. These will affect the shotcretes spraying ability, adhesion, curing time and cured strength.

As a product shotcrete exists in to variants: dry mix and wet mix (Martin et al., 2015). Dry mix shotcrete is delivered on site in bags or by bulk. The dry mix of cement, aggregate and some admixtures is put in a hopper, before compressed air and a rotating barrel are feeding the mix into the delivery hose. Water is first added at the tip of the delivery hose through a nozzle. Dry mix shotcrete requires smaller and less expensive equipment than wet mix. It is therefore more mobile and more applicable in many situations.

Wet mix shotcrete is mixed at a batch plant. A vehicle then transports the shotcrete from the plant to the application site. At the site, the mixed shotcrete is poured into a pump which delivers it to the hose. At the nozzle air and an accelerating admixture is added before it is sprayed on the given surface. Wet mix shotcrete demands bigger and more expensive equipment but has the capability to deliver a significantly higher amounts of shotcrete than dry mix. Table 1 sums up some advantages and disadvantages with both dry and wet mix shotcrete.

Table 1: Advantages and disadvantages with various types of shotcrete (AuSS, 2010)

	Advantages	**Disadvantages**
Dry mix	Low cost High transport distance Suited for start/stop operations May be delivered in various packaging	High maintenance Mixing at worksite More labour required Higher generation of dust Quality may vary more Lower placement rate
Wet mix	Low maintenance Accurate batch mixing at plant Low dust Consistent quality Low generation of dust High placement rate	Low transport distance High cost Mostly applicable for high consumptions

Because wet mix shotcrete is mixed at a plant it is done more accurately than dry mix (Martin et al., 2015) when water is only added at the nozzle. A typical mix design of a wet mix shotcrete for civil engineering purposes is given in table 2.

Table 2: Typical mix design of a wet mix shotcrete (Martin et al., 2015)

Ingredient	Amount (kg/m^3)	Percentage (%)
Cement	420	18,1
Silica fume	40	1,7
Aggregate	1600	68,9
Steel fiber	60	2,6
Accelerator	13	0,6
Superplasticizer	60	0,3
Water reducer	20	0,1
Water	180	7,7
Total	**2321**	**100**

The water/cement-ratio is an important parameter regarding any type of concrete. It affects the curing time, strength development, adhesion, durability and workability. For dry mix shotcrete the water/cement-ratio is around 35 %, while it is a bit higher for wet mix shotcrete, usually around 45 % (Martin et al., 2015). The mix design of table 2 produces a shotcrete with a water/cement-ratio of 43 %.

Shotcrete is, as other types of concrete, often reinforced. For weak rock conditions, welded wire mesh is one way of reinforcing the shotcrete (Martin et al., 2015). The mesh is then attached to the rock and sprayed with shotcrete till its covered. This kind of reinforcement gives high support capability and also allows for a bit deformation.

Another way of reinforcing shotcrete is by applying fibers to the mix. Fibers are either made of steel or polypropylene (AuSS, 2010). The main purpose of adding fibers is to increase the post cracking load capability of the shotcrete, or in other words the toughness of the shotcrete. Compared to mesh reinforcement, fibers are easier to use and less time demanding. It also distributes the reinforcement more evenly than wire mesh, and follows an uneven rock profile better.

Whenever possible the surface which shotcrete is to be applied to should be sprayed with water beforehand (AuSS, 2010). This is done to remove dust and particles such that sufficient adhesion between rock mass and shotcrete is achieved.

2.2.3 Reinforced ribs of sprayed concrete

Reinforced ribs of sprayed concrete is quiet simply rebars placed in and arch around the tunnel profile, mounted with radial rock bolts and covered with shotcrete (Holmøy and Aagaard, 2002). Shotcrete is also often used to even out the profile before placing the rebars. The rebars is placed together, usually 6 at the time with spacing 5-10 cm. Cross beams are then welded to the rebars to assure stiffness and interaction between them. The radial rock bolts used to mount the rib are usually fully embedded bolts, and are placed with a spacing distance of approximately 1,5 meters. At last a layer of shotcrete is added and should cover the rebars with a thickness of at least 5 cm. This procedure completes one rib, and it is repeated for as many times as necessary to give the rock mass good support. Spacing between neighboring ribs are typically between 1,5 and 2,5 meters dependent on the quality of the rock mass.

Figure 3 shows how reinforced ribs of sprayed concrete looks both in large scale and in detail.

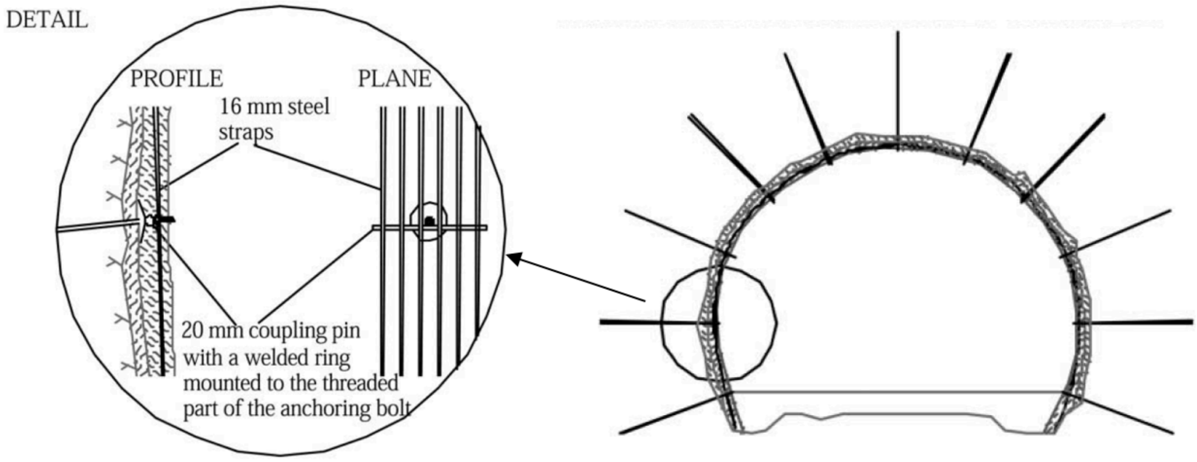

Figure 3: Construction of reinforced ribs of sprayed concrete (Holmøy and Aagaard, 2002)

Holmøy and Aagaard (2002) discusses that reinforced ribs of sprayed concrete may in many cases replace the traditional fully casted concrete lining. The method provides good enough support and is a more flexible method than traditional concrete lining. It is flexible in the way than one can vary the distance between neighboring ribs, the thickness of shotcrete, the number of rebars in each rib and the number of layers of rebars in each rib.

2.2.4 Concrete lining

Concrete lining may either be in situ casted or segmental elements put together to provide support to the rock mass. This is a heavy support method mostly used in weak and soft rock masses, however it is also used as a last measure in exceptional situations in hard rock cases as well (Bollingmo et al., 2010). Such situations may be when passing through large zones of weakness with low bearing capacity, for instance faults containing clay or crushed rock. It has mostly been used as a last resource in especially difficult situations.

2.2.5 Steel sets/ringbeams

Steel sets is used as rock support when the rock mass quality is poor and in difficult ground conditions. Steel sets may be designed as a deformable system to deal with special cases of ground behavior (Sve et al., 2008). This is mostly appropriate in conditions related to high stress and squeezing problems. The steel sets are then designed to be able to shrink a given distance in interaction with the rock mass. With this approach, the high forces affecting the rock mass is absorbed by the support system.

The main measure to deal with poor ground conditions when building a tunnel with an open TBM is steel sets (Barton, 2000). These steel sets are often called ringbeams, because of the circular cross section in bored tunnels. Most open TBMs has a ringbeam erector installed which puts the ringbeams in place right behind a protective shield, where difficult ground is encountered.

2.3 Rock mass classification systems

The field of rock mass classification has been developing for more than 100 years (Hoek, 2006). Many attempts have been done to formalize tunnel design, with special focus on rock support. These attempts are in general done with an empirical approach, derived from a series of case records.

The first known attempt to use rock mass classification in tunnel design was done by Terzaghi in 1946 (Hoek, 2006). He divided rock mass into seven categories. A rock mass was put into one of these categories based on Terzaghi's descriptions. The categories were each given a recommendation with respect to rock support.

Since Terzaghi's descriptions of rock masses until today, many systems for rock mass classifications has been developed.

The oldest approach to describe the quality of a rock mass which is still widely used today is Rock Quality Designation (RQD). The index was developed by Deere in the 1960's. RQD are mainly focusing on the degree of fracturing of the rock mass, and gives no indication of the need for rock support.

Further in the early 1970's three colleagues at Norwegian Geotechnical Institute (NGI) proposed the Q-system (Barton et al., 1974). This work was done by Barton, Lien and Lunde. The Q-system has gained wide acceptance in the field, and is to this date one of the most used classification systems for description of rock mass.

Also through the early 1970's, Bieniawski developed a new system for classification of rock masses (Bieniawski, 1973). This was called Geomechanics Classification, but is better known as Rock Mass Rating or even just RMR. RMR is a table work which quantifies different parameters of the rock mass. The Rock Mass Rating system underwent a large modification with Bieniawskis book on engineering rock mass classification in 1989 (Bieniawski, 1989). This is the most used version of the system. It is, like the Q-system, widely used also today.

RQD, Q and RMR is presented in more details in the following chapters.

2.3.1 Rock Quality Designation (RQD)

RQD is a factor expressed in percent. it is calculated by the sum of core pieces longer than 10 cm divided by the total length of the core examined (Deere, 1989). This gives the following definition of RQD shown in equation 2.1.

$$RQD = \frac{\sum Core\ pieces > 10\ cm}{Total\ length\ of\ core} \cdot 100 \tag{2.1}$$

Figure 4 under shows a sketch of how a two meter long core could look like, how the length of the pieces are measured, and how the RQD-value is calculated.

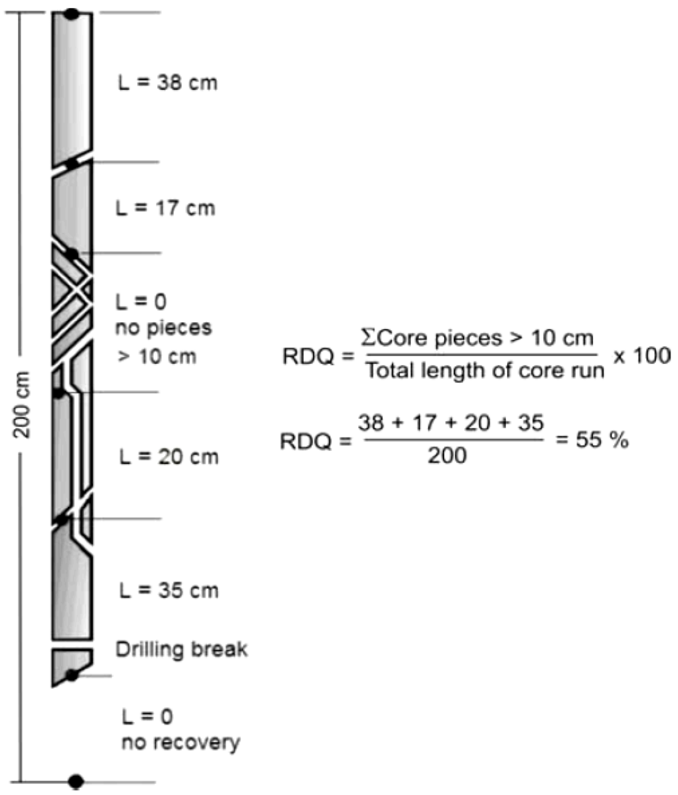

Figure 4: Principle and calculation of RQD (Deere, 1989)

RQD takes only the fracturing of the rock mass into account. It is therefore not a complete system to classify rock mass (Bieniawski, 1984), but it expresses the degree of fracturing in a good way.

In both the Q-system and RMR, RQD is taken into account as an input parameter. This is one of the reasons why RQD still a commonly used factor in rock engineering more than 50 years after it was presented.

Some issues with the usage of RQD has been discovered, despite its relatively concise definition. Pells et al. (2017) concludes that RQD is used in different ways in different parts of the world. Analyzes of core logs are also done differently dependent on the person performing it and the rock type. This could result in different RQD-values for the same core, which is not favorable for the general understanding of rock masses. Pells et al. (2017) therefore recommend that RQD should be phased out. However, since both Q and RMR still are dependent of RQD it will take some time to implement a new parameter.

In addition to this there are two factors which may influence the results. Both Hoek (2006) and Zhang (2016) states that the direction of the drilled hole and the length of the core examined may have an impact on the result in many cases.

In the light of these problems many attempts have been made to rule out the problem of the directional dependence of RQD. Palmström has suggested a relationship between RQD and a parameter J_v, called volumetric joint count. Equation 2.2 was proposed in 1982 (Palmstrøm, 1982), and was later updated to the expression shown in equation 2.3 in 2005 (Palmstrøm, 2005).

$$RQD = 115 - 3.3 \cdot J_v \tag{2.2}$$

$$RQD = 110 - 2.5 \cdot J_v \tag{2.3}$$

J_v = sum of the number of joints per unit length, for all joint sets

This method uses a volumetric joint count, which reduces the directional dependence of the traditional RQD method in many cases. Palmström (2005) concludes that this approach still is a good way of estimating rock mass jointing. Equation 2.3 may be used to estimate RQD where there are no core logs available.

Figure 5 illustrates the issue of directional dependence connected to RQD in a good way. If the distance between joints is around 10 cm, one may for the almost same degree of fracturing get RQD-values varying from 0 to 100.

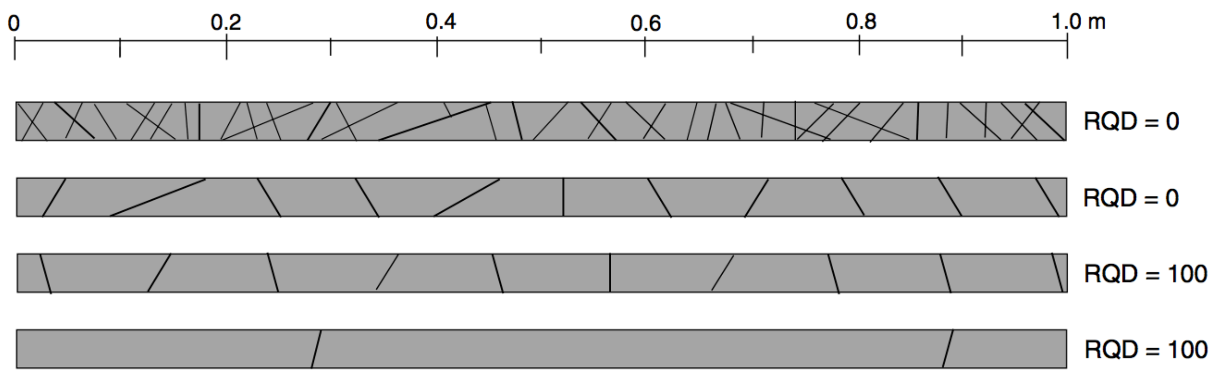

Figure 5: Examples of RQD-values for various drill cores (Palmstrøm, 2001)

2.3.2 Rock Mass Quality Index (the Q-system)

The Rock Mass Quality Index, called the Q-system, is a well known method for describing rock mass quality. It was first published in 1974 (Barton et al., 1974). The Q-system classifies rock mass in a quantitative way by a number between 0,001 and 1000. Where 1000 is a exceptionally good rock mass and 0,001 is exceptionally poor. In addition to this, the Q-system gives recommendations regarding necessary rock support.

The definition of the Q-value, displayed in equation 2.4, is still the same as in 1974, but the support requirements has gone through several updates. The reason for these updates is the development in both support techniques and support materials (NGI, 2015). In particular two big updates has been carried out. One in 1993 (Grimstad and Barton, 1993) and one in 2002 (Barton, 2002). The basis for these two updates is a large number of case records. Nearly 2000 case records have been looked into to secure good recommendations for rock support.

$$Q = \frac{RQD}{J_n} \cdot \frac{J_r}{J_a} \cdot \frac{J_w}{SRF} \tag{2.4}$$

RQD = Rock Quality Designation
J_n = joint set number
J_r = joint roughness number
J_a = joint alternation number
J_w = joint water reduction factor
SRF = Stress Reduction Factor

One can see from the definition that the Q-value is a product of three factors multiplied together. These three factors each represent an aspect of the stability of rock mass, which is:

$\frac{RQD}{J_n}$ = Relative block size

$\frac{J_r}{J_a}$ = The minimum shear strength of the rock mass

$\frac{J_w}{SRF}$ = Active stress

A review of the six parameters in the Q-system is given in the following paragraphs, based on NGIs publication on how to use of the Q-system from 2015 (NGI, 2015). This publication contains the most recent version of the Q-system.

RQD in the Q-system

The input value of RQD in the Q-system is set to the percentage value it has from its definition, as explained in section 2.3.1. However, given the definition of Q an input value of 0 will result in a Q-vaule of 0, which is not a valid Q-value. Therefore, the minimum RQD-value to be implemented in the Q-system is set to 10 by definition.

The joint set number (J_n)

The joint set number, J_n, is dependent on the number of joint sets occurring. It is also affected by any random joints that may be present. The value varies between 0,5 and 20, and is decided on the basis of information given in table 3.

Table 3: Values of J_n (NGI, 2015)

Class	Description	J_n
A	Massive, none or few joints	0,5-1,0
B	One joint set	2
C	One joint set, and random joints	3
D	Two joint sets	4
E	Two joint sets, and random joints	6
F	Three joint sets	9
G	Three joint sets, and random joints	12
H	Four or more joint sets, random and heavily fractured	15
J	Crushed rock, earthlike	20

Joint roughness number (J_r)

The friction along joints is dependent on the character of the surface of the joints. The surface's character is decided based on its appearance in both a small and a large scale. At a small scale one have to decide whether the surface is rough, smooth or slickensided. This refers to structures on the surface with the extent of a centimeter or a few millimeters. At a larger scale, say some decimeters or a meter, one decides if the joint is undulating, planar or chopped in its appearance. Then the joint roughness number can be quantified as table 4 dictates.

Table 4: Values of J_r (NGI, 2015)

Class	Description	J_r
A	Discontinuous joints	4
B	Rough or uneven, undulating	3
C	Smooth, undulating	2
D	Slickensided, undulating	1,5
E	Rough, uneven, planar	1,5
F	Smooth, planar	1
G	Slickensided, planar	0,5
H	Zone containing clay with no rock contact	1

Joint alternation number (J_a)

The filling material is together with the roughness decisive for the friction along the joints. There are two main factors that decides the joint alternation number: thickness and strength of the filling material. When J_a is to be quantified, joints are divided into three categories. These are: a) rock wall contact, b) rock wall contact before 10 cm shear deformation and c) no rock wall contact when sheared. Table 5 gives more information about this process.

Table 5: Values of J_a (NGI, 2015)

Class	Description	Φ_r	J_a
	a) Rock wall contact		
A	Tight joints with hard minerals, like quartz		0,75
B	Unaltered joints, oxidation only on surface	25-35°	1
C	Slightly altered joints, non-softening mineral coating	25-35°	2
D	Silty- or sandy-clay coatings	20-25°	3
E	Softening, low friction clay coating	8-16°	4
	b) Rock wall contact before 10 cm shear deformation		
F	Sandy particles, clay-free, disintegrating rock	25-30°	4
G	Very over-consolidated, non-softening clay mineral (< 5 mm)	16-24°	6
H	Medium or low over-consolidation, softening clay mineral fillings (< 5 mm)	12-16°	8
J	Fillings of swelling clay (< 5 mm). J_a depends on water access and percentage of swelling clay	6-12°	8-12
	c) No rock wall contact when sheared		
K	Zones or bands of disintegrated rock. Strongly over-consolidated, non-softening filling.	16-24°	6
L	Zones or bands of disintegrated rock. Medium to low over-consolidated, non-softening filling.	12-16°	8
M	Zones or bands of clay or disintegrated rock. J_a depends on percentage of swelling clay.	6-12°	8-12
N	Thick continuous zones of clay. Strongly over-consolidated	12-16°	10
O	Thick continuous zones of clay. Medium to low over-consolidated	12-16°	13
P	Thick continuous zones of clay. Swelling clay. J_a depends on percentage of swelling clay.	6-12°	13-20

Joint water reduction factor (J_w)

Water pressure may reduce the normal stress between joints, making it easier for blocks to fall out. Therefore are the joint water reduction factor included in the last factor in the expression of the Q-value. This factor accounts for active stresses in the rock mass. J_w is decided on the basis of observed water pressure, and water leakages. The descriptions in table 6 gives the quantification of J_w.

Table 6: Values of J_w (NGI, 2015)

Class	Decription	J_w
A	Dry, or minor inflow	1,0
B	Medium inflow or pressure, possible outwash of fillings	0,66
C	Large inflow or pressure in competent rock, unfilled joints	0,5
D	Large inflow or pressure, considerable outwash of fillings	0,33
E	Exceptionally high inflow or pressure which is declining with time	0,2-0,1
F	Exceptionally high inflow or pressure, not declining with time	0,1-0,05

Stress reduction factor (SRF)

The stress reduction factor expresses the relationship between the rock stresses and the rock strength. The stress situation is divided into four categories: a) weakness zones intersecting the tunnel or cavern, b) competent rock with stability problems related to high stresses or the lack of stresses, c) squeezing rock conditions and d) swelling rock conditions. The effect of stresses in rock masses may be observed in many forms, such as spalling, deformation, rock burst, squeezing and outfall of blocks. It may occur immediately after blasting or develop over time. Table 7 gives a guidance to quantification of SRF.

Table 7: Values of SRF (NGI, 2015)

Class	Description			SRF
a)	**Weakness zones intersecting excavation, may cause loosening of rock**			
A	Multiple weakness zones containing clay or chemically weathered rock			10
B	Multiple shear zones within a short section in competent rock			7,5
C	Single weakness zones (< 50 m)			5
D	Unconsolidated, open joints, heavily jointed, “sugar cube”			5
E	Single weakness zones (> 50 m)			2,5
b)	**Competent rock, stress problems**	σ_c/σ_1	σ_θ/σ_c	
F	Low stresses, near the surface	> 200	< 0,01	2,5
G	Medium stresses	200-10	0,01-0,3	1
H	High stress, tight structure. Usually favorable (But may be unfavorable for stability*)	10-5	0,3-0,4	0,5-2 2-5*
J	Moderate spalling after hours	5-3	0,5-0,65	5-50
K	Spalling and rockburst after minutes	3-2	0,65-1	50-200
L	Intense rockburst immediately	< 2	> 1	200-400
c)	**Squeezing rock**		σ_θ/σ_c	
M	Mild squeezing rock pressure		1-5	5-10
N	Heavy squeezing rock pressure		> 5	10-20
d)	**Swelling rock**			
O	Mild swelling rock pressure			5-10
P	High swelling rock pressure			10-15

When all of the six parameters are quantified and the Q-value is calculated using equation 2.4 the rock mass has been classified. Once you have got the Q-value in hand you can use the chart, shown in figure 6 to place the given rock mass into a rock mass class, and to get a recommendation for necessary rock support. To use the chart, one also need to know the span of the tunnel or cavern in question and the area of application of it. This will decide the

Excavation Support Ratio (ESR), which describes the level of safety. Some examples of values of ESR are given in table 8.

Table 8: Values of ESR (Hoek, 2006)

Class	Description	ESR
A	Temporary mine openings	3-5
B	Permanent mine openings, water tunnels, pilot tunnels	1.6
C	Storage rooms, water treatment plants, minor road and railway tunnels, surge chambers, access tunnels	1.3
D	Power stations, major road and railway tunnels, civil defense chambers, portal intersections	1.0
E	Underground nuclear power stations, railway stations, sports and public facilities, factories	0.8

The information in table 8 is then implemented into the chart (figure 6) as the fraction, shown in equation 2.5. The result of the fraction is called the equivalent dimension, or D_e.

$$D_e = \frac{Span, diameter\ or\ heiht\ of\ opening\ (m)}{ESR} \tag{2.5}$$

D_e = equivalent dimension

ESR = Excavation Support Ratio

One may now read the chart in figure 6 with the Q-value on the x-axis and the result of the fraction from equation 2.5 on the y-axis, and get a recommendation for rock support.

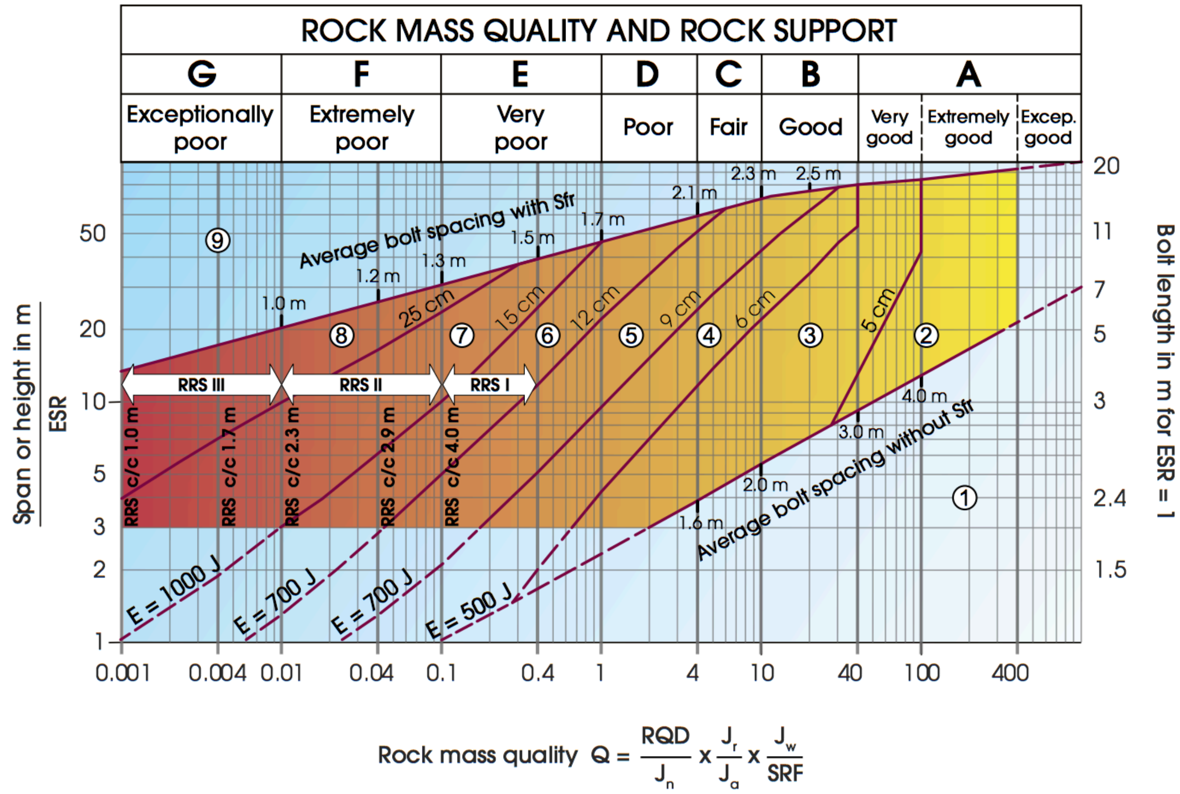

Figure 6: Recommendation of rock support using the Q-system (NGI, 2015)

From figure 6 it can be seen that the Q-system divides rock mass into seven categories from A: Exceptionally good rock mass, down to G: exceptionally poor rock mass. The recommended use of rock support is divided into nine categories, represented by the numbers from 1 to 9 in the chart area. See table 9 for more information about these categories.

The recommendations given are firstly concerning rock bolts and shotcrete as support methods. Indications for average distance between rock bolts are given on the borderlines of the orange field. The upper borderline suggests bolting distance when shotcrete is used, while the lower borderline suggests distance between rock bolts when no shotcrete is used. The red lines crossing the orange field suggests thicknesses of the layer of shotcrete needed. This version of the chart also gives indication for when it may be necessary to use reinforced ribs of sprayed concrete. In the chart, this is referred to as the abbreviation RRS. As it can be seen from figure 6, it is recommended to use reinforced ribs of sprayed concrete when the Q-value is lower than 0,4. RRS are then divided into three categories each for specific intervals of Q-values. More information about the categories of RRS and rock support measures using the Q-system is given below in table 9.

Table 9: Support categories in the Q-system (NGI, 2015)

Support category	Description of recommended rock support	Class
1	Unsupported or spread bolting	
2	Spread bolting	SB
3	Bolting with fiber reinforced shotcrete 5-6 cm	B+Sfr
4	Fiber reinforced concrete, 6-9 cm and bolting	Sfr (E500) + B
5	Fiber reinforced concrete, 9-12 cm and bolting	Sfr (E700) + B
6	Fiber reinforced concrete, 12-15 cm + reinforced ribs of sprayed concrete, class I and bolting	Sfr (E700) + RRS I + B
7	Fiber reinforced concrete > 15 cm + reinforced ribs of sprayed concrete, class II and bolting	Sfr (E1000) + RRS II + B
8	Full cast concrete lining or fiber reinforced concrete > 15 cm + reinforced ribs of sprayed concrete, class III + bolting	CCA or Sfr (E1000) + RRS III + B
9	Special cases, special measures are needed	
Support categories concerning reinforced ribs of sprayed concrete (10 m span width)		
RRS I	Single layer of 6 rebars (Ø 16-20), 30 cm thickness of shotcrete	
RRS II	Double layer of 6 rebars (Ø 16-20), 45 cm thickness of shotcrete	
RRS III	Double layer of 6 rebars (Ø 20), 55 cm thickness of shotcrete	

Concerning table 9: B = bolting, Sfr = fiber reinforced shotcrete, E500 = energy absorption in shotcrete of 500 Joule.

2.3.3 Rock Mass Rating (RMR)

The Geomechanics Classification was introduced in 1973 by Bieniawski (Bieniawski, 1973). Bieniawski realized that the various classification systems existing at the time didn't take all the factors that are influencing the rock mass into account. The goal of this new classification system was "to provide one comprehensive classification which could meet most practical applications, and which, most of all, could ensure *effective communication between the engineer* and the *geologist* or between the *designer* and the *contractor*" (Bieniawski, 1973).

Later this system is better known as Rock Mass Rating, especially after the modification of 1989 (Bieniawski, 1989). Table 18 shows the modified version of the RMR system which were introduced in this book.

To be able to quantify a rock mass with a number, the RMR system takes five parameters regarding the rock mass into account. These are:

1. Strength of intact rock material
2. Drill core quality – RQD
3. Spacing of discontinuities
4. Conditions of discontinuities
5. Groundwater

For all of these parameters a number is given which indicates the quality of the given parameter. At last all of these numbers are summed together which gives the RMR-value. This results in a number between 0 and 100. Where a value of 100 indicates the best rock mass possible.

In addition to these five parameters an adjustment factor has been added. This accounts for the orientation of discontinuities, and ranges from vary favorable to very unfavorable.

In the following paragraphs a walkthrough of the five main parameters and the additional adjustment factor is given.

Strength of intact rock material

The strength is measured by the Point-Load Index, the Uniaxial Compressive Strength or both of them. The results of the tests are measured in MPa and given a rating as shown in table 10.

Table 10: Rating related to strength of intact rock material (Bieniawski, 1989)

Strength index	**Strength (MPa)**						
Point-Load Strength Index	> 10	10 – 4	4 – 2	2 – 1	Uniaxial Compressive Strength is preferred		
Uniaxial Compressive Strength	> 250	250 – 100	100 – 50	50 – 25	25 – 5	5 – 1	< 1
Rating	15	12	7	4	2	1	0

Drill Core Quality – RQD

The parameter concerning drill core quality is quantified based on the Rock Quality Designation-value as described in section 2.3.1. Table 11 shows the rating in the RMR-system in relation to given intervals of RQD.

Table 11: Rating related to drill core quality (Bieniawski, 1989)

	Drill Core Quality				
RQD	100 – 90 %	90 – 75 %	75 – 50 %	50 – 25 %	< 25 %
Rating	20	17	13	8	3

Spacing of discontinuities

The distance between discontinuities affect the rock mass' behavior and is taken into account in the RMR-system as shown in table 12.

Table 12: Rating related to spacing of discontinuities (Bieniawski, 1989)

	Spacing in cm				
Spacing	> 200	200 – 60	60 – 20	20 – 6	< 6
Rating	20	15	10	8	5

Condition of discontinuities

The discontinuities appearance and the degree of separation will affect the friction between blocks in the rock mass. A guide to decide the rating of this parameter is given in table 13. A more detailed guide is presented in table 18 under bullet E.

Table 13: Rating related to conditions of discontinuities (Bieniawski, 1989)

Rating	Description
30	Very rough surface, not continuous, no separation, unweathered
25	Slightly rough surface, separation < 1 mm, slightly weathered
20	Slightly rough surface, separation < 1 mm, highly weathered
10	Slickensided surface or gouge < 5 mm thick or separation 1 – 5 mm, continuous
0	Soft gouge > 5 mm or separation > 5 mm, continuous

Groundwater

The presence of groundwater or water pressure underground will also affect the rock mass. The ratings in hence to this in the RMR-system are shown in table 14.

Table 14: Rating related to groundwater (Bieniawski, 1989)

Factor					
Inflow per 10 m tunnel (L/min)	None	< 10	10 – 25	25 – 125	> 125
Joint water pressure, major principal stress	0	< 0.1	0.1 – 0.2	0.2 – 0.5	> 0.5
General conditions	Dry	Damp	Wet	Dripping	Flowing
Rating	15	10	7	4	0

Adjustment for discontinuity orientation

The quantification of the adjustment factor for the orientation of discontinuities is done on the basis of table 15. This factor gives negative ratings when the orientation is not very favorable. This adjusts the RMR-value to also account for the orientation of joints.

Table 15: Rating related to discontinuity orientation (Bieniawski, 1989)

Type of structure	Rating related to orientation				
	Very favorable	Favorable	Fair	Unfavorable	Very unfavorable
Tunnels	0	- 2	- 5	- 10	- 12
Foundations	0	- 2	- 7	- 15	- 25
Slopes	0	- 5	- 25	- 50	- 60

When all the parameters have been quantified, the RMR-value can easily be found by summing all the ratings together. Bieniawski has then defined five classes of rock mass which are decided by the RMR-value. These classes, shown in table 16, is supposed to simplify the communication between disciplines and stakeholders in a project.

Table 16: Rock mass classes related to RMR-values (Bieniawski, 1989)

	Rock mass classes				
Total rating (RMR-value)	100 – 81	80 – 61	60 – 41	40 – 21	< 21
Class	I	II	III	IV	V
Description	Very good rock	Good rock	Fair rock	Poor rock	Very poor rock

Bieniawski has developed recommendations regarding rock support for the five rock mass classes in the RMR system. These recommendations are shown in table 17 below.

Table 17: Rock support recommendations using RMR (Bieniawski, 1989)

Rock mass class	Excavation	Rock bolts (20 mm diameter, fully grouted)	Shotcrete	Steel sets
I - Very good rock *RMR*: 81-100	Full face, 3 m advance.	Generally no support required except spot bolting.		
II - Good rock *RMR*: 61-80	Full face , 1-1.5 m advance. Complete support 20 m from face.	Locally, bolts in crown 3 m long, spaced 2.5 m with occasional wire mesh.	50 mm in crown where required.	None.
III - Fair rock *RMR*: 41-60	Top heading and bench 1.5-3 m advance in top heading. Commence support after each blast. Complete support 10 m from face.	Systematic bolts 4 m long, spaced 1.5 - 2 m in crown and walls with wire mesh in crown.	50-100 mm in crown and 30 mm in sides.	None.
IV - Poor rock *RMR*: 21-40	Top heading and bench 1.0-1.5 m advance in top heading. Install support concurrently with excavation, 10 m from face.	Systematic bolts 4-5 m long, spaced 1-1.5 m in crown and walls with wire mesh.	100-150 mm in crown and 100 mm in sides.	Light to medium ribs spaced 1.5 m where required.
V – Very poor rock *RMR*: < 20	Multiple drifts 0.5-1.5 m advance in top heading. Install support concurrently with excavation. Shotcrete as soon as possible after blasting.	Systematic bolts 5-6 m long, spaced 1-1.5 m in crown and walls with wire mesh. Bolt invert.	150-200 mm in crown, 150 mm in sides, and 50 mm on face.	Medium to heavy ribs spaced 0.75 m with steel lagging and forepoling if required. Close invert.

Table 18 contains the whole procedure of the Rock Mass Rating system. It includes some more notes and details than what is mentioned above. This sheet is of great help when one is to classify rock mass using the RMR-system.

Table 18: Procedure of RMR (Bieniawski, 1989)

A. CLASSIFICATION PARAMETERS AND THEIR RATINGS

	Parameter		Range of values						
1	Strength of intact rock material	Point-load strength index	>10 MPa	4 - 10 MPa	2 - 4 MPa	1 - 2 MPa	For this low range - uniaxial compressive test is preferred		
		Uniaxial comp. strength	>250 MPa	100 - 250 MPa	50 - 100 MPa	25 - 50 MPa	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	3		
3	Spacing of		> 2 m	0.6 - 2 . m	200 - 600 mm	60 - 200 mm	< 60 mm		
	Rating		20	15	10	8	5		
4	Condition of discontinuities (See E)		Very rough surfaces Not continuous No separation Unweathered wall rock	Slightly rough surfaces Separation < 1 mm Slightly weathered walls	Slightly rough surfaces Separation < 1 mm Highly weathered walls	Slickensided surfaces or Gouge < 5 mm thick or Separation 1-5 mm Continuous	Soft gouge >5 mm thick or Separation > 5 mm Continuous		
	Rating		30	25	20	10	0		
5	Groundwater	Inflow per 10 m tunnel length (l/m)	None	< 10	10 - 25	25 - 125	> 125		
		(Joint water press)/ (Major principal σ)	0	< 0.1	0.1, - 0.2	0.2 - 0.5	> 0.5		
		General conditions	Completely dry	Damp	Wet	Dripping	Flowing		
	Rating		15	10	7	4	0		

B. RATING ADJUSTMENT FOR DISCONTINUITY ORIENTATIONS (See F)

Strike and dip orientations		Very favourable	Favourable	Fair	Unfavourable	Very Unfavourable
Ratings	Tunnels & mines	0	-2	-5	-10	-12
	Foundations	0	-2	-7	-15	-25
	Slopes	0	-5	-25	-50	

C. ROCK MASS CLASSES DETERMINED FROM TOTAL RATINGS

Rating	100 ← 81	80 ← 61	60 ← 41	40 ← 21	< 21
Class number	I	II	III	IV	V
Description	Very good rock	Good rock	Fair rock	Poor rock	Very poor rock

D. MEANING OF ROCK CLASSES

Class number	I	II	III	IV	V
Average stand-up time	20 yrs for 15 m span	1 year for 10 m span	1 week for 5 m span	10 hrs for 2.5 m span	30 min for 1 m span
Cohesion of rock mass (kPa)	> 400	300 - 400	200 - 300	100 - 200	< 100
Friction angle of rock mass (deg)	> 45	35 - 45	25 - 35	15 - 25	< 15

E. GUIDELINES FOR CLASSIFICATION OF DISCONTINUITY conditions

Discontinuity length (persistence) Rating	< 1 m 6	1 - 3 m 4	3 - 10 m 2	10 - 20 m 1	> 20 m 0
Separation (aperture) Rating	None 6	< 0.1 mm 5	0.1 - 1.0 mm 4	1 - 5 mm 1	> 5 mm 0
Roughness Rating	Very rough 6	Rough 5	Slightly rough 3	Smooth 1	Slickensided 0
Infilling (gouge) Rating	None 6	Hard filling < 5 mm 4	Hard filling > 5 mm 2	Soft filling < 5 mm 2	Soft filling > 5 mm 0
Weathering Ratings	Unweathered 6	Slightly weathered 5	Moderately weathered 3	Highly weathered 1	Decomposed 0

F. EFFECT OF DISCONTINUITY STRIKE AND DIP ORIENTATION IN TUNNELLING**

Strike perpendicular to tunnel axis		Strike parallel to tunnel axis	
Drive with dip - Dip 45 - 90°	Drive with dip - Dip 20 - 45°	Dip 45 - 90°	Dip 20 - 45°
Very favourable	Favourable	Very unfavourable	Fair
Drive against dip - Dip 45-90°	Drive against dip - Dip 20-45°	Dip 0-20 - Irrespective of strike°	
Fair	Unfavourable	Fair	

* Some conditions are mutually exclusive . For example, if infilling is present, the roughness of the surface will be overshadowed by the influence of the gouge. In such cases use A.4 directly.

2.3.4 Relation between Q and RMR

Over the years some attempts have been done to describe a relationship between Q-values and RMR-values. For instance, both Barton and Bieniawski have each proposed an equation trying to express a connection between Q and RMR.

Bieniawski published the following equation, 2.6, already in 1976. It is based on 111 case records from that time (Bieniawski, 1989).

$$RMR = 9 \cdot \ln(Q) + 44 \tag{2.6}$$

The following equation, 2.7, was published by Barton in 1995 (Barton, 1995).

$$RMR = 15 \cdot \log(Q) + 50 \tag{2.7}$$

2.3.5 Limitations of rock mass classification systems

One of the biggest limitations of Q and RMR as classification systems is the inaccuracy caused by RQD (Palmstrøm, 2005). The author argues that it is due time that the fracturing or the block size of the rock mass is accounted for in a more accurate way than RQD.

The Q-system gives quiet detailed recommendations regarding rock support, through a chart displayed in figure 6. Palmstrøm, Blindheim and Broch (2002) argues that the scope of this chart is disproportionally big. For both very large and very small spans it is not likely that the support will be based on Q-values. For rock mass classes F and G, extremely poor and exceptionally poor rock mass, there is no simple way of deciding the amount of rock support needed. In such situations, careful considerations must be made to ensure proper rock support. Also for very high Q-values there are often only need for spot bolting and there are in general no need for recommendations through a chart. In figure 7 Palmstrøm, Blindheim and Broch have marked the area which they mean the Q-system is applicable.

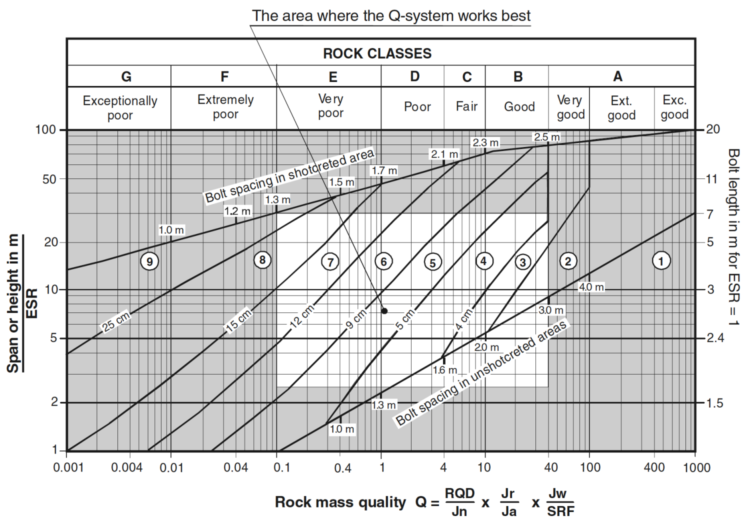

Figure 7: Limitations of the support recommendation as discussed by Palmstrøm et al. (2002)

Palmstrøm and Broch (2006) discusses two possible approaches regarding geological mapping when using a classification system. The first is to only evaluate parameters that are included in the given classification system. The second is to characterize the rock mass in a best possible way including all relevant information, and then later give quantified ratings to the parameters in a classification system. The authors emphasize that they recommend the second approach. It is important that all factors influencing the rock mass are included and not only the parameters in a classification system. The approach is displayed as a flowchart in figure 8.

Hoek (2006) also recommends this method of mapping rock mass. He states that a complete description of the rock mass may be easily translated into whatever classification system. If only ratings were recorded during mapping it would be impossible to carry out verification studies.

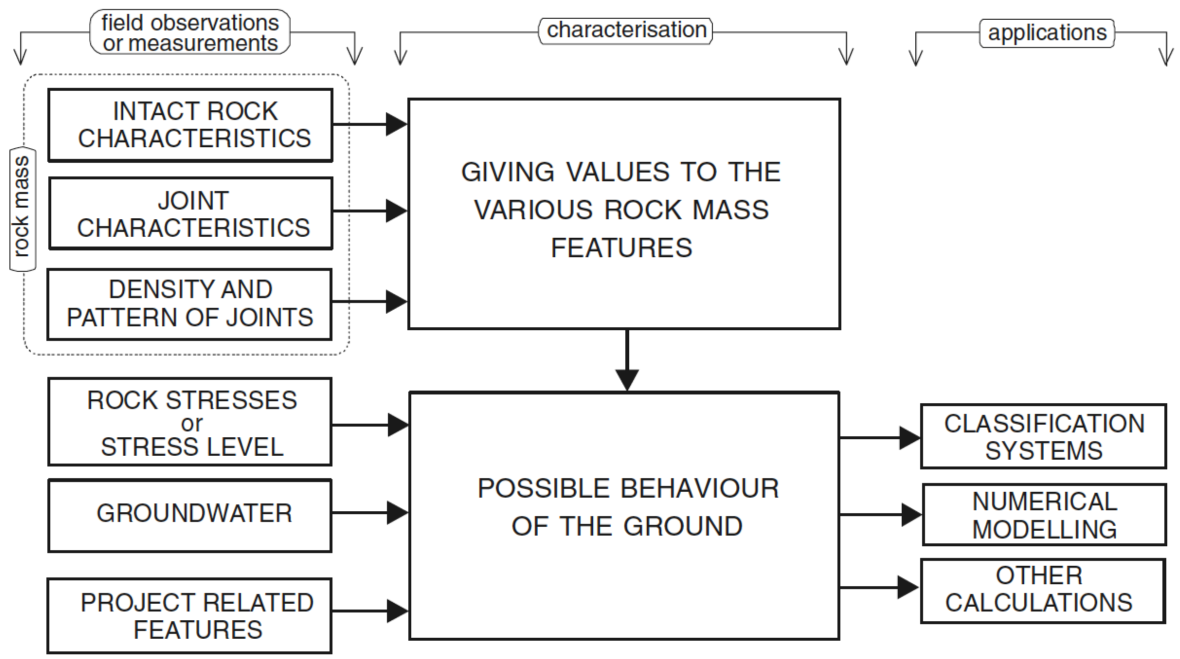

Figure 8: Recommended approach by Palmstrøm and Broch (2006)

The type of ground conditions where classification systems are applicable have been questioned. Palmstrøm and Broch (2006) states that the Q-system works best in jointed and fractured rock mass which is otherwise competent. In conditions like this outfall of blocks is the main concern. It is also stated that the Q-system works fair in slabbing or spalling rock conditions. In other conditions than the ones mentioned it is questioned whether the Q-system is suitable.

Given the limitations that exists in using rock mass classification systems, Hoek (2006) argues that one should use at least two classification systems when classifying rock mass. This is a way of assuring that more information about the rock mass is taken into consideration.

2.4 Upper Kon Tum Hydroelectric Power Project

The Upper Kon Tum Hydroelectric Power Project (HPP) is located in Kon Plông district in the northern part of the Kon Tum province, which is in the central highlands of Vietnam. The Kon Tum province is located by the Vietnamese border to both Laos and Cambodia. Mountains and valleys with rivers characterizes the scenery, which is ideal for hydropower plants. Figure 9 under shows the locations described. The red marker in the picture to the right indicates the location of the project area.

Figure 9: Location of Kon Tum to the left, and of Kon Plông district to the right (Google Maps)

The Upper Kon Tum HPP is one of many hydropower plants along the Se San river in Vietnam. The water will be collected in a dam at the western side of the project area, and lead through a headrace tunnel of 18 km to the outlet. See the map in figure 10, where the red dot is the dam, the black line is the headrace tunnel and the red marker is the outlet spot.

Figure 10: Map showing the project area (Google Maps)

The headrace tunnel is excavated with a main beam tunnel boring machine (see chapter 2.4.2) and driven from the outlet side from an elevation of about 500 meters above sea level. From this point 13 km of the tunnel is excavated by TBM, where it is met by a drill and blasted tunnel driven form the dam side of the project. The inclination of the TBM tunnel is about 3 %. The elevation of the tunnel portal at the dam side is around 1150 meter above sea level.

2.4.1 Geology

The geological structure of the project site can be described as a hard rock region. The rock is dived into mainly to formations along the tunnel profile. From the outlet side where the TBM started there is hard gneiss and granite gneiss for around 9 km. At about 9 km into the mountain it is predicted to change to biotite granite. The biotite granite is predicted to consist of 30 % quartz, 30 % plagioclase, 30-40 % orthoclase and 5-10 % mica. In figure 11 the gneiss can be seen as the area with the undulating stripes to the right, and the biotite granite can be seen as the more dotted area to the left.

The faulting of the rock mass at a large scale follow a pattern which is shown in figure 11 under. It is characterized by steep dips, approximate 75-85°, with strike towards west which is almost orthogonal to the tunnel alignment.

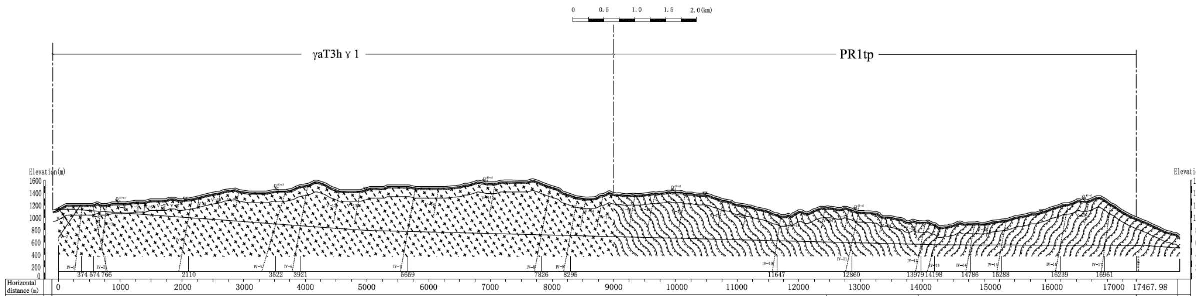

Figure 11: Geological profile of the headrace tunnel at Kon Tum, both TBM and drill and blast

A larger picture of figure 11 is provided in appendix E.

2.4.2 The TBM

The TBM used to excavate the headrace tunnel is a main beam TBM, also callsed open gripper TBM. Grippers on the side of the machine pushes against the sidewalls locking the machine in place. Then propel cylinders pushes the rotating cutterhead at the front of the machine against the rock in the tunnel face. At the cutterhead, disc cutters are placed to roll over the rock as the cutterhead is rotating. This creates the breaking of the rock, as describes in chapter 2.1. A roof shield covers the area right behind the cutterhead, which provides safety for both workers and the machine. The main beam sitting in the center of the machine provides bearing. A sketch of a traditional Robbins main beam TBM is shown in figure 12, and table 19 sums up some key numbers regarding the machine at Upper Kon Tum HPP.

Table 19: Key numbers for the TBM at Upper Kon Tum HPP

Parameter	Value
Diameter	4,5 meters
Number of cutters	30
Diameter of cutters	17 " (432 mm)
Thrust force	8800 kN

Some areas on the TBM are often given simple names, to simplify communication. On a main beam TBM areas are referred to as:

- L1: The area just behind the protective roof shield. In figure 12 this is where the yellow platforms are.
- L2: The area at the back of the TBM. In figure 12 this is the area behind the grippers.
- L3: The backup area. This is not shown on figure 12, but includes for instance systems for hydraulic fluids, electronics, cooling and muck transport. The backup is towed on rails by the TBM.

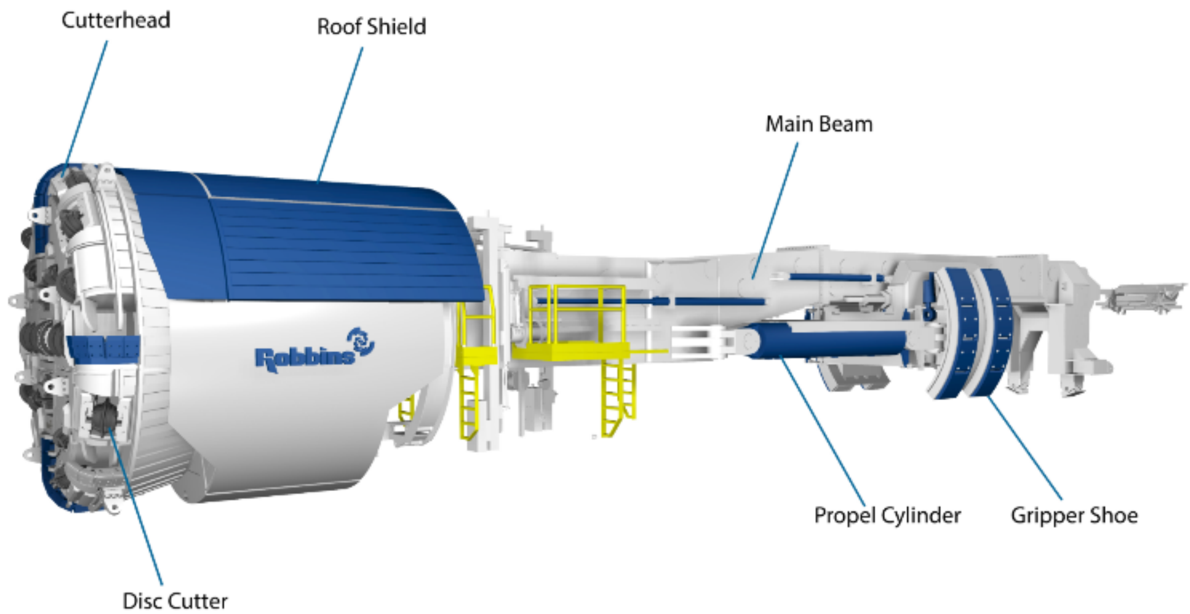

Figure 12: Main beam TBM (TheRobbinsCompany, Without year)

As mentioned the roof shield provides safety. To preserve the safety, rock support needs to be installed right behind the roof shield when the conditions calls for it. Equipment for installing rock support is mounted in both the L1 and the L2 area. In L1, there are bolting guns, ring beam erectors and systems to mount mesh and liner plates. In L2, there are robots for shotcreteing. When it is necessary, rock support is mounted also in the L3 area, or even further back as well.

2.4.3 Rock support strategy

The project owner has trough a written report defined how the rock support for the tunnel is to be carried out. In this report, it is stated that both the RMR-system and the Q-system should be used to classify the rock mass. The project owner has also provided a definition of rock mass classes which is to be used when deciding the amount of rock support. These rock mass classes are only based on the Q-system, and is given in table 20.

Table 20: Classification of rock mass classes stated by the project owner

Q-value	**1000 – 100**	**100 – 4**	**4 – 0.1**	**0.1 – 0.01**	**0.01 – 0.001**
Rock mass class	I	II	III	IV	V

Four different types of rock support are used on the project. This includes rock bolts, wire mesh, ringbeams and shotcrete. The amounts of rock support that is to be used related to the rock mass classes are given in table 21.

Table 21: Robbins TBM ground support guidelines at Kon Tum HPP

Class	**Rock bolt length (m)**	**Rock bolts per meter**	**Mesh (m^2/m)**	**Ringbeams**	**Shotcrete thickness (cm)**
I	1.8	3	0	No	0
II	1.8	3	0	No	0
III	1.8	5-6	6	If necessary	8-10
IV	1.8	12-14	10	Yes	8-12
V	1.8	12-14	0	Yes	12-15

3 Methodology

This chapter describes the methodology used to be able to answer the research questions asked in this master thesis. At the end of the chapter the strengths and weaknesses with the applied method is discussed.

3.1 Research design

The methodology is an important part of a research assignment. It explains the procedure of the assignment and how the results are obtained (Olsson, 2011). On the basis of the explained methodology the reader is supposed to understand the applied procedure and to able to recreate the results. Firstly, the choice of method stands between a qualitative approach, a quantitative approach or a combination of the two.

3.1.1 Quantitative and qualitative methods

Quantitative method is an examination of a large number of units. The data in such a research method is usually in numbers and they are often analyzed with statistical methods (Dahlum, 2017). For this thesis, a quantitative approach is used to examine data collected from geological mapping of the tunnel. This data contains a large number of observations and statistical methods is necessary to produce well-presented and good results.

Qualitative method revolves around more copious data than a quantitative method (Nordbø, 2009). It is based on less findings but there is more content in each finding that needs to be analyzed. During the work with the geological mapping of a tunnel under construction, some geological parameters needs to be described by a qualitative judgement on site. This is the only way such an exercise is feasible.

This thesis is, as described above, based on a combination of qualitative and quantitative methods. In addition, laboratory testing has been performed on rock samples gathered during the geological mapping. More about the method of laboratory testing is given in chapter 3.5.

Validity

The validity of the research method represents whether the measured phenomena is what it is sought to measure (Dahlum, 2015).

In the tunnel, a conveyor belt and ventilation installations hid some of the tunnels contour. Because of the somewhat limited access to parts of the tunnels cross section, the validity is lowered. One cannot be absolutely certain that what is observed elsewhere is also the case were the rock mass is hidden.

In addition to the limited access, the rock was alternately covered with a thin layer of dust generated by the boring process. This combined with bad lighting made it hard to observe the rock mass some places. Headlamps were used to cope with the bad lighting, and it was necessary to sweep of the dust to examine the discontinuities of the rock mass closely.

To increase the validity 755 meter of the tunnel has been mapped in total. Findings has been observed throughout the mapped section, and has never been generalized based on parts of the section. Hopefully the basis of data is big enough to reveal features of the rock mass that was hidden during the mapping. Although individual events occurring in the rock mass may nevertheless have stayed hidden.

Regarding the laboratory tests, the validity is rather low. Whether the rock samples can represent the rock mass of the project area is questionable. However, it was strived to get rock samples from an area in the tunnel who represents the rock mass in a best possible way.

Reliability

The reliability describes the stability or the trustworthiness in the measurements performed (Svartdal, 2018). It should say something about whether the measurements are independent of time, persons in charge and devices used to do the measurements.

Geological mapping is a survey strictly dependent on the people conducting it (Seo et al., 2018). The factor of subjectivity affecting the mapping results in a lowered reliability. This is the nature of the activity of geological mapping, and one can only take actions to increase the reliability.

Actions taken to increase the reliability of the methodology in this thesis is that the mapping is performed by two students in cooperation. The two students have the same objectives for the mapping activity, and they had approximately the same amount of experience when it comes to conducting geological mapping. None of the students had any connections to or were paid by any parties involved in the project. This makes the survey more objective, because the only goal for the geological mapping was to describe the rock mass as accurately as possible.

In addition, it has been strived to follow the approach proposed by Palmstrøm and Broch (2006) as shown in figure 8. These measures are supposed to make the survey less dependent on the people performing it, and thus easier to recreate by others.

While the validity of laboratory testing is low, the reliability is high. The tests were performed by SINTEFs experienced staff in standardized equipment. This should produce trustworthy results, which are independent of both the people performing it and the devices used to do the testing.

3.2 Review of literature

Through the two courses TBA 4151 Construction engineering advanced course and TBA 4128 Project management advanced course a literature review was performed. The most important result of this work was that a good method for collecting literature was rehearsed. This method has been continued for the work with this master thesis.

The method consists of finding literature in reliable databases and then evaluate the source in the light of four criteria. In table 22, these criteria are presented and described.

Table 22: Principle of evaluating literature (Norås, 2015)

Criteria	Description
Credibility	Evaluate both the authors and the publisher's credibility. Questions to be asked may be: what are the authors background, education and position? How well known is the publisher and/or the journal within the given area of research?
Objectivity	The source of data used in the research should be evaluated, and if the results is in compliance with other research on the field. One should also try to address whether the author is trying to inform you or if he is trying to persuade you in any matter.
Accuracy	Questions to be asked regarding accuracy may be: Are the research method used described in a good way? Are the data source used up to date, and is it possible to confirm this information in other sources? One should also make an evaluation of the references listed in the source.
Suitability	It is utmost importance that the sources found fits the scope of this thesis. One should therefore make an evaluation of this matter before spending a lot of time reading it. One last thing to think trough is what kind of knowledge one need to have to read the material? Do one need to be at an advanced level within the field or is it written for everybody to understand?

Validity of the literature review

By evaluating literature on the basis of the criteria's listed in table 22, it assures high validity for this method of gathering information. The validity of the information is controlled according to the four criteria. Ones thoughts around the information is sent through a process to help you decide whether the information is useful for further work or not.

Reliability of the literature review

The reliability of such a literature review is also high. When done right be addressing sources at the right places the reader should be able to track down the information used in the task. A high degree of reliability means that there is large verifiability in the work that has been done.

3.3 Field investigation

The field investigation performed for this master thesis is the geological mapping of a section of the headrace tunnel of Upper Kon Tum hydropower project in Vietnam. The tunnel, who was excavated with TBM, was mapped between chainage 8750 and 8100. In addition, an access tunnel excavated by drill and blast was mapped. This was done to compare a circular cross section of a TBM excavated tunnel to a more irregular cross section of a drilled and blasted tunnel considering both mapping and rock support.

The geological mapping was performed in several steps, and was based on the approach suggested by Palmstrøm and Broch (2006). It includes a thorough mapping of all relevant geological features visible at the site, before starting to quantify parameters to be included in a rock mass classification system. See chapter 2.3.5 for more information about this approach.

The mapping was conducted by two students together. The two students had the same goals for the survey and the same level of experience regarding geological mapping. It was decided to do the mapping together, because it was seen as an advantage to get two opinions simultaneously. The fact that the two students had the same goals for the survey meant that all relevant information for the both of them was considered. Given that both students had the same level of experience on the field meant that both opinions were valued, and one would not overrule the other.

The tunnel was in general mapped in sections of five meters at the time. This was found to provide information on a suitable level of details. A trial effort was done to map sections of 25 meters, but it was concluded that doing it five by five meters were more favorable because of the level of details it produced. The approach to map a five-meter section is described in table 23.

Table 23: The procedure of mapping a five-meter section of the tunnel

Step	Geological feature	Description
1	Joints	Joints in the rock mass were first drawn onto a mapping sheet as accurately as possible.
2	Orientation	Strike and dip of the joint planes were measured with a compass and registered on the mapping sheet.
3	Fracturing	The fracturing of the rock mass was mapped by the volumetric joint count method explained in chapter 2.3.1, and equation 2.3 was later used to calculate RQD.
4	Other features	Other important visible features of the rock mass were then registered. This includes rock type, fillings of joints, opening of joints, weathering of joint surfaces, inflow of groundwater and roughness of joints.

The mapping sheet referred to in table 23 is shown in figure 13.

The procedure described in table 23 was then repeated for every five-meter section mapped.

As no core logs were available for the studied tunnel, RQD could not be calculated using the traditional method (see chapter 2.3.1). Therefore, the volumetric joint count method was used for estimating RQD. The way it was performed during the mapping was a counting of all fractures within a representative area of 1 m^2 for the right wall, the roof/crown and the left wall. This was done for every meter of the five-meter section. Afterwards a RQD-value was calculated from the average of all the joint counts for the five-meter section.

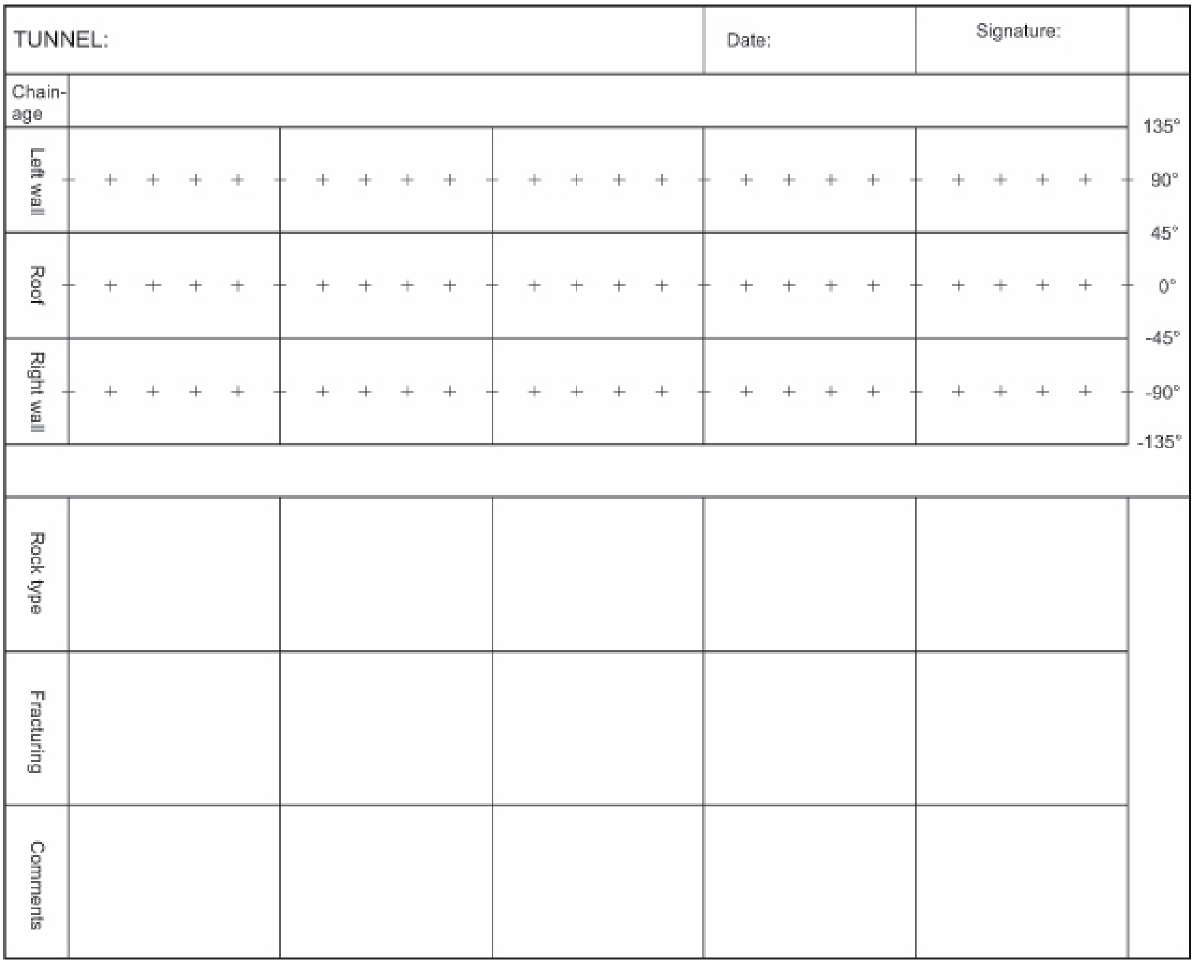

Figure 13: Sheet for geological mapping of rock mass

Other features that characterizes the rock mass were then closely examined, and noted down on the mapping sheet under comments.

3.4 Classification of rock mass

After the mapping and registration of all geological features, the work with quantifying the parameters of the Q-system and the RMR-system began. All the material collected was looked through and examined closely. If there were any uncertainty around some parameters two additional sheets were filled out. These sheets are shown in figure 14 for Q-parameters, and in figure 15 for RMR-parameters. For given chainage numbers, a value for the uncertain parameters were suggested. The sheet was then brought along for the next mapping session, where the suggested values could be either confirmed or adjusted.

Tunnel: Upper Kon Tum HPP			**Q-Parameters**	Date:	Sign:	
Chainage		Jn	Jr	Ja	Jw	
From	To	Ett sett: Jn = 2, Ett + tilfeldige: Jn = 3 To sett: Jn = 4, To + tilfeldige: Jn = 6 Tre sett: Jn = 9, Tre + tilfeldige: Jn = 12, Fire eller mer: Jn = 15	Diskontinuerlige sprekker: Jr = 4, Ru, ujevn, bølgete: Jr = 3 Glatt, bølgete: Jr = 2 Ru, ujevn, plan: Jr = 1,5	Harde mineraler: Ja = 0,75, Uomvandlet: Ja = 1, Siltig el sandig belegg: Ja = 3, Skjærdeformasjon knust berg, Ja = 4	Tørt: Jw = 1, Mellomting ca 0,75 Mange drypp: Jw = 0,66 Vannstråler: Jw = 0,5	

Figure 14: Sheet for logging Q-parameters

Tunnel: Upper Kon Tum HPP		**RMR-Parameters**	Date: Sign:	
Chainage		Spacing of discontinuities	Conditions of discontinuities	Kommentarer
From	To	> 2 meter : 20 2 - 0,6 meter: 15 0,6 - 0,2 meter: 10 20 - 6 cm: 8 < 6 cm: 5	Lengde: <1m: 6, 1-3m: 4, 3-10m: 2, 10-20m: 1, >20m: 0 Separasjon: Ingen: 6, <0,1mm: 5, 0,1-1mm: 4, 1-5mm: 1, > 5mm: 0 Ruhet: Veldig: 6, Ru: 5, Litt: 3, Smooth: 1, Glatt, 0 Fylling: Ingen: 6, Hard <5mm: 4, Hard >5mm: 2, Soft <5mm: 2, Forvitring: Ingen: 6, Litt: 5, Moderat: 3, Veldig: 1, Oppløst: 0	

Figure 15: Sheet for logging RMR-parameters

The tunnels were mainly mapped in sections of 5 meters at the time. With the mapping data as a basis for the classification, the rock mass has been classified for every 5 meter section as well.

The equations referred to in chapter 2.3.4 was then used to examine the relation between the Q-system and the RMR-system. As they are based on a high number of case records the results from this comparison is used to evaluate the validity of the information retrieved from the geological mapping.

On the basis of this exercise of comparing results from the Q-system and the RMR-system with each other, an equation which fitted the relation between Q- and RMR-values in this thesis was proposed. This information is further used to evaluate which of the two equations, presented in chapter 2.3.4, who seems to give the best relation in this case.

The collected data has been processed in the software Microsoft Excel. Simple statistical methods such as average, standard deviation and regression has been used to compare results and to present the data more clearly. The results are displayed in chapter 4.2, and can be seen in completeness in Appendix A.

3.5 Laboratory testing

For this report, laboratory testing have been performed on rock samples gathered at site in Kon Tum, Vietnam. Core samples were taken from inside the headrace tunnel and brought back to Trondheim, where they were tested at SINTEFs facilities. Because of practical reasons regarding the equipment only 5 core samples could be collected. These were taken in the rock type biotite granite, at chainage 8190. SINTEF has earlier tested the gneiss from the same tunnel for Uniaxial Compressive Strength (UCS). Therefore, it was decided that the collected samples of the biotite granite also should be tested for UCS to provide a basis for comparison between the two types of rock.

The uniaxial compressive strength is a parameter widely used in rock engineering to describe the strength characteristics of rock (Kahraman, 2001). The method for testing has been standardized by both American Society for Testing and Materials (ASTM) and International Society for Rock Mechanics (ISRM). In this thesis the methodology described by ISRM has been used (ISRM, 1979). The procedure was done in an approved compression testing machine operated by Niklas Haugen at SINTEF Building and Infrastructure in Trondheim.

The samples were cut and prepared in advance of the tests, according to ISRMs standard (ISRM, 1979).

When rock samples fail under compression the mode of failure can be characterized by the fracture planes. The number of fractures and the orientation of them decides the mode of failure. The different modes of failure are shown in figure 16.

After the compression test was conducted, the mode of failure on the tested specimens were examined.

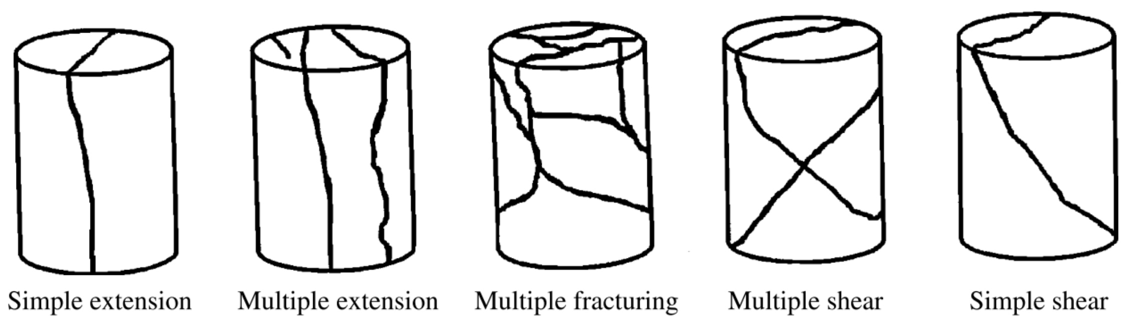

Figure 16: Modes of sample failure (Szwedzicki, 2007)

Because of the limited amount of rock samples that were possible to collect at site, it was not possible to conduct any other laboratory tests related to the topic of this thesis. It was prioritized to carry out UCS tests on the specimens to be able to compare the results with previously performed laboratory tests.

3.6 Strengths and weaknesses of the applied methods

The field investigations conducted for this thesis is performed in entireness by two students in pair. This provides the opportunity of discussion about the observed phenomena before a quantification of it is done. This method of continuously getting two opinions before notes are taken is expected to provide more correct results.

The mapping activity was done independent from all parties involved in the observed project. This means that the two individuals observing the geology has no other interest than conducting the survey in a most possible correct way.

Although the field investigation is performed in cooperation between two students, there will always be a factor of subjectivity involved in an observation of geological features. Seo et al. (2018) detects this as the work of three researchers who independent from each other mapped a tunnel. The paper concludes that geological mapping by a team of more than one person can improve mapping quality and reduce errors related to subjectivity. Even though errors because of subjectivity is reduced they cannot be totally excluded.

Geological mapping is a survey consisting of a great deal visual observation of the rock mass. In a tunnel under construction several challenges related to this was encountered. This includes dust and dirt covered walls reducing the ability to observe the rock mass closely, and making it harder to see whether joints were continuous or not. Bad lighting made it difficult to observe rock mass from a distance, and the accessibility was reduced some places because of installations and equipment needed for the ongoing work.

The headrace tunnel excavated by TBM could only be mapped in the area close to the boring machine. This was because of safety reasons at site. It meant that the distance between the mapped portal tunnel, excavated by drill and blast, and the mapped section of the bored tunnel was 9,5 km. Ideally this distance would have been shorter to be able to compare the need for rock support in the two kinds of tunnels at more or less the same location. However, when this was not possible this time, the evaluation related to rock support needs to take some more considerations regarding the geology at the specific locations.

As introduced in chapter 3.1.1, the laboratory testing includes both strengths and weaknesses. The strengths consisted of how the testing was done. Performed at SINTEF according to ISRMs standard (ISRM, 1979) it produced reliable results. The weaknesses were connected to the low number of samples that was possible to collect. This caused an issue with representativeness.

Fellows and Liu (2015) refers to the combination of quantitative and qualitative approaches as triangulated studies. The goal is to reduce the disadvantages of each individual approach, and at the same time gain the advantages of each of them. While quantitative methods provide a broad data set with information that is to be analyzed, quantitative descriptions give more in depth understanding regarding the observations made.

4 Results

Within this chapter the results of this thesis will be presented. It includes results from geological mapping conducted at Kon Tum Hydroelectric Power Project (HPP), classification of the mapped rock mass, estimated need for rock support related to the classification and lab results.

4.1 Geological mapping at Upper Kon Tum HPP

4.1.1 Mapping of the headrace tunnel excavated by TBM

The headrace tunnel was mapped in the area behind the backup rig of the tunnel boring machine. In this area, most of the tunnel contour was visible and the conditions for conducting the geological mapping was good.

The mapping of the headrace tunnel has been performed form chainage 8750 to 8100. This section consists mainly of biotite granite, but some gneiss is also present. The gneiss only exists in the first 50 meters of the mapped section, that is chainage 8750 – 8700. Here the rock is varying between granite and granitic gneiss, which indicates that this is the border between the two rock types. The rock types are shown in figure 17.

Figure 17: Biotite granite to the left (approx. 10x10 cm), and gneiss to the right (approx. 1x1 m) (Photo: O. Hobbelstad)

The fracturing was mapped by RQD, according to chapter 2.3.1 and 3.3. The volumetric joint count method was applied (Palmstrøm, 2005), and the result is shown in figure 18 beneath. In general, the rock mass was not of a particularly fractured nature, as figure 18 implies. Between chainage 8560 and 8490 no RQD-value was obtained, because the rock wall was covered with ringbeams, shotcrete and occasionally other support materials. This, however suggest that the rock was so heavily fractured that extensive rock support had to be carried out. With this in mind the RQD-value has been displayed as 0 in figure 18, even though it might not be entirely correct.

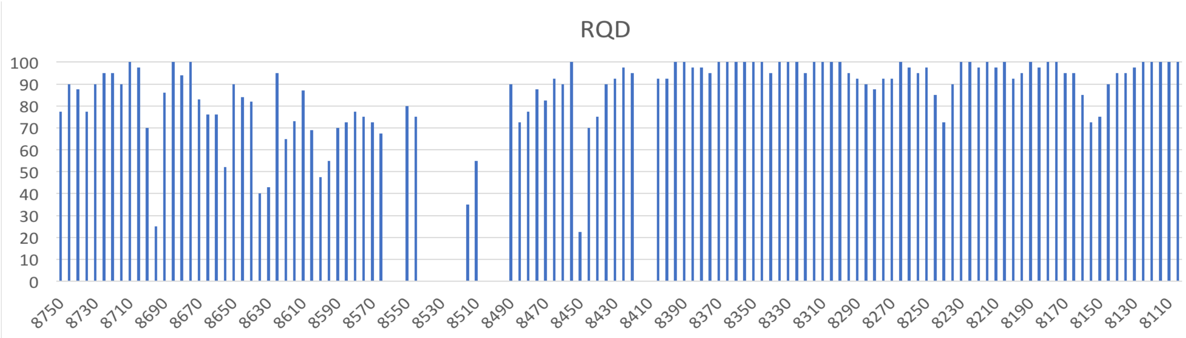

Figure 18: Registered RQD-values along the headrace tunnel

For the rest of the mapped section the RQD-values are overall quite high, with the exception of some special events. Especially for the last part of the section RQD-values are very high and even at 100 several places. This indicates that the rock mass is solid and intact.

The jointing of the rock mass was mapped by drawing the visible joints onto mapping sheets. Then the dip and strike angle were measured where it was possible and applicable. Putting series of measured strike and dip angles into stereonets revealed the number of joint sets that occurred in given sections. In figure 19 to 21 different stereonets with the poles of the planes are shown.

Strike and dip were only possible to measure for some of the joints in the selected sections shown in figure 19 to 21. This is because of the reduced accessibility to parts of the cross section and the smooth contour of the bored tunnel caused some of the joint planes not to be exposed. In the figures, the number of joints measured are given on the lower right side. This number does not indicate how many joints that existed in the given sections. It only says how many joints that it was possible to measure strike and dip angles of.

Figure 19 shows a quiet scattered pole plot. This indicates that many random joints are occurring. A concentration of poles may be seen at about N160°E. This concentration represents a set of joints for this section of the tunnel. One concentration of poles and random placed poles elsewhere indicates that there are one joint set with random joints here.

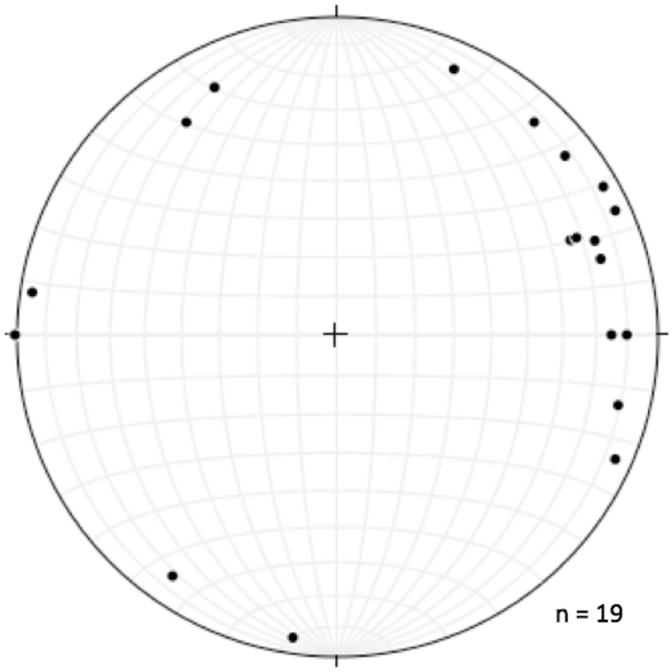

Figure 19: Poles of joint planes between chainage 8600 and 8550

Figure 20 shows a somewhat different picture. Here two joint sets may be seen at N160°E and at N210°E. Elsewhere some random joints are also present, but they do not characterize the rock mass. One may say that this section of the tunnel differs between having two marked joint sets and two joint sets with some random joints occurring.

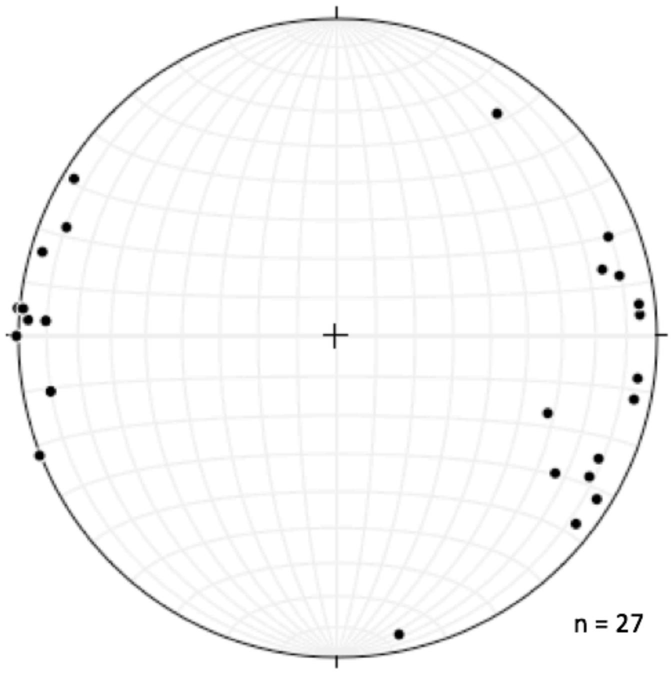

Figure 20: Poles of joint planes between chainage 8490 and 8435

Between chainage 8420 and 8280 there is one main joint set around N180°E to N190°E, as can be seen by figure 21. In addition, another joint set occurs. However, this is less frequent than the other one. In figure 21 this low frequent joint set is seen by the four poles lying around N340°E to N10°E. In total, this section of the tunnel varies between having one and two joint sets, depending on what is present in the given smaller parts of it.

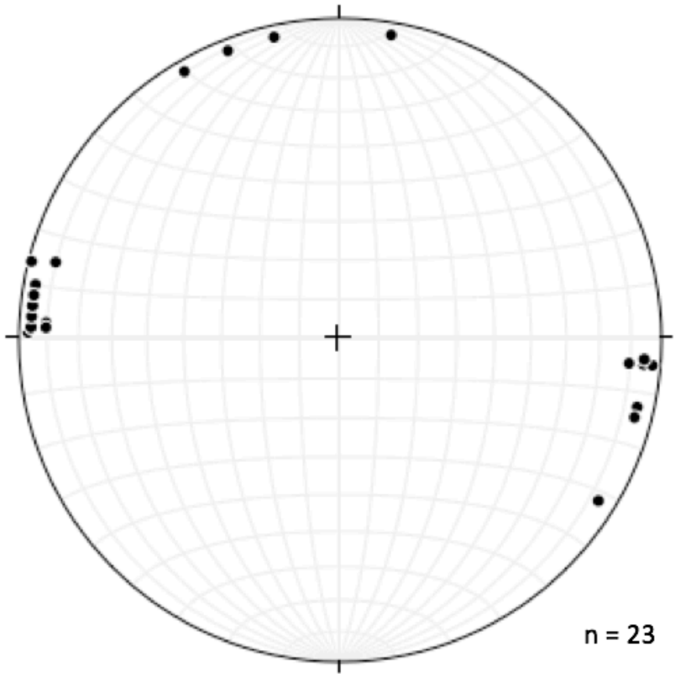

Figure 21: Poles of joint planes between chainage 8420 and 8280

A common feature for all the joints throughout the tunnel is the steep dip angles. For the joints displayed in figure 19 to 21 the dip angle is ranging from 60 to 90 degrees, with the majority between 80 and 90 degrees.

The condition of these joints has also been subject to the mapping performed. This comprises the appearance and shape of the joints, if they contain any fillings or if they have an opening, the weathering of the joint surface and the roughness of the surface. The logging of all these features cannot be easily displayed here. It is therefore referred to appendix B and C for the complete data set from the mapping.

Some of the main findings and the most occurring geological features may nevertheless be described shortly as follows in table 24.

Table 24: Main findings from geological mapping regarding conditions of discontinuities

Feature	Main findings and description
Shape	Almost all joints examined had an undulating shape. Some few random joints were of a more planar shape.
Length	Most of the joints were longer than the tunnel span width. When mapping the joints could be followed all the way around the contour of the tunnel. Discontinuous joints were also found some placed, these were generally random joints.
Fillings	Small quartz fillings dominated the mapped section. A great part of the discontinuities was also unfilled.
Openings	Mostly the discontinuities were intergrown. Some small openings were found, usually related to areas with high stress-problems.
Weathering	Both unweathered and slightly weathered joint surfaces was detected. The slightly weathered surfaces were covered by iron oxide.
Roughness	Almost all surfaces were of a very rough kind. Some random appearing joints had however a smooth surface, there were only a few of this kind.

The parameter "condition of discontinuities" that are part of the RMR-system describes this part of the rock mass in a good way. A number between 1 and 6 is given to all factors that affect the condition of the discontinuities, and then they are added together to quantify the total parameter. Figure 22 beneath shows this parameter for the entire mapped section of the headrace tunnel.

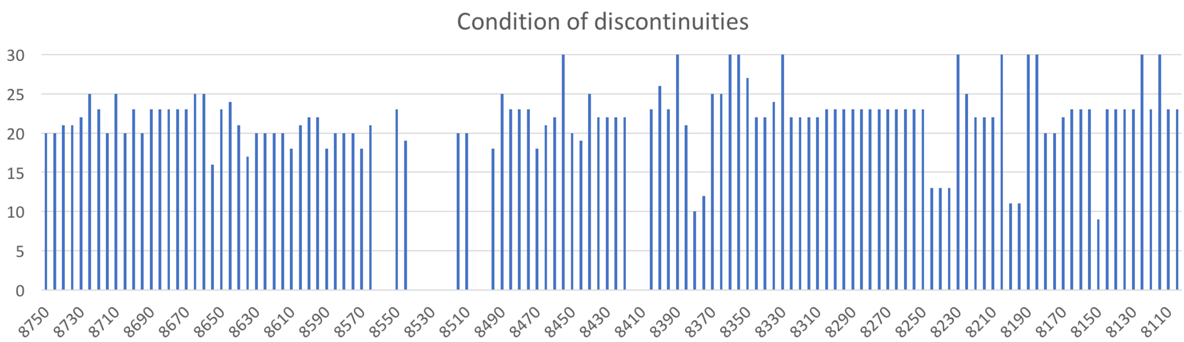

Figure 22: Conditions of discontinuities from the RMR-system

All factors described in table 24 are relatively advantageous with respect to the condition of the discontinuities. This is confirmed by figure 22, where the parameter from the RMR-system shows high ratings. The majority lies around 20 out of the possible 30.

Another important parameter that relates to the quality of the rock mass is leakage of groundwater into the tunnel. Figure 23 shows the mapping results with respect to water leakage. It shows the groundwater parameter used in the RMR-system plotted in an "opposite" fashion. That means that the higher the reading on the graph is the higher is the water leakage. It has been done this way purely to indicate where along the tunnel water leakage was found, and the magnitude of water leakage present at these locations.

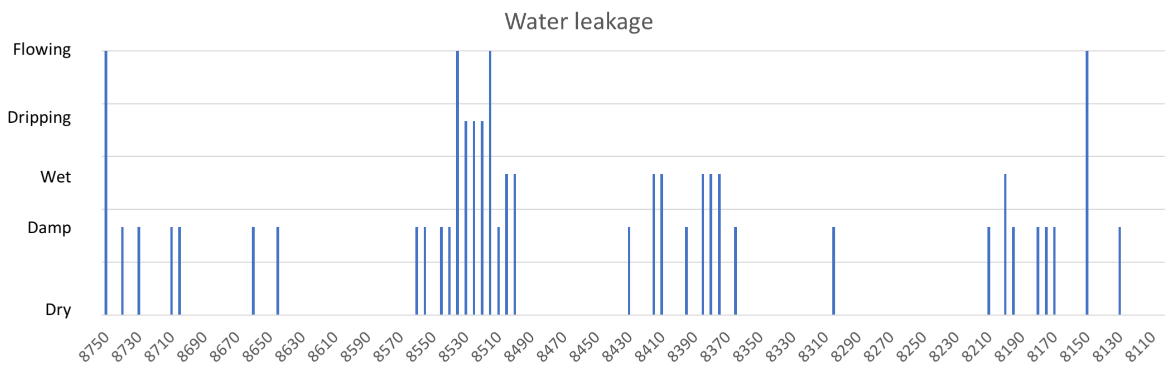

Figure 23: Water leakage related to the groundwater parameter in the RMR-system

At four locations, seen in figure 23 as the highest poles, flowing water leaking into the tunnel was detected. Readings one step lower than this indicates dripping water conditions, while the two lowest readings indicates wet or damp conditions respectively. The height of the poles does not express the volume of leaked water, it only indicates where the leakage is and the magnitude of leakage related to the RMR-system. Areas in between poles, in figure 23, means completely dry conditions. Which make up the majority of the mapped section.

During the mapping, already installed rock support were also registered. Some of this rock support covers the tunnel contour, which made it difficult to map the geology. For certain areas, the only thing possible to map was in fact the type and amount of rock support. Table 25 sums up installed rock support with reference to chainage numbers.

Table 25: Installed rock support in headrace tunnel excavated by TBM

Chainage	Type of rock support
8598 – 8597	3 rock bolts in a spotted pattern and ca 2 m^2 mesh
8560 – 8555	Ringbeams c/c 0,9 m, reinforcement and thick layers of shotcrete
8538 – 8534	Ringbeams c/c 0,9 m, reinforcement and thick layers of shotcrete
8534 – 8531	Shotcrete with mesh reinforcement
8530 – 8526	Ringbeams c/c 0,9 m, reinforcement and thick layers of shotcrete
8523 – 8521	Ringbeams c/c 0,9 m, reinforcement and thick layers of shotcrete
8521 – 8515	Shotcrete with mesh reinforcement
8505 – 8493	Shotcrete covers roof and walls, with mesh reinforcement in roof
8448 – 8445	Shotcrete with mesh reinforcement
8413 – 8407	Zone of weakness supported with shotcrete and mesh reinforcement

The complete data set retrieved from the geological mapping of the headrace tunnel may be seen in appendix B.

4.1.2 Mapping of the portal tunnel excavated by drill and blast

The first 40 meters of the portal was covered in shotcrete, and not possible to map. The mapping therefore started right after the shotcreted area. This point is referred to as chainage 0 for simplicity reasons. From here on, 105 meters of the portal tunnel was mapped in consideration of geological features and installed rock support.

The rock is a foliated gneiss. It is characterized by conjoined joints and fractures without any fillings. It is moist throughout the section with severe dripping of water several places. Many of the joint surfaces has been slightly weathered, and they are slightly rough to smooth in their appearance.

For the first 55 meters of the mapped section there was two clearly marked joint sets. One parallel to the foliation of the gneiss, and one perpendicular to it (See figure 24). In addition, there was some random joints appearing. A couple of this random joints were very long and went almost parallel to the tunnel axis, with a steep dip. The dip was not possible to measure because this joint appeared in the crown, but it was estimated to 60-70°. The joint set perpendicular to the foliation has a strike almost perpendicular to the tunnel axis, and has a steep dip against the tunnel drive direction. As the foliation is more or less perpendicular to the tunnel axis the joint set which is parallel to it, is also perpendicular to the tunnel axis. This joint set has a more moderate dip of around 40°, with the tunnel drive direction.

Figure 24: The jointing of the rock mass in the portal tunnel (Photo: O. Hobbelstad)

Some meters into the tunnel, about 100, the rock becomes more massive, and only one joint set is appearing. This is the joint set perpendicular to the foliation with dip against the tunnel drive direction, and the spacing is larger than before.

Rock support in the mapped section of the portal tunnel consists of rock bolting. Spotted rock bolting was done around joints and fractures, and systematically rock bolting had been done for a few areas. Table 26 sums up the installed rock support for the portal tunnel.

Table 26: Installed rock support in portal tunnel excavated by drill and blast

Chainage	Type of rock support	Bolts per meter
0 – 25	Spotted bolting, 28 registered bolts	1,12
25 – 45	Both systematic and spotted bolting, 36 registered bolts	1,8
45 - 105	Spotted bolting, 35 registered bolts	0,58

The sections listed in table 26 have been divided into groups with similar rock support measures. Within these sections approximately the same bolting frequency were registered.

The complete data set from the geological mapping of the portal tunnel may be seen in appendix C.

4.2 Rock mass classification

After having geologically mapped the rock mass it has been classified using two classification systems. These systems are Barton's Q-system and Bieniawski's RMR-system. They have been applied independent from each other, but with the same basis of information.

4.2.1 The Q-system

The results from the classification done with the Q-system are shown in the figures below. Figure 25 shows a diagram of the Q-values plotted against the chainage numbers.

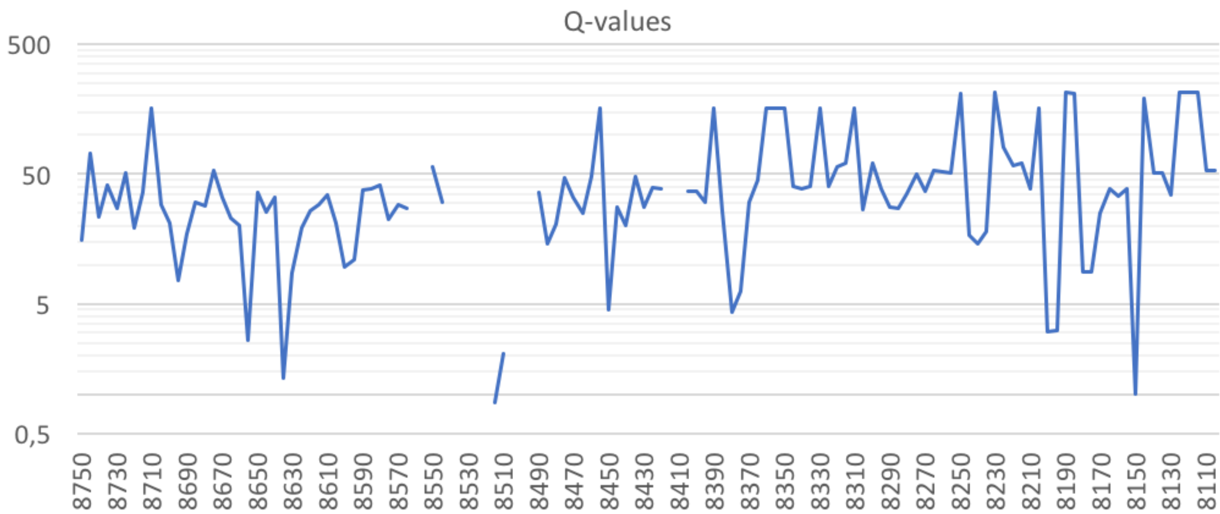

Figure 25: Registered Q-values

Figure 26 shows a pie chart of the rock mass classes the Q-values results in.

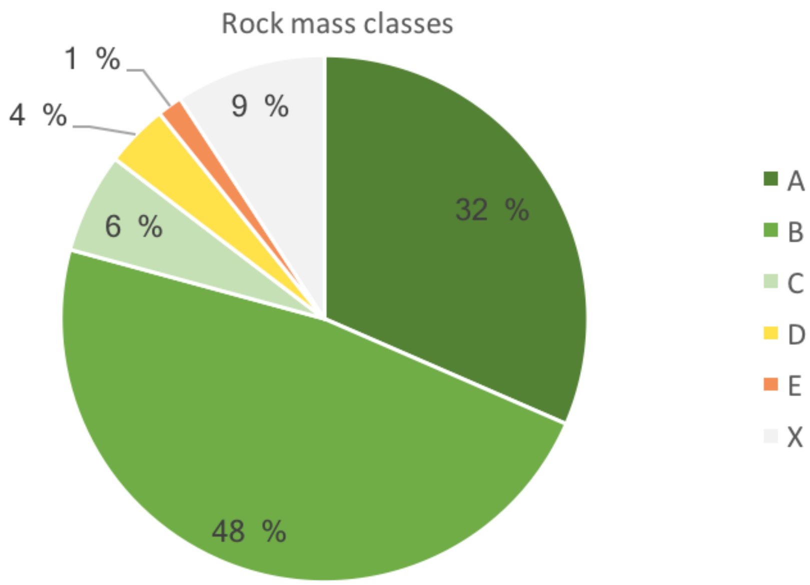

Figure 26: Rock mass classes using the Q-system

For more detailed information about the parameters which make up the Q-system see appendix A. Values for all parameters for every section of the mapped tunnel is given there.

4.2.2 The RMR-system

The results from the classification done with the RMR-system is shown in the figures below. Figure 27 shows the calculated RMR-values plotted against the chainage numbers.

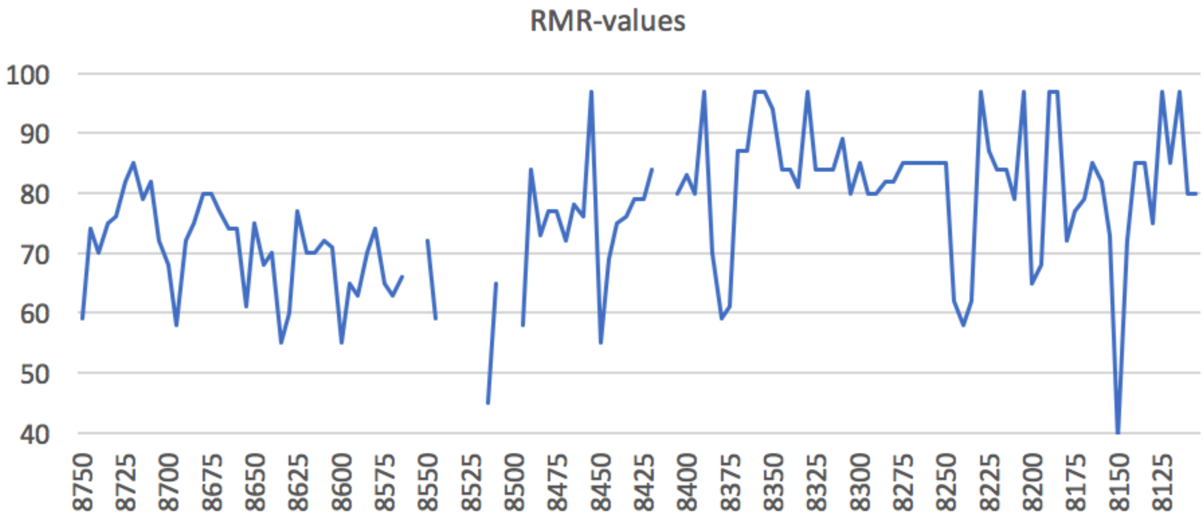

Figure 27: Registered RMR-values

Figure 28 shows the rock mass classes the RMR-values results in.

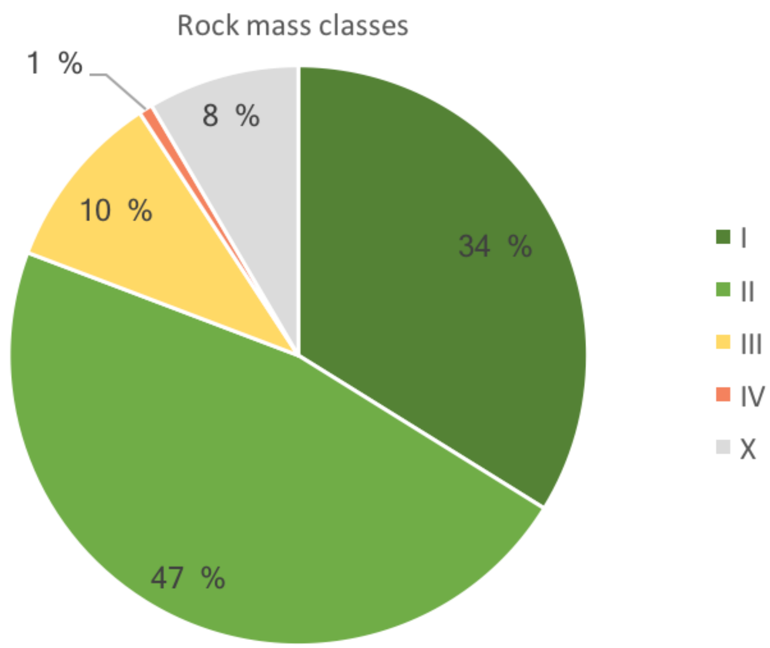

Figure 28: Rock mass classes using the RMR-system

For more detailed information about the parameters in the RMR-system and the calculation of the RMR-values, see appendix A.

4.2.3 Estimated classification for areas covered with rock support

Some areas of the tunnel were covered by extensive rock support. This includes both ringbeams and shotcrete. The rock mass was not visible here and it was not possible to carry out any mapping. Based on the amount of rock support installed in these sections some estimated values for Q and RMR has been found.

It is necessary to clarify that these estimated values are only made to fill in the gaps between the mapped values. It will be used as a basis for further discussion, but no other results or possible further work should take these values into account.

The Q-system

In the sections where ringbeams and thick layers of shotcrete were used it is estimated that this corresponds to reinforced ribs of sprayed concrete category II (RRS II) in figure 6. This is based on the spacing of the ringbeams and the thickness of the shotcrete. In this area of the rock support chart, in figure 6, the Q-value is between 0,01 and 0,1. This again corresponds to rock mass class F, extremely poor rock mass.

In sections where the rock was covered by shotcrete reinforced with mesh and occasional rock bolts, it is estimated that it corresponds to the area right above RRS I in figure 6. Here the Q-values ranges between 0,4 and 1, and the rock mass class is E very poor rock mass.

The estimated Q-values are illustrated in figure 29 beneath for the relevant chainages of the tunnel.

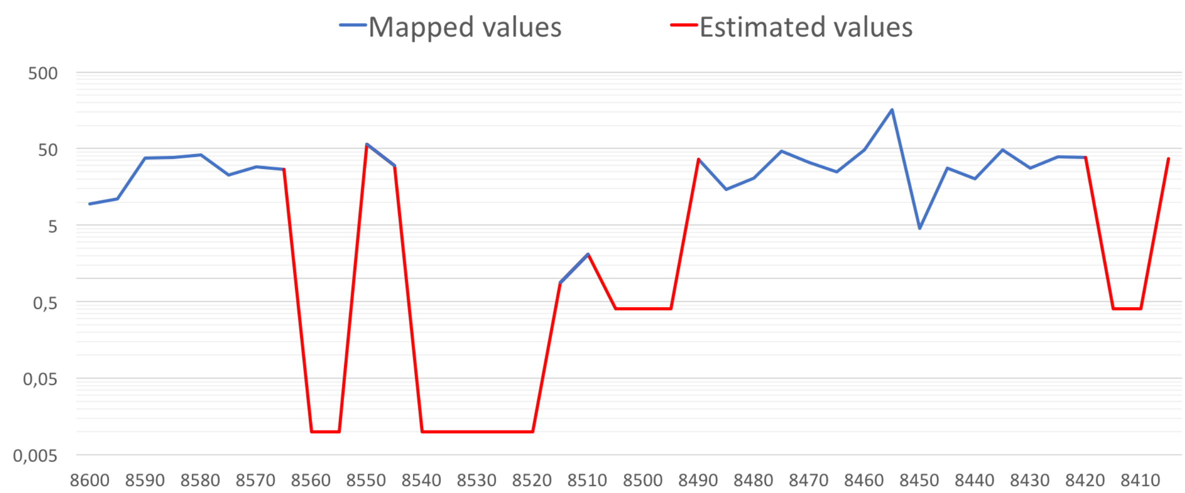

Figure 29: Estimated Q-values based on the consumption of rock support

The RMR-system

Using the same procedure to estimate RMR-values, the following results is achieved:

For the sections with ringbeams and thick layers of shotcrete the RMR-values is estimated to be on the border between rock mass class IV and V. This Results in RMR-values around 20.

For the sections with thinner layers of shotcrete and some mesh reinforcement, the RMR-values is estimated to be in the area of the border between rock mass class III and IV. This corresponds to RMR-values around 40. Estimated RMR-values are illustrated in figure 30.

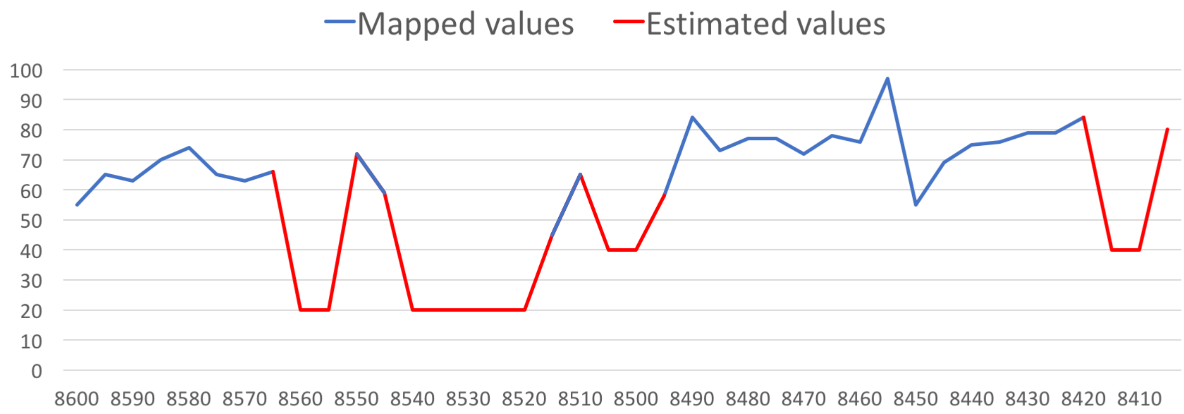

Figure 30: Estimated RMR-values based on the consumption of rock support

4.2.4 Relation between Q and RMR

Beneath, in figure 31, the results of the rock mass classification with both the Q-system and the RMR-system is plotted in the same diagram. This has been done to show the relation between the two systems. The chart has two vertical axes. The one on the left side relates to the Q-system with its logarithmic scale. The vertical axis to the right is of a linear scale and is related to the RMR-values.

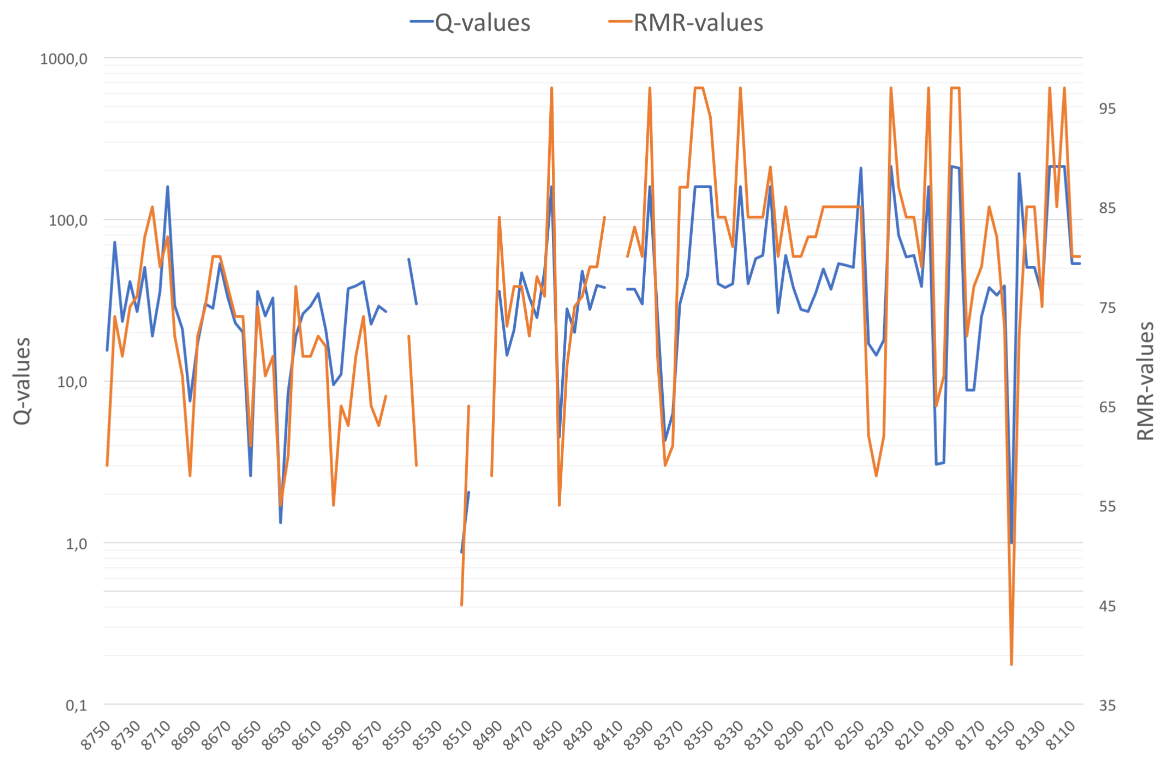

Figure 31: Q-values plotted against RMR-values

Using the information given in chapter 2.3.4, an exercise has been done to compare the results from the two rock mass classification system used here.

In figure 32, Q-values are plotted against RMR-values for the same sections of the tunnel. The best fitting line for this data series is included, and shown by the blue line. For comparison reasons, the line representing Bieniawski's relation between Q and RMR has also been included here. This is equation 2.6 and corresponds to the orange line in figure 32.

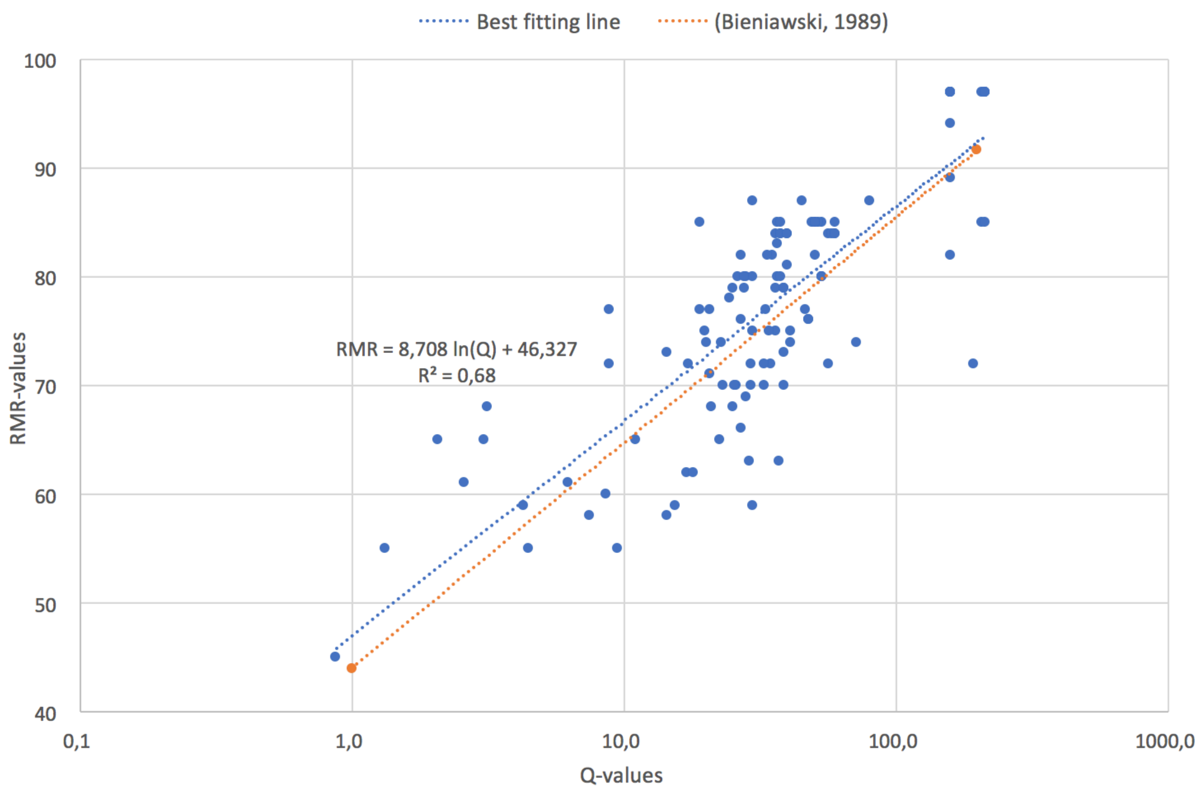

Figure 32: Registered Q- and RMR-values plotted against each other with best fitting line

Figure 33 and 34 shows the deviation between registered RMR-values from the geological mapping and RMR-values calculated on the basis of registered Q-values. The deviation is displayed by the magnitude on the vertical axis, while the horizontal axis displays the registered RMR-values. RMR-values has been calculated using two equations describing the relationship between the Q-system and the RMR-system. Equation 2.6, by Bieniawski is represented by the blue dots in figure 33, and equation 2.7 by Barton is represented by the orange dots in figure 34. In addition, trendlines for the two data series have been included in the charts.

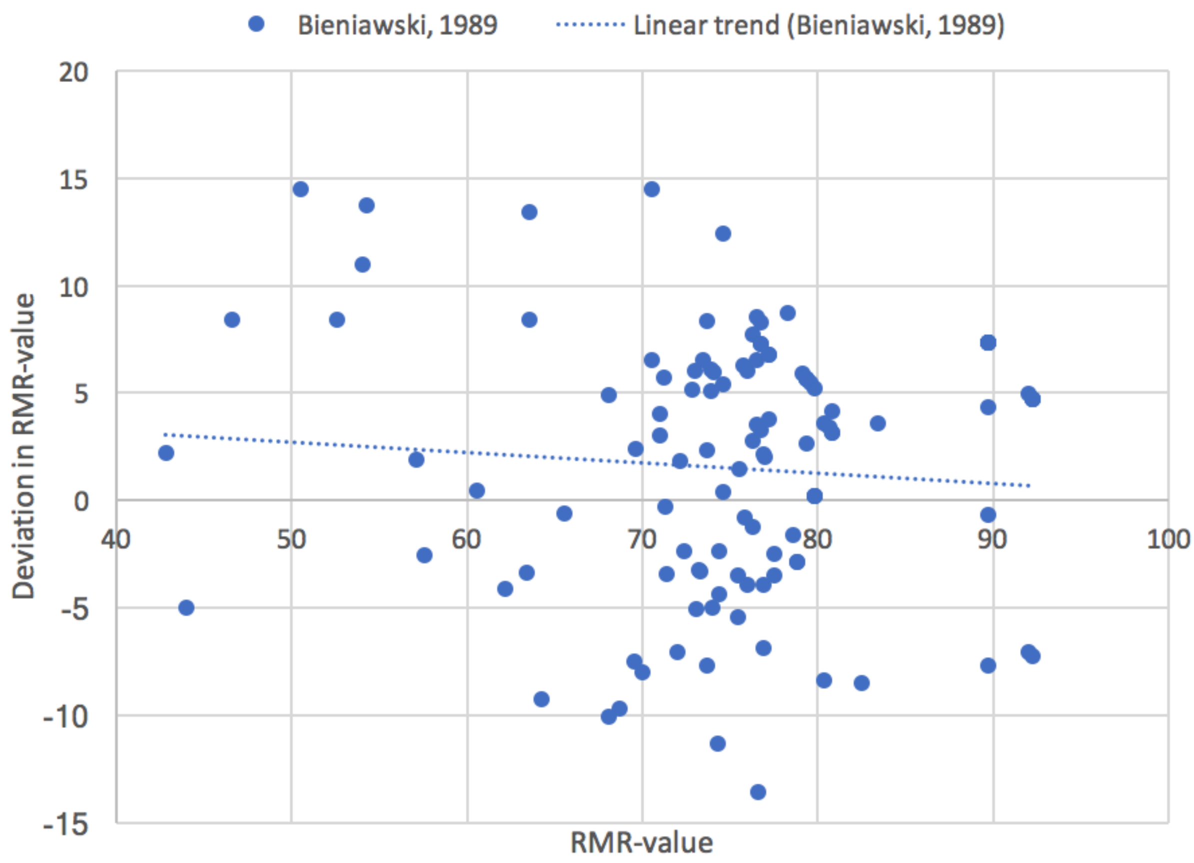

Figure 33: Deviation between registered and calculated RMR-values using equation 2.6

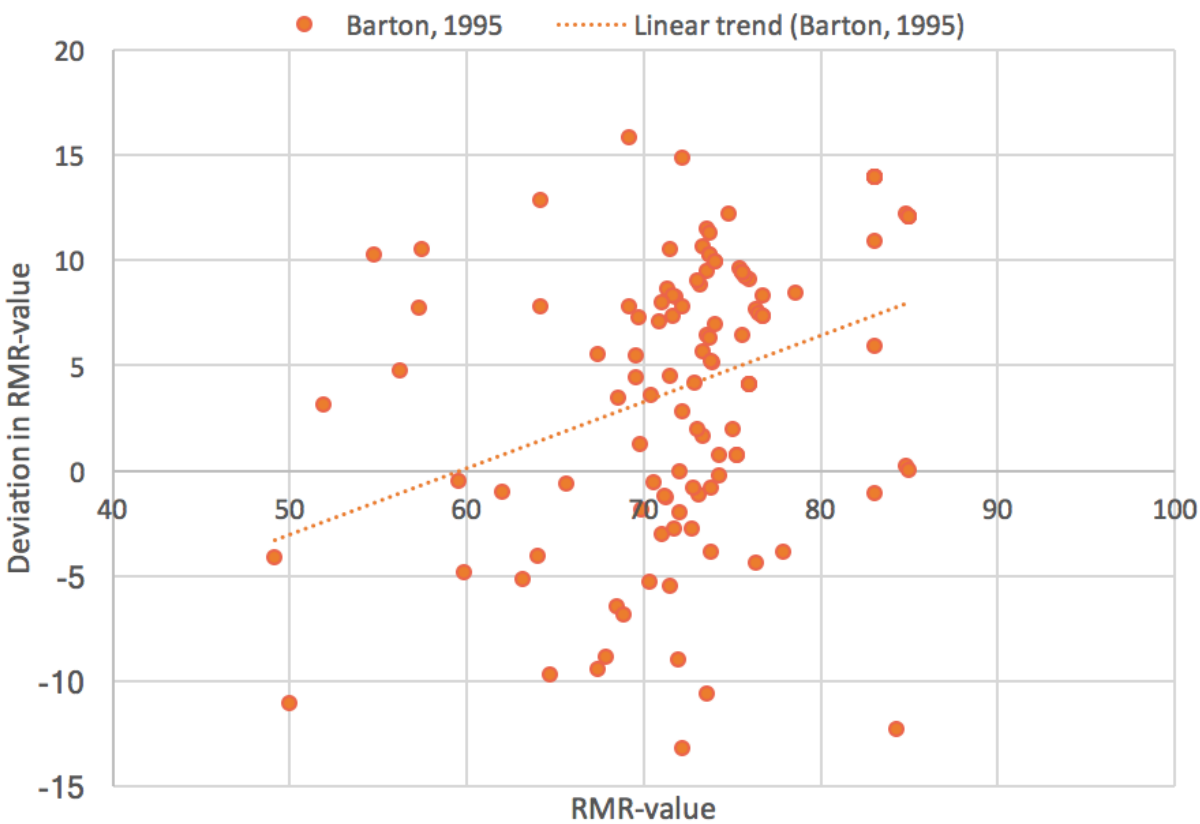

Figure 34: Deviation between registered and calculated RMR-values using equation 2.7

4.3 Rock support

In this section, a reasoning for the needed quantity and type of rock support is given. It is based on the two rock mass classification systems, Q and RMR. Quantity and type of rock support is decided for both the headrace tunnel, excavated by TBM, and the portal tunnel, excavated by drill and blast.

For the calculation of rock support the following assumptions is used as a basis.

Figure 35 shows how the cross section has been divided into zones, named: crown, sides and invert. This sectioning will be used to calculate amounts of rock support where recommendations is given to specific parts of the cross section.

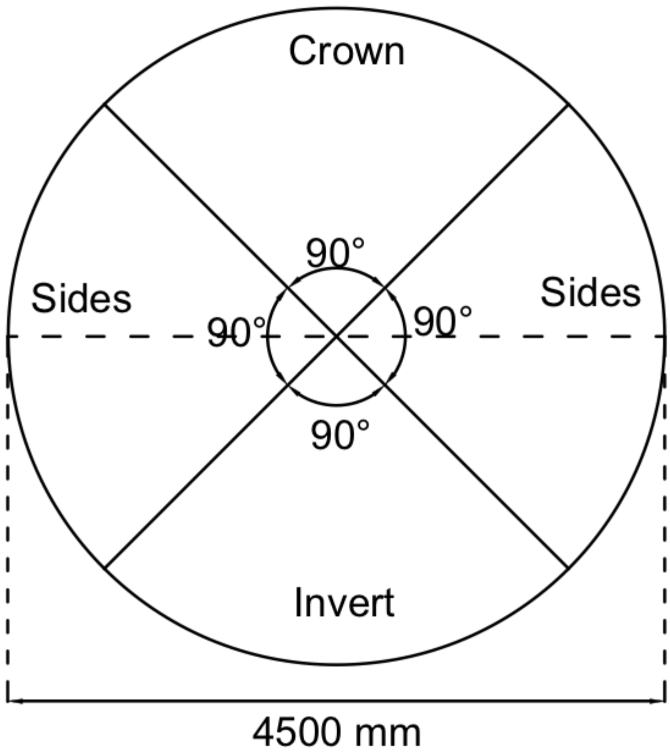

Figure 35: Sectioning of the tunnel profile

Norwegian Public Roads Administration's (NPRA) handbook R761 requires minimum 80 mm thickness of shotcrete when it is used as rock support (NPRA, 2015). The reason for this requirement is for an arc of shotcrete to be self-bearing it needs to be at least 80 mm thick. It normally concerns road tunnels, but the handbook is frequently used as guideline for other kinds of tunnels as well. With this in mind, a minimum thickness of 80 mm for shotcrete will be used in this thesis.

4.3.1 Headrace tunnel, TBM

The Q-system

Using equation 2.5 to calculate the equivalent dimension for the TBM tunnel the following value is retrieved:

$$D_e = \frac{Span, diameter\ or\ height}{ESR} = \frac{4{,}5\ m}{1{,}6} = 2{,}8$$

Figure 36 shows the rock support chart of the Q-system with the line of the equivalent dimension drawn as the red line.

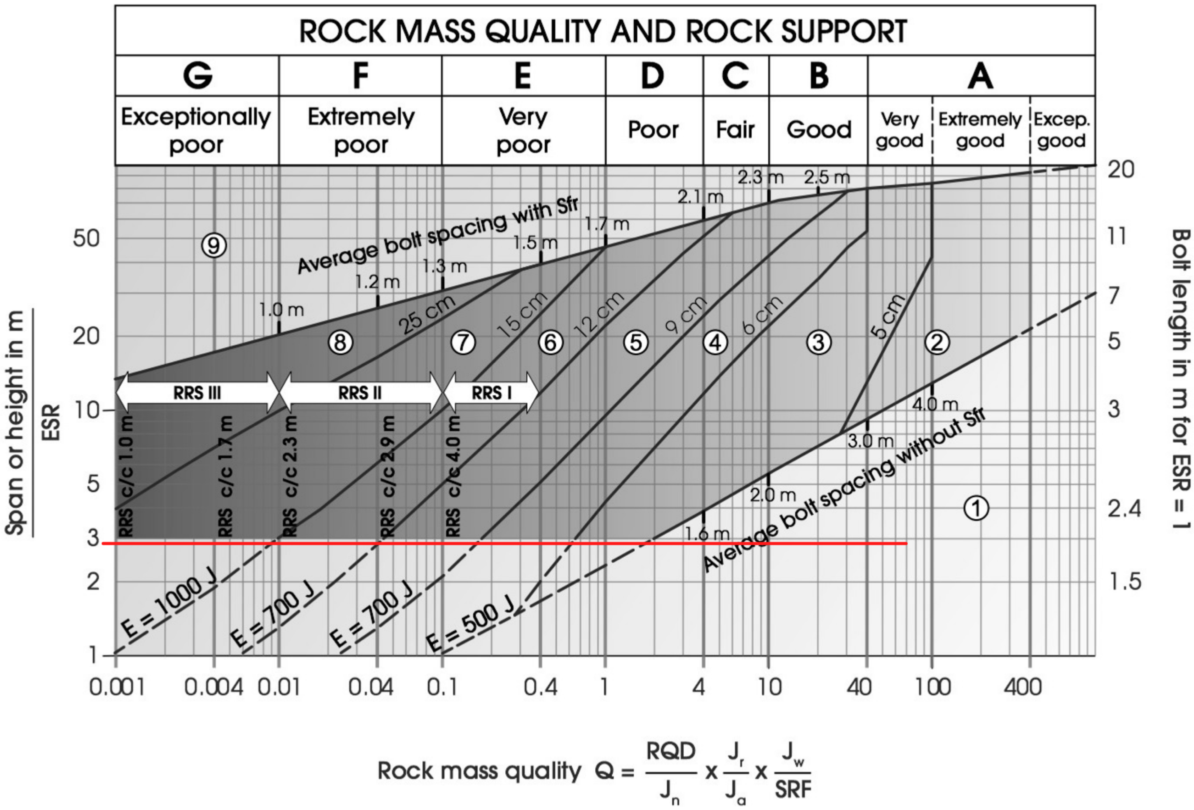

Figure 36: Rock support chart of the Q-system (NGI, 2015), with an equivalent dimension of 2,8

Figure 36 indicates that rock support is needed when the Q-value is lower than 2. Table 27 sums up the suggested need for rock support related to given ranges of Q-values.

Table 27: Recommended rock support with equivalent dimension of 2,8

Q-values	Support category	Type of rock support
1000 – 2	1	No rock support, only spotted rock bolting
2 – 0,6	3	Rock bolting with spacing 1,7 m and 6 cm shotcrete
0,6 – 0,15	4	Rock bolting with spacing 1,5 m and 9-6 cm shotcrete. Below Q = 0,4 heavier support may be needed such as RRS I
0,15 – 0,04	5	Rock bolting with spacing 1,3 m and 12-9 cm shotcrete. Below Q = 0,1 heavier support may be needed such as RRS II
0,04 – 0,009	6	Rock bolting with spacing 1-1,2 m and 15-12 cm shotcrete. Below Q = 0,1 heavier support may be needed such as RRS II
0,009 – 0,001	7	Rock bolting with spacing 1 m and 25-12 cm shotcrete. Below Q = 0,01 heavier support is needed, for instance RRS III

Amounts of rock support is found by comparing mapped Q-values to the support categories given in table 27. In this case the result is that 570 meters of the mapped headrace tunnel has Q-values higher than 2. This part of the tunnel then ends up in support category 1, which means only spotted bolting is required. Further is 20 meters of the tunnel mapped with a Q-value lower than 2, and higher than 0,6. These sections ends up in support category 3, with systematic bolting and some shotcrete as shown in table 27. The last 60 meters of the headrace tunnel was the part that was covered with rock support, and therefore not possible to map. Based on the estimated values provided in chapter 4.2.3 support categories may be indicated for these sections as well. 25 meters were covered by shotcrete reinforced with wire mesh. This is estimated to be in support category 4. While the parts of the tunnel that were supported with ringbeams and thick layers of shotcrete may correspond to support category 6. Table 28 sums up the total consumption of support material proposed by the Q-system.

Table 28: Consumption of rock support material for headrace tunnel using the Q-system

Support category	Meters of tunnel	Percentage	Bolts (quantity)	Shotcrete* (m^3)	Steel sets (quantity)
1	570	88 %	57	0	0
3	20	3 %	71	17	0
4	25	4 %	117	21	0
6	35	5 %	371	56	39
Total	650	100 %	616	94	39

*Minimum thickness 80 mm

The RMR-system

The RMR-system relates recommendations for rock support to the rock mass class. The calculations here is done on the basis of figure 28, table 17 and chapter 2.3.3, and is shown in table 29 beneath.

Table 29: Recommended rock support in the headrace tunnel using mapped values for RMR

Rock mass class	Meters of tunnel	Recommended rock support
I	220	No support, only spotted bolting
II	305	Local support where needed. Bolts with 2,5 m spacing, and 50 mm shotcrete
III	65	Systematic bolting with 1,5 – 2 m spacing and wire mesh in crown. 50 – 100 mm shotcrete in crown and 30 mm in sides.
IV	5	Systematic bolting with 1 – 1,5 m spacing and wire mesh in crown. 100 – 150 mm shotcrete in crown and 100 mm in sides. Steel sets spaced 1,5 meters where required.
V	0	Systematic bolting with 1 – 1,5 m spacing and wire mesh in crown. 150 – 200 mm shotcrete in crown and 150 mm in sides. Steel sets spaced 0,75 meters.

Table 30 sums up the consumption of rock support materials using the RMR-system to estimate rock support. Estimated RMR-values taken from chapter 4.2.3 has also been included here.

Table 30: Consumption of rock support material for headrace tunnel using the RMR-system

Rock mass class	Meters of tunnel	Percentage	Bolts (quantity)	Shotcrete* (m^3)		Mesh (m^2)	Steel sets (quantity)
				Crown	Sides		
I	220	34 %	22	0	0	0	0
II	305	47 %	61	0	0	0	0
III	65	10 %	225	18	37	230	0
IV	25	4 %	170	11	18	88	0
V	35	5 %	237	22	37	124	47
Total	650	100 %	715	143		442	47

*Minimum thickness 80 mm

4.3.2 Portal tunnel, drill and blast

The Q-system

Using equation 2.5 to find the equivalent dimension gives:

$$D_e = \frac{Span, diameter\ or\ height}{ESR} = \frac{7{,}0\ m}{1{,}6} = 4{,}4$$

This D_e results in the following rock support chart shown in figure 37, with the equivalent dimension expressed as the red line.

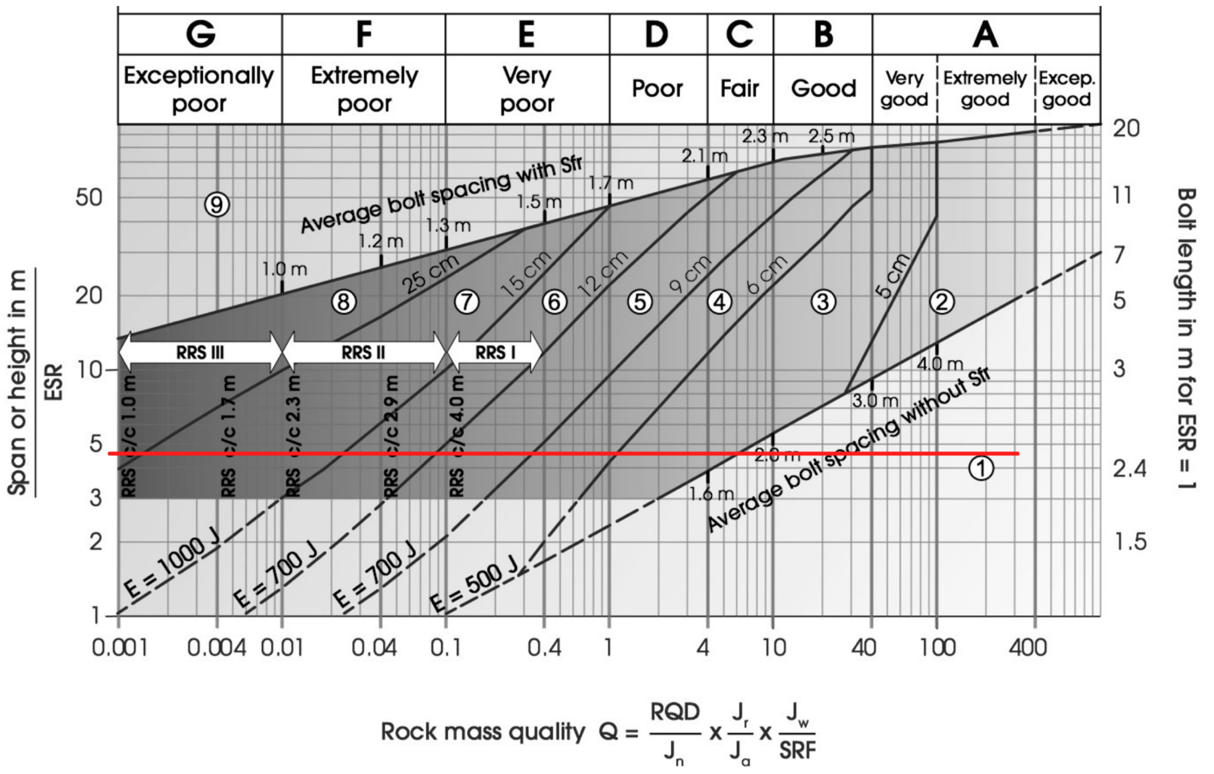

Figure 37: Rock support chart of the Q-system (NGI, 2015), with an equivalent dimension of 4,4

Figure 37 indicates that rock support is needed when the Q-value is lower than 6. Table 31 sums up the suggested need for rock support related to given ranges of Q-values.

Table 31: Recommended rock support with equivalent dimension of 4,4

Q-values	Support category	Type of rock support
1000 – 6	1	No rock support, only spotted rock bolting
6 – 1,1	3	Rock bolting with spacing 1,7 m and 6 cm shotcrete
1,1 – 0,35	4	Rock bolting with spacing 1,5 m and 9-6 cm shotcrete. Below Q = 0,4 heavier support may be needed such as RRS I
0,35 – 0,09	5	Rock bolting with spacing 1,3 m and 12-9 cm shotcrete. Below Q = 0,1 heavier support may be needed such as RRS II
0,09 – 0,025	6	Rock bolting with spacing 1-1,2 m and 15-12 cm shotcrete. Below Q = 0,1 heavier support may be needed such as RRS II
0,025 – 0,003	7	Rock bolting with spacing 1 m and 25-12 cm shotcrete. Below Q = 0,01 heavier support may be needed such as RRS III
0,003 – 0,001	8	Rock bolting with spacing 1 m and >25cm shotcrete. Heavier support is needed such as RRS III

The consumption of rock support estimated by using the Q-system is found by comparing the recommendations in table 31 to mapped Q-values. In total, 105 meters of the portal tunnel were examined. Among this, 50 meters had a recorded Q-value lower than 6, and higher than 1,1. In table 31 this means support category 3. The rest of the portal tunnel had Q-values higher than 6, which means that it is in support category 1. Table 32 sums up the consumption of rock support by using the Q-system for the portal tunnel.

Table 32: Consumption of rock support material for portal tunnel using the Q-system

Support category	Meters of tunnel	Percentage	Bolts (quantity)	Shotcrete* (m^3)
1	55	52 %	32	0
3	50	48 %	431	78
Total	105	100 %	463	78

*Minimum thickness 80 mm

The RMR-system

Using the same approach as done for the RMR-system in chapter 4.3.1, by deciding rock support based on rock mass class the recommendations shown in table 33 has been found.

Table 33: Recommended rock support in the portal tunnel using RMR

Rock mass class	Meters of tunnel	Recommended rock support
I	20	No support, only spotted bolting
II	25	Local support where needed. Bolts with 2,5 m spacing, and 50 mm shotcrete
III	60	Systematic bolting with 1,5 – 2 m spacing and wire mesh in crown. 50 – 100 mm shotcrete in crown and 30 mm in sides.
IV	0	Systematic bolting with 1 – 1,5 m spacing and wire mesh in crown. 100 – 150 mm shotcrete in crown and 100 mm in sides. Steel sets spaced 1,5 meters where required.

Table 34 beneath sums up the consumption of rock support when the RMR-system is used to estimate rock support for the portal tunnel.

Table 34: Consumption of rock support material for portal tunnel using the RMR-system

Rock mass class	Meters of tunnel	Percentage	Bolts (quantity)	Shotcrete* (m^3)		Mesh (m^2)
				Crown	Sides	
I	20	19 %	12	0	0	0
II	25	24 %	77	19	20	0
III	60	57 %	380	57	60	565
Total	105	100 %	469	156		565

*Minimum thickness 80 mm

4.3.3 Comparison of excavation methods of a water tunnel related to rock support

The utilization of a bored cross section is very good in water tunnels. The smooth contour lowers the friction between the rock and the water, and the circular shape makes as much water as possible pass through per square meter. An interesting angle to the topic would be to examine the impact on the amount of rock support if the tunnel were to be excavated with drill and blast instead of by TBM. Bruland (2013) has looked into the relation between cross sections excavated by drill and blast and by TBM for water tunnels. Figure 38 shows this relation, with the diameter of the TBM on the left vertical axis, and the cross section of a drilled and blasted tunnel on the horizontal axis. The lines which is increasing towards the right corresponds to the equivalent cross section that transports the same volume of water.

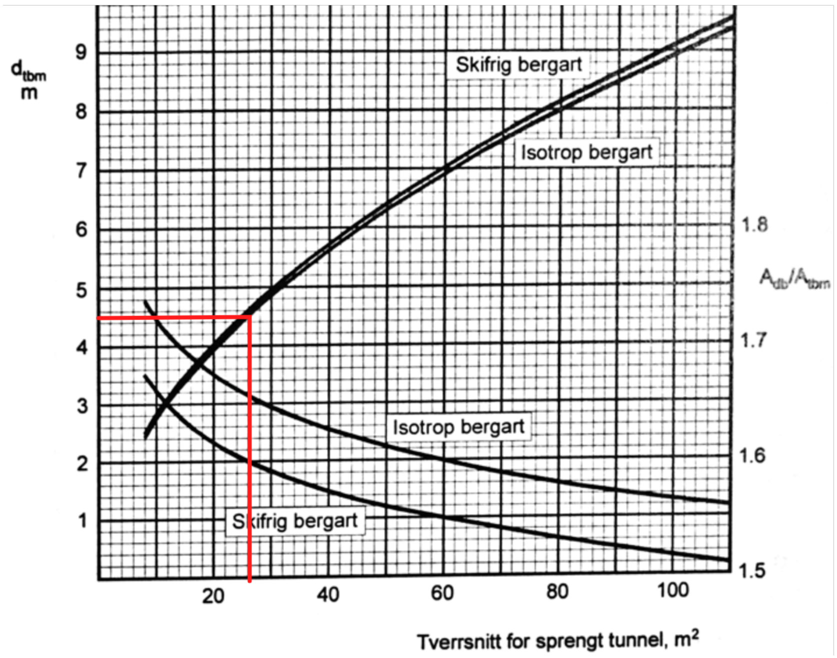

Figure 38: Equivalent cross sections for water tunnels (Bruland, 2013)

The red line in figure 38 shows that for a TBM-tunnel with diameter of 4,5 meters, one needs a cross section of 26 m^2 if the tunnel had been excavated with drill and blast. Assuming that the cross section would have a regular shape with horizontal invert, vertical walls and a curved crown it would need to be around 5,7 meters high. Using equation 2.5 again to calculate the equivalent dimension with this height one gets:

$$D_e = \frac{Span, diameter\ or\ height}{ESR} = \frac{5,7\ m}{1,6} = 3,6$$

An equivalent dimension of 3,6 then results in a rock support chart shown in figure 39.

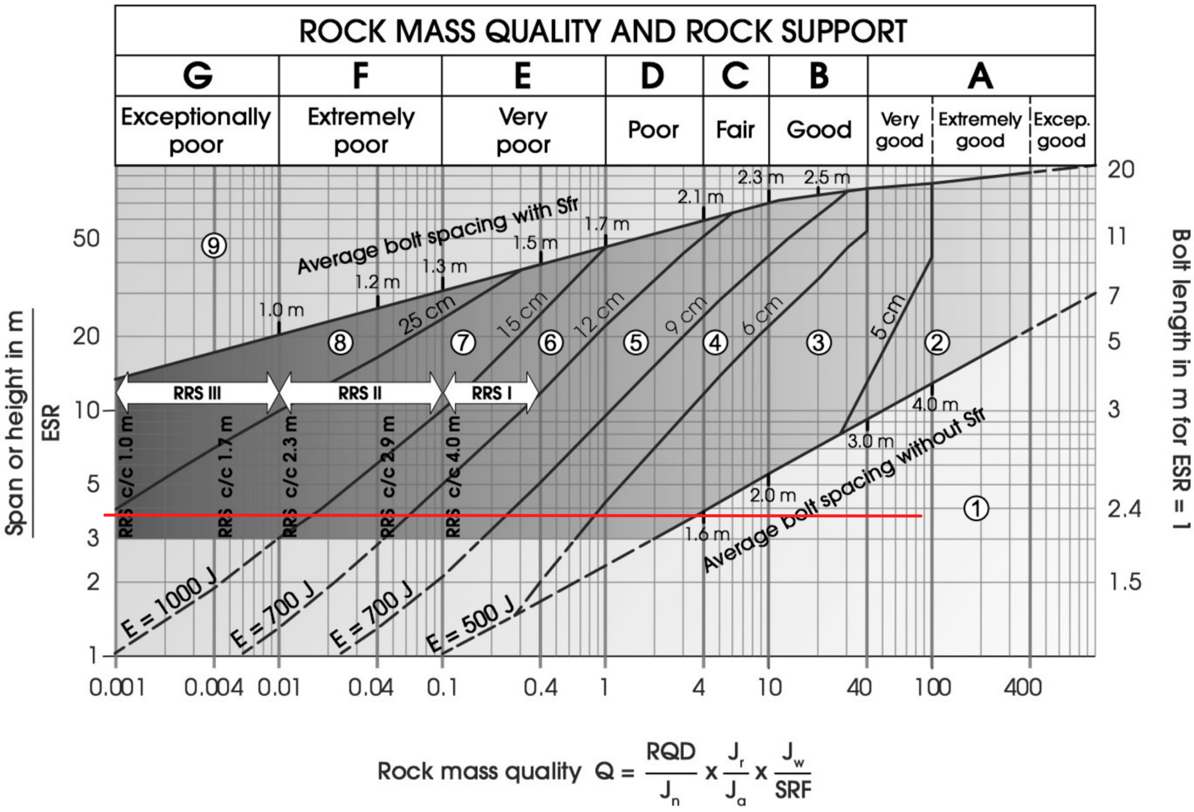

Figure 39: Rock support chart of the Q-system (NGI, 2015), with an equivalent dimension of 3,6

As can be seen from figure 39, the Q-system suggests that rock support is needed when the Q-value is lower than 3,5. This increased Q-value for recommended use of rock support causes 15 meters of the headrace tunnel to fall in under support category 3. These 15 meters were in support category 1 for a bored, circular cross section. Apart from this adjustment the rest of the tunnel is in the same support categories, related to the Q-system, as described in chapter 4.3.1 for a bored tunnel.

4.3.4 Costs of rock support

Table 35 sums up costs of support material which will be used in further calculations.

Table 35: Costs of rock support

Type	Combinational bolts	Shotcrete (m^3)	Mesh (m^2)	Steel sets/ ringbeams
Cost (NOK)	750	1500	56	10350

The prices listed in table 35 is based on a selection of contracts from Norwegian tunneling projects. These contracts cannot be referred to because of contractual agreements. Limited access has however been given for the elaboration of this thesis.

The Q-system

Table 36 shows the calculations of total costs of rock support using the Q-system for the TBM-excavated headrace tunnel. Information from table 28 and 35 has been used.

Table 36: Costs of rock support in the mapped area of the headrace tunnel using the Q-system

Type of support	Bolts	Shotcrete (m^3)	Steel sets
Quantity	616	94	39
Price per unit (NOK)	750	1500	10.350
Costs (NOK)	462.000	141.000	403.650
Total costs (NOK)		1.006.650	

Table 37 shows the calculations of total costs of rock support using the Q-system for the drill and blast-excavated portal tunnel. Information from table 32 and 35 has been used.

Table 37: Costs of rock support in the mapped area of the portal tunnel using the Q-system

Type of support	Bolts	Shotcrete (m^3)
Quantity	463	78
Price per unit (NOK)	750	1500
Costs (NOK)	347.250	117.000
Total costs (NOK)	464.250	

If the headrace tunnel was excavated by drill and blast according to chapter 4.3.3, the amounts and costs of rock support would be as shown in table 38.

Table 38: Costs of rock support for a drill and blast excavated headrace tunnel using the Q-system

Type of support	Bolts	Shotcrete (m^3)	Steel sets
Quantity	882	107	39
Price per unit (NOK)	750	1500	10.350
Costs (NOK)	661.500	160.500	403.650
Total costs (NOK)		1.225.650	

The RMR-system

Table 39 shows the calculations of total costs of rock support using the RMR-system for the TBM-excavated headrace tunnel. Information from table 30 and 35 has been used.

Table 39: Costs of rock support in the mapped area of the headrace tunnel using the RMR-system

Type of support	Bolts	Shotcrete (m^3)	Mesh (m^2)	Steel sets
Quantity	715	143	442	47
Price per unit (NOK)	750	1500	56	10.350
Costs (NOK)	536.250	214.500	24.752	486.450
Total costs (NOK)	**1.261.952**			

Table 40 shows the calculations of total costs of rock support using the RMR-system for the drill and blast-excavated portal tunnel. Information from table 34 and 35 has been used.

Table 40: Costs of rock support in the mapped area of the portal tunnel using the RMR-system

Type of support	Bolts	Shotcrete (m^3)	Mesh (m^2)
Quantity	469	156	565
Price per unit (NOK)	750	1500	56
Costs (NOK)	351.750	234.000	31.640
Total costs (NOK)	**617.390**		

Summary of costs related to rock support

Table 41 sums up the main results from table 36 to 40. Costs are given as both total costs, and as cost per meter tunnel excavated.

Table 41: Summary of costs related to rock support

Tunnel section	Headrace tunnel (TBM)		Portal tunnel (drill and blast)	
Classification system	Q	RMR	Q	RMR
Total costs (NOK)	1.006.650	1.261.952	464.250	617.390
Costs per meter (NOK)	1549	1941	4421	5880

If the headrace tunnel had been excavated with drill and blast the total costs of rock support had come to 1.225.650 NOK. This corresponds to 1886 NOK per meter tunnel. This number has been found by using the information given in chapter 4.3.3 and the calculations in table 38.

4.4 Laboratory results

Table 42 and 43 sums up the results from the laboratory testing.

Table 42: Laboratory test results

Specimen	Diameter (mm)	Length (mm)	Weight (g)	Length/diameter ratio	Density (kg/m^3)
1 – 1	43,1	110,9	431,0	2,57	2663
1 – 2	43,2	101,1	397,2	2,34	2668
1 – 3	43,2	111,9	438,9	2,59	2683
1 – 4	43,2	111,8	436,5	2,59	2669
1 – 5	43,2	89,9	352,1	2,08	2679

Table 43: Laboratory test results

Specimen	UCS (MPa)	Angle of failure	Mode of failure
1 – 1	200,1	19 °	Multiple shear
1 – 2	175,5	21 °	Multiple shear
1 – 3	214,5	23 °	Multiple shear
1 – 4	179,4	24 °	Multiple shear
1 – 5	193,3	21 °	Multiple shear
Average	192,6	22 °	
Standard deviation	15,8	2 °	

Figure 40 shows pictures before and after the testing was conducted. The pictures after testing shows the mode of failure for the five tested specimens.

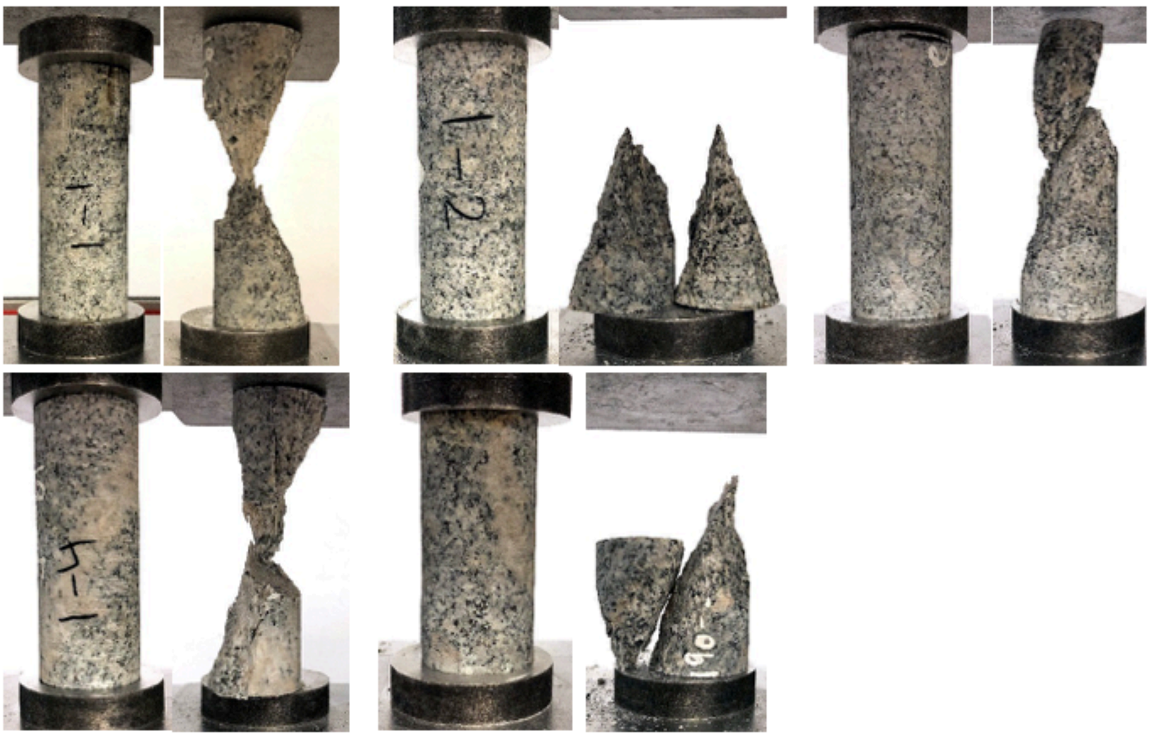

Figure 40: Modes of failure of the test specimens (Photo: N. Haugen)

5 Discussion

Within this chapter the results will be discussed on the basis of relevant theory and information.

5.1 Geological mapping

During the mapping activity, it was tried to follow the approach recommended by Palmstrøm and Broch (2006) and Hoek (2006). This approach suggests that one should register all important features of the rock mass before quantifying any parameters of a rock mass classification system. The purpose of this method is to be able to include all the important aspects of the rock mass in the mapping procedure. A weakness of rock mass classification systems is that they don't consider all factors that has influence on the rock mass (Palmstrøm and Broch, 2006). If the mapping is performed on the basis of a rock mass classification system one may be affected by the parameters that makes up the system. By attempting to comply with the suggested approach this issue with incompleteness of the mapping results were hopefully avoided.

However, it was experienced that it was hard to gather all necessary information observing the rock mass the first time. Therefore, a double checking of difficult parameters for some sections was done. Especially J_r and J_a in the Q-system, and the condition of discontinuities in the RMR-system was challenging. For these parameters, a method for double checking was done more of less systematically. The method is explained in chapter 3.4 and the sheets used for this double checking is shown in figure 14 and 15. It was found to be an efficient way to assure all information were taken into consideration when quantifying parameters for the rock mass classification systems.

It has been established that geological mapping is a procedure exposed to the influence of subjectivity. Amongst others Seo et al. (2018) detects this error when three researchers are set to map the same section of a tunnel. The outcome was that they all came up somewhat different results. To deal with this error, the geological mapping has been performed by two students in a team. When the geology was observed a discussion between the participants led to the characterization which were noted on the mapping sheet. This discussion underway the mapping was particularly important as it unraveled some misperceptions that were made individually. It was found to be an efficient way to minimize the error of subjectivity.

On the other hand, the lack of experience of the participants may have affected the results of the geological mapping in a more negative way. The magnitude of this possible error is hard to asses but it should not be disregarded. However, the two students that conducted the mapping had approximately the same level of experience, and they performed the mapping independent from any parties involved in the project. These two factors affected the outcome in a positive way. They made the participants focus on doing the mapping as realistic as possible, and the intension of the activity was to learn as much as possible while doing so.

Some difficulties were encountered when the mapping started in the middle of a tunnel. For example, the different sets of joints were hard to identify in the beginning. After a while it became clear that many joints had approximately the same orientation, and similar appearance in hence to shape, fillings, roughness etc. These joints made up a joint set. For the sections which were mapped first, these sets were not detected as easily as for the rest of the tunnel. This exemplifies that it is an advantage to see the big picture of the rock mass through the whole tunnel when conducting geological mapping.

The most challenging parameter to register concerning the rock mass was the fracturing. Both the Q-system and the RMR-system accounts for the fracturing by the RQD-value. The RQD-value is defined for examination of core logs, ref. chapter 2.3.1. As no core logs were available for the studied project other methods had to be applied. The volumetric joint count method suggested by Palmstrøm (2005), was proposed by the site geologist as a good replacement. While applying this method in firstly a tunnel excavated by TBM and then later in a tunnel excavated by drill and blast some interesting experiences were made. It was somehow easier to count fractures in a drill and blast tunnel where more small rocks had fallen out and the contour was more uneven. The fracture planes were more noticeable in a drill and blast tunnel. On the other hand, in a TBM-tunnel the even contour made it harder to see fractures, and to know whether it was a fracture or an intrusion or just an unevenness. To exemplify this, three pictures in the figures 41 to 43 is provided.

Figure 41: Very uneven surface, approx. 2,5 x 2 meters (Photo: O. Hobbelstad)

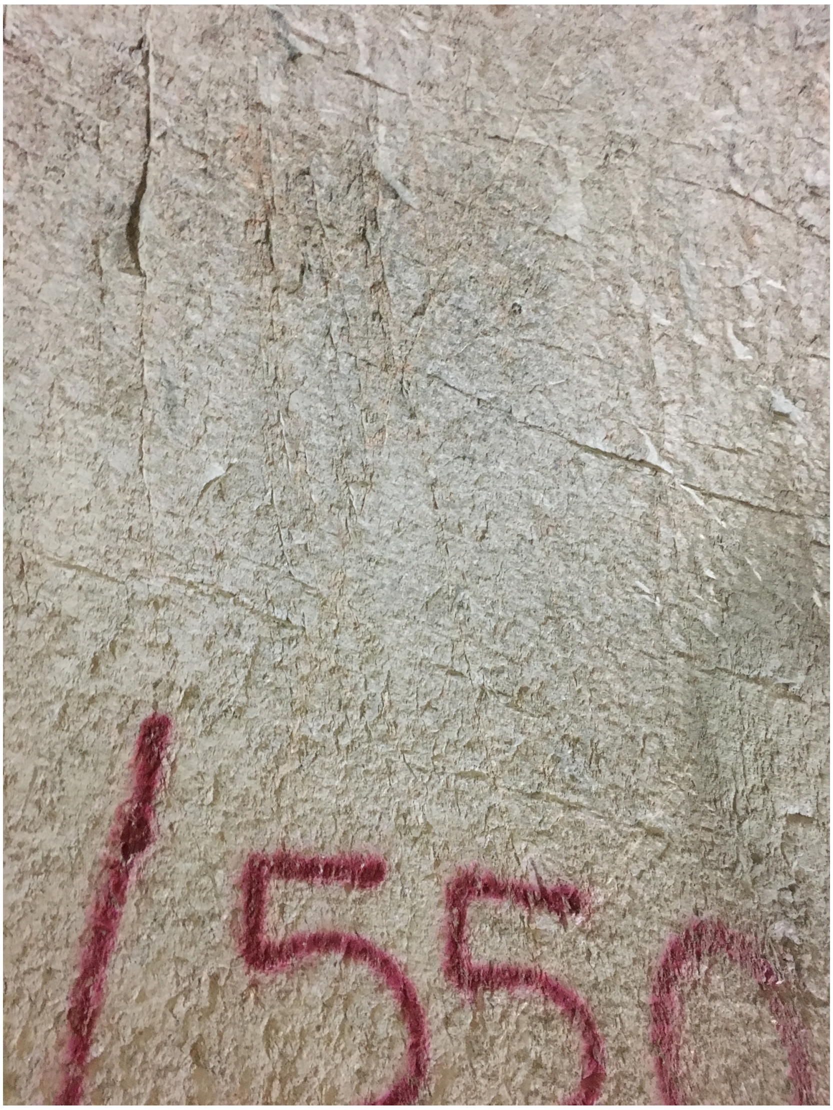

Figure 42: Uneven surface with some fractures, approx. 1,5 x 2 meters (Photo: O. Hobbelstad)

Figure 41 shows a very uneven surface. Here it is possible to see fractures, small cracks and something that’s just unevenness. This is hard to map in the way that it is difficult to find an area that is representative and it is difficult to see what that should be counted when using the volumetric joint count method. The rock mass in figure 41 was given a Q-value in the range of 27 – 29, and a RMR-value in the range of 63 – 66. In figure 42 the contour is smoother but there is still some unevenness. Some fractures are visible, and these are much easier to count than in the previous photo. Note that there is only 15 meters between these two locations. This shows that the appearance of the rock mass changes very quickly. The sudden change in smoothness of the contour is also a factor that makes the mapping of fractures difficult. The rock mass in figure 42 was given a Q-value of 57, and a RMR-value of 72.

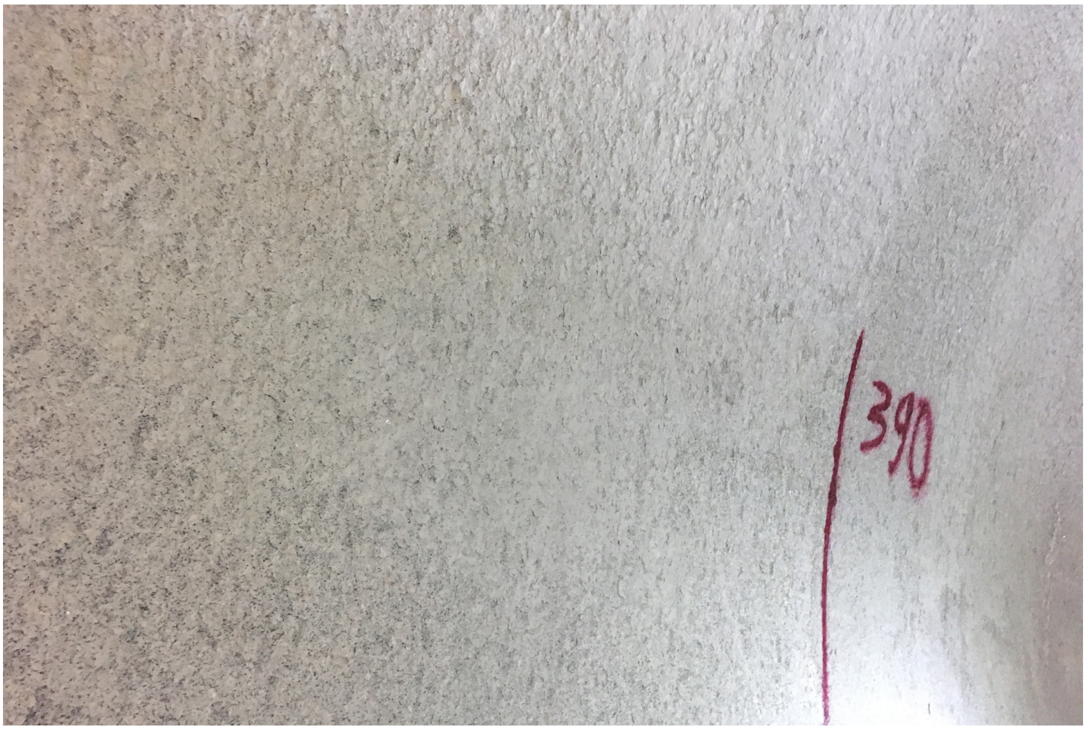

Figure 43: Extremely smooth surface, approx. 4 x 3 meters (Photo: O. Hobbelstad)

Figure 43 shows a location about 160 meters down the tunnel from the two previous photos. Here one can see that the contour is extremely smooth without any fractures at all. This was the case for long sections of the tunnel. The rock mass in figure 43 was given a Q-value of 160, and a RMR-value of 97. When the tunnel goes through large areas with such good quality rock mass as in figure 43 one may get biased that the rock mass elsewhere is poorer than it really is. This problem occurred many times, and it was experienced that being two students conducting the mapping was very helpful. Having a discussion about the observations really helped and may have avoided some errors of subjectivity as Seo et al. (2018) discusses.

Performing geological mapping in a tunnel excavated by TBM is a challenging task (NGI, 2015). Joints may be harder to see when a tunnel is excavated with TBM compared to drill and blast. This was experienced especially for joints that were more or less perpendicular to the tunnel axis. Joints with this orientation had in many cases no visible surfaces which made it difficult to inspect them. Some inaccuracy may therefore exist in parameters describing joint roughness and alternation. In addition, it made it impossible to measure strike and dip angles accurately. Joints may even be concealed in TBM excavated tunnels in good rock mass (NGI, 2015). This was observed many times during the geological mapping. Joints appeared to be short and discontinuous on one side of the tunnel, but was later observed on the other side some distance ahead. In the crown of the tunnel they were almost completely concealed, and was only recognized when the pattern was spotted on both sides.

The use of two rock mass classification systems for this thesis has been done because of several reasons. One is to provide more information about the rock mass than one system would have done. The RMR-system and the Q-system takes many of the same factors into account, but they emphasize them somewhat differently. Hoek (2006) strongly recommends that one should use at least two rock mass classification systems when mapping rock mass. Another reason is to be able examine the correlation between two systems. This has been done both as a quality check for the geological mapping, and to evaluate the accuracy of the already established correlation equations between the Q- and the RMR-system. More about rock mass classification and the correlation between Q and RMR is to come in chapter 5.2.

When conduction geological mapping in a tunnel under construction the safety is extremely important to keep in mind. Safety instructions were given ahead of the mapping and risk assessments were thoroughly gone through. Even a health and safety supervisor participated in the first visit inside the tunnel. In addition, a working safety analysis were performed midway the field work, just to be aware of the dangers that may be encountered. This is provided in appendix D.

5.2 Classification of rock mass

The classification of rock mass was done on the basis of the results from the geological mapping. After each session of mapping the parameters of the two classification systems were quantified. Then the rock mass was classified with a Q-value and a RMR-value and placed in a rock mass class in the given systems. While doing this on a daily basis during the field investigations the practical application and behavior of the Q-system and the RMR-system were experienced.

Hoek (2006) points out that the biggest difference between Q and RMR is the lack of a stress parameter in the RMR-system. The Q-system accounts for rock mass stresses in the Stress Reduction Factor (SRF), while the RMR-system does not take the stress situation into consideration at all. The effect of this were experienced during this work, when some sections of the tunnel were exposed to high stresses because of high overburden of rock mass. The normal magnitude of SRF was 2,5 were the rock mass showed no sign of stress problems. Where the rock mass had minor spalling on the walls, the SRF-factor was set to 5 and one place it was even set to 10. This could not be accounted for in the RMR-system, and led to some deviation between the two systems in these areas. The spalling may be seen in figure 44.

Figure 44: Spalling of rock mass at chainage 8170 (Photo: O. Hobbelstad)

As can be seen in figure 44, the spalling occurred in the middle part of the walls of the tunnel. This indicates that it is due to high vertical stresses caused by high rock overburden. At this point the overburden is estimated to be 750 meters, based on the geological profile in figure 11. The spalling happened during a slow process which did not cause big problems or danger.

When it comes to the filling material in joints it is accounted for in both Q and RMR. It is however done in very different ways. As presented in chapter 2.3.2 the Q-system uses the parameter joint alternation, J_a, to express the filling material. This parameter may vary between 0,75 and 20. It may cause a great impact on the Q-value, as it is directly implemented in the expression of the Q-value, that is based on multiplication. In the RMR-system the filling of joints is accounted for in the conditions of discontinuities parameter, as described in chapter 2.3.3. The part of this parameter that regards the filling of joints may only between 0 and 6. When the RMR-system is based on addition of parameters, this will not have that big of an impact on the final RMR-value. The consequences of this may be if for example a joint is coated with a very slippery clay material it is more accounted for in the Q-system than in the RMR-system. This effect was experienced at two locations in the field work. At chainage 8655 and at 8200 clay were found in joints. The J_a-parameter in the Q-system was given the value 8, which was the reason for large deviation between Q and RMR here.

In chapter 2.3.4, two equations expressing possible relations between Q and RMR is presented. Figure 33 and 34 in chapter 4.2.4 shows the calculation of these equations with the Q- and RMR-values produced in this thesis. The two plots are rather scattered with deviations mainly within $\pm$ 15 RMR-values. At first sight the two plots may look similar to each other, but the trendlines tells a different story. The trendline in figure 33 using equation 2.6 is more horizontal and shows deviation around 1 to 3 RMR-values, while the trendline in figure 34 using equation 2.7 has a steep slope with deviation varying from -4 to 8 RMR-values. This result indicates that equation 2.6 gives a better relation between Q and RMR in this case. Figure 32 confirms the good relation between the rock mass classification performed and equation 2.6. The best fitting line between mapped Q- and RMR-values and the relation given by equation 2.6 almost coincides. Calculations done on this matter may be seen in appendix A.

The use of correlation equations, such as the ones referred to in chapter 2.3.4, should be done with great care (Palmstrøm and Broch, 2006). Since different classification systems emphasize rock mass parameters differently, deviations in the results must be expected. This also the case for the results found in this thesis. Both figure 33 and 34 show large deviations when using correlation equations. The equations have mainly been used here as an assurance for the quality of the field work. The fact that equation 2.6 in figure 32 and 33 shows such good correlation with the work performed is a good sign that the classification has been done more or less correctly. However, it is important to emphasize that the equations have not been used directly to estimate values in another system for other applications.

Palmstrøm and Broch (2006) specifies that the Q-system is best suited for jointed and fractured rock mass, and that it works fair in slabbing and spalling rock conditions. In other kinds of rock masses, they questions whether the Q-system is suitable as a rock mass classification system. Concerning the rock mass at Upper Kon Tum HPP, it is characterized as a hard and competent rock mass with joints and fractures. At two locations spalling rock conditions was observed, but no other special events were discovered. This indicates that the Q-system should be applicable for classifying rock mass at Upper Kon Tum HPP.

Even though there are some weaknesses connected to the use of rock mass classification systems, a great benefit is the general basis of communication it provides. Palmstrøm et al. (2002) argues that rock mass classification systems are a good tool for communication between various disciplines involved in a project. It gives a common understanding about the rock mass in question. This effect of rock mass classification systems was experienced while being present at an international tunneling project during the field work of this thesis. People from all around the world with different backgrounds were able to discuss the quality of the rock mass based on a quantified number from a classification system.

5.3 Rock support

The use of rock support is based on considerations concerning safety, stability (long and short term) and costs (Bollingmo et al., 2010). This goes for both the amount of rock support and what type of rock support that is to be used.

In chapter 4.3.1 and 4.3.2 amounts and type of rock support has been calculated based on the Q-system and the RMR-system. The different types of rock support given by the classification systems are rock bolts, shotcrete, wire mesh and steel sets. The specific type of these rock support measures is not given in any more detail by the classification system. In the following, the amounts and the type of rock support will be discussed on a more specific level.

As the main danger towards stability is fallout of block and rocks in a bored tunnel in hard rock conditions (Barton, 2000), some end anchoring of rock bolts would be favorable. The end of the bolt is placed in solid rock mass, and an anchor hold a possible loose block in place. Ordinary end anchored bolts provide this feature, but in Norwegian tunneling they are not allowed as permanent support (Rostad et al., 2014). A better solution would therefore be a combinational bolt. As described in chapter 2.2.1, this kind of rock bolt uses both end anchoring and embedment to support the rock mass. This makes it more durable and it works as both temporary and permanent support. However, fully embedded rock bolts are often used as they are a less costly solution.

The calculated amount of rock bolts for areas which only require spot bolting is based on the recorded amounts of rock support from the mapping. In the headrace tunnel, there was both little need for and little use of spotted bolting. Therefore, a conservative assumption of 1 bolt per a section of 10 meter tunnel has been made. In the portal tunnel, the quantity of bolts needed for spotted bolting has been calculated based on the number of bolts per meter that was counted during the mapping. This information is listed up in chapter 4.1.2.

Combinational bolts have been chosen as recommended type of bolts based on the discussion above. They have been subject to the cost analysis done in chapter 4.3.4. From the cost analysis, it can be seen that rock bolts makes up the largest part of the costs. Choosing the right type of bolts is therefore important.

Shotcreting may be done in two ways, either by dry mixing or wet mixing, as described in chapter 2.2.2. For the studied tunnel, shotcrete is only necessary some places. The places with need for shotcreting may also be far apart from each other. Based on this, dry mixing of shotcrete may be the most applicable solution for this project. This is because it is a more flexible method than wet mixing, as described in table 1. The amounts of shotcrete proposed by the classification systems for the headrace tunnel in chapter 4.3.1 is pretty similar to what is actually used as rock support in the tunnel. On the other hand, in the portal tunnel the rock mass classification systems suggests a lot more shotcrete than what is used.

For heavy rock support the RMR-system recommends the use of steel sets, while the Q-system recommends reinforced ribs of sprayed concrete (RRS). The TBM at the examined project is rigged with erectors for installing steel sets/ringbeams. This is the only heavy support measure that is possible to install in the L1-area, directly behind the protective roof shield. Therefore, an assumption has been made that RRS in the Q-system corresponds to ringbeams as heavy rock support. A zone of weakness crossed the tunnel around chainage 8530 and was the cause of poor rock mass in this area. Heavy rock support was already installed when the mapping was carried out. Q- and RMR-values was based on the installed rock support here, and the amount of heavy rock support calculated is therefore more or less the same as the installed rock support.

Another kind of heavy rock support that may be applicable is fully cast concrete lining. For the same reason as for RRS, fully cast concrete lining is not applicable as immediate rock support right behind the protective shield. It might, however, be a measure if the rock mass shows serious signs of weakness behind the TBM. Some spalling of rock mass was observed at two locations in the mapped section. If this tends to develop further, heavy rock support as for example fully cast concrete lining may be necessary to cope with the problem.

Palmstrøm et al. (2002) draws up an area of the support diagram of the Q-system of which they argue the diagram works the best. This is described in chapter 2.3.5 and shown in figure 7. The area includes Q-values ranging from 0,1 up to 40. By comparing this to the mapped Q-values displayed in chapter 4.2.1 it can be seen that the majority of Q-values is within this interval. Some values are above 40, but the argument for that the interval stops at 40 is that such high Q-values describes rock mass that generally don't need recommendations for rock support (Palmstrøm et al., 2002). The fact that most of the Q-values is within 0,1 to 40 means that the Q-system should be applicable for rock support estimations.

The RMR-system is different from the Q-system in the way that it recommends rock support. Estimations is given after the division into five rock mass classes, as shown in table 17. Every rock mass class spans over 20 RMR-values, which is a pretty coarse division. For example, is a section with RMR-value of 41 given the same recommendations for rock support as a section with RMR-value of 60. With this in mind some inaccuracy is anticipated in the rock support recommendations from the RMR-system. Bieniawski (1997) argues that rock mass classification systems should not be used to estimate rock support during construction, while Løset (1997) at NGI states that the Q-system is made to be used for deciding rock support during construction. This may be the reason why the rock support chart of the Q-system gives more detailed information about rock support than the RMR-system.

The example discussed above, where the same amount of rock support is estimated for rock mass with RMR-value 41 and for rock mass with RMR-value 60, may have some consequences. For RMR-values in the upper part of this interval, rock support may be overestimated. Amounts of rock support calculated in chapter 4.3.1 and 4.3.2 shows that RMR consequently recommends higher amounts of rock support. Also, the cost estimations in chapter 4.3.4 shows significantly higher costs using the RMR-system. This indicates that there may be an issue with overestimation of rock support in the RMR-system because of the coarse division of rock mass classes.

Given that the rock support recommendations in the RMR-system is made to be used in the preliminary stages of a project, as Bieniwaski (1997) argues, a slight overestimation of rock support might not be a bad approach. As Nilsen (2014) discusses, the pre-construction investigations is supposed to provide a basis for estimating time and cost and should give input to analyzing the need for rock support. When little information is available, a conservative estimate regarding rock support and costs may be a good way to start.

The rock mass at Upper Kon Tum HPP is characterized as a hard rock mass regularly intersected by zones of weakness. One of these zones of weakness occurred around chainage 8530, and lead to the use of heavy support measures. Elsewhere the hard rock mass is intersected by long continuous joints, which means that the main stability problem here is fallout incidents of blocks and smaller rocks. Thus, the need for rock bolting as calculated in chapter 4.3.1. The joints mainly intersect the tunnel with steep dip angles, which increases the friction and thus the stability of possible loose blocks and wedges. However, fallouts may still occur. One incident of this kind is shown in figure 45, where a small fracture intersected a larger joint in the crown of the tunnel.

Figure 45: Fallout of wedge between steep joint intersected by a fracture (Photo: O. Hobbelstad)

Both NGI (2015) and Barton (2000) recognizes the issue related to fallout of loose wedges as the main stability problem during mechanized tunneling in hard competent rock mass. Awareness about the problem and good routines for rock support is the tools to cope with it.

The rock mass had great capability in supporting itself. This allegation is based on the low quantity of rock support used throughout the headrace tunnel. However, the presence of water seemed to be the triggering cause to the need of rock support measures. For the section of tunnel which was mapped for this thesis inflow of groundwater was a common feature of where rock support was installed. The collected data is not sufficient to make a final conclusion on this matter, but the clear tendency is worth commenting on.

The fact that the distance between the mapped section of the TBM-excavated headrace tunnel and the portal tunnel is 9,5 km may have harmed the comparison of rock support in the two locations. At the portal tunnel the rock type is gneiss, while the mapped section of the headrace tunnel starts in gneiss/granitic gneiss and moves onwards into biotite granite. A SINTEF report from December 2017 has been provided by the contractor. This report concerns compressive strength analysis of the gneiss at site. Uniaxial Compressive Strength (UCS) has been measured at 10 locations which gives an average result of 191,9 MPa.

During the field work 5 rock samples were gathered at one location in the biotite granite. These samples were tested for UCS at SINTEFs facilities in Trondheim in May 2018. The average result of UCS for the five specimens was 192,6 MPa, as presented in chapter 4.4. The laboratory testing also showed very little variation between the tested specimens with a standard deviation of only 15,8 MPa. Based on the fact that the UCS is so similar in the gneiss and the biotite granite it has been assumed that the behavior of the rock mass in the two rock types are comparable. This means that the two locations where geological mapping was conducted can be compared to each other even though the distance between them is 9,5 km.

In chapter 2.4.3 the rock support strategy stated by the project owner is given. The rock support is related to the rock mass classes based on intervals of Q-values. These rock mass classes are however different from the original rock mass classes in the Q-system. This is a distinctive approach, who aims to be specially suited for this project.

The guidelines for rock support is also different from the recommendations given by the Q-system. The reason for this appears to be that they use other types of rock support than what is suggested by the Q-system. This concerns mainly ringbeams and wire mesh, which is used at a large scale. A specially designed guideline for rock support measures is therefore suitable in this case.

The rock mass classes stated by the project owner is, as said above, defined on the basis of intervals of Q-values. This is a bit strange given that a report issued by the owner, clearly states that both the RMR-system and the Q-system is to be used when classifying rock mass. Hoek

(2006) argues that it is favorable to use at least two rock mass classification systems as a way of assuring that more information about the rock mass is taken into consideration. This is probably the reason why it was decided to use both Q and RMR in the first place for this project. With this in mind, the rock mass classes should have been defined for intervals of both Q-values and RMR-values.

The amounts of rock support specified for rock mass class I and II, seems to be a bit exaggerated. For temporary support during construction, it has not been found necessary to use as much as three rock bolts per meter tunnel in fair to extremely good rock mass. Long sections of the tunnel pass through rock mass of this characteristics, and has been left unsupported without large problems. As Palmstrøm et al. (2002) argues, for rock mass with Q-value over 40 (very good rock mass) only spotted bolting is necessary. It is then better to leave the decisions to an experienced tunneling crew regarding placing of bolts than to follow a strict scheme from a classification system. The fact that sections of the tunnel that pass through very good rock mass is left unsupported indicates that an approach similar to the one recommended by Palmstrøm et al. (2002) has been followed.

Reasons to reduce the amount of rock support is for instance to save costs and time and to spare the environment for unnecessary use of materials. On the other hand, safety concerns should always be prioritized. The costs of rock support represent only a small share of the total costs of constructing a tunnel. This raises the question of why not use a bit extra rock support to be on the safe side. The balance between these two aspects is hard to find, but it is important to keep in mind when deciding type and quantity of rock support. Incidents like the wedge-fallout in figure 45 is one example of an event that could have been avoided by only the use of rock bolting.

An interesting project to compare the results found in this thesis with is Fløyfjellstunnelen in Bergen, Norway. The tunnel was built in the years of 1984-1989, and consists of two parallel tunnels which in total is 7 km long. They were excavated by TBM and later the lower sides of the tunnel were blasted out so that the cross section would fit a two-lane roadway (Jakobsen and Arntsen, 2014). During the TBM excavation only 200 rock bolts were used as rock support. When the sides of the tunnel later were blasted out systematic rock bolting was found necessary to implement. Bolting was performed on each side of the tunnel in the transition between blasted and bored contour with spacing of 1 meter. Only based on the spacing, this means that the estimated total consumption of rock bolt comes to a quantity of 14.000 after the blasting was carried out. This corresponds to roughly 2 rock bolts per meter tunnel.

This large difference in consumption of rock bolts is remarkable, and suggests that blasting damages the rock mass such that more rock support is required. Compared to the results found at Upper Kon Tum HPP this can be somehow justified. The portal tunnel excavated by drill and blast had a much higher consumption of rock bolts than the bored headrace tunnel. The rock support used at Upper Kon Tum HPP is listed in table 25 and 26, for the headrace tunnel and the portal tunnel respectively. In fair to very good rock mass only three rock bolts were found in the bored headrace tunnel. In the portal tunnel, where systematic bolting was carried out, the number of rock bolts per meter was 1,8.

Another interesting project to compare the results with is Røssåga HPP. The headrace tunnel here was excavated by TBM and was 7,4 km long with a diameter of 7,2 meters (TheRobbinsCompany, 2015). It was built during the years of 2013 – 2016, and went through very varying geology. For the whole tunnel, 1375 rock bolts and 377 m^3 of shotcrete were used as rock support. The mapped section of the headrace tunnel at Upper Kon Tum HPP also went through varying geology, and had some consumption of shotcrete because of this. In fact, shotcrete was used for 39 meters of the 650 meters that were mapped. With an assumed average thickness of 100 mm, this leads to a quantity of around 41 m^3. Per meter tunnel this quantity is very similar to the used amount of shotcrete at Røssåga. The consumption of rock bolts, however, is a bit larger at Røssåga. Some of this difference may be explained by the diameter of the tunnel, which is 2,7 meters larger at Røssåga.

Based on the comparison with these two projects it can be seen that the consumption of rock support material is similar in many ways. Both regarding rock support in TBM excavated tunnels and excavation related to drill and blast. The amount of rock bolts used in Fløyfjellstunnelen indicates that the need for rock support is lower in a bored tunnel than when blasting works are carried out. The results from Upper Kon Tum HPP also indicates that the need for rock bolts is lower in a mechanically excavated tunnel.

An interesting finding is that it is used much more rock bolts in the drilled and blasted portal tunnel than in the bored headrace tunnel. Between chainage 55 and 105 the rock mass in the portal tunnel is of similar quality compared to large parts of the headrace tunnel. In the sections mentioned, the rock mass was mapped to a quality of fair to extremely good. Based on the laboratory results, it has been assumed that a comparison of the rock mass in the two rock types is feasible. This indicates that the smooth contour which is achieved with mechanized tunneling reduces the need rock support in fair to extremely good rock mass conditions in the form of rock bolts.

555 meters of the headrace tunnel was mapped to be in the rock mass classes of fair to extremely good. If this tunnel had been excavated by drill and blast, according to chapter 4.3.3, the total consumption of rock bolts had come to 882 pieces. By excavating with TBM the amount of rock bolts necessary is estimated to 616 pieces. This difference is because of the enlarged need for spotted bolting. To decide the number of bolts needed for spotted bolting the results from the mapping has been used as a basis.

This results in a difference of 266 rock bolts. Using the costs given in the cost analysis in chapter 4.3.4, this gives a total difference in costs of 199.500 NOK. Translated into costs per meter tunnel excavated this is 359 NOK/m. These calculations are based on all the mapped rock mass within the rock mass classes of fair to extremely good, by using the Q-system. The Q-system has been used for this purpose because it is easy to compare recommendations for rock support for different cross sections.

When the total costs of all rock support for the entire mapped section of the headrace tunnel is considered, the difference comes to 219.000 NOK. This corresponds to 337 NOK per meter tunnel excavated. The small difference between this case and the example above, with rock mass is the classes of fair to extremely good, is related to the zone of weakness that was encountered. Heavy rock support was needed in this zone regardless of the tunneling method.

In rock mass of poorer quality than fair, according to the Q-system, it has been found that classification systems recommend more rock bolts than what is actually necessary in this case. For other types of rock support, like shotcrete and steel sets/ringbeams the recommendations seems to be more accurate. The basis of data in this case is however limited, and more mapping of poor quality rock mass needs to be done to make a conclusion on this matter.

6 Conclusion

The effects on rock support caused by the method of excavation of tunnels has been examined in this thesis. A hypothesis has formed the basis for the assignment. This states that:
"The amount of rock support is reduced in tunnels excavated by tunnel boring machines compared to tunnels excavated by drill and blast".
To be able to make a conclusion upon this topic two research questions were formulated.

Research question number 1:
"Can the effect of a better rock contour achieved with mechanized tunneling be related to reduced rock support?"

The need for rock bolting as rock support seems to be higher in tunnels excavated with drill and blast. Both spotted and systematically rock bolting has been found necessary to implement in the portal tunnel, excavated by drill and blast, to a large extent. In the headrace tunnel, excavated by TBM, almost no rock bolting has been carried out. The reason seems to be the method of excavation, which causes lower disturbance of the surrounding rock mass.

For other types of rock support, such as shotcrete, wire mesh and steel sets/ringbeams, it appears to be little relation between the need for rock support and the method of excavation. However, the basis of data on this matter is limited and a final conclusion cannot be made in this case.

These findings are substantiated by the field work and the rock mass classification performed.

Research question number 2:
"How much can the amount of rock support and the costs be reduced in fair to extremely good rock mass conditions?"

In fair to extremely good rock mass conditions only spotted rock bolting is necessary as rock support for the studied tunnel. This is because of the small diameter of the headrace tunnel at Upper Kon Tum HPP. In these areas, it has been found that the amount of rock bolts can be reduced by a quantity of 266 for a section of 555 meters. This makes up a total cost of 199.500 NOK, which corresponds to 359 NOK per meter of tunnel excavated.

To sum up the findings in the light of the hypothesis, the conclusion is that the amount of rock support is reduced in tunnels excavated by tunnel boring machines compared to tunnels excavated by drill and blast in the form of rock bolts. For other kinds of rock support, no relation to the method of excavation has been found. However, the basis of data is not strong enough to make up a conclusion. It is, nevertheless, likely that further studies will be able to confirm this indication.

Further work

To be able to do further research on this field, more data needs to be collected. Tunnels excavated with both TBM and drill and blast in the same geological formations should be examined to get a wider basis of data. It would also be advantageous to map sections of tunnels with different methods of excavation closer to each other than what was achieved for this thesis.

As geological mapping is a time demanding work, it could also be beneficial to map selected sections in a more "shallow" fashion. For example, only map the installed rock support in a tunnel and possible stability problems. This approach is purely to provide a bigger volume of data to be examined in the light of the hypothesis stated for this thesis.

More laboratory testing is also a subject for further work. Two sections of the headrace tunnel at Upper Kon Tum HPP were exposed to stress problems. Testing of active stresses in field, and relevant laboratory tests for the topic would be interesting. It could then provide the possibility to address the effects of stresses on the rock mass and potential rock support.

This thesis has mainly concerned rock mass of quality between fair and extremely good. A natural scope for further work within this topic would be to map more rock mass of poorer quality. The need for rock support in such rock mass quality could then be examined with a bigger basis of data, for both mechanically excavated tunnels and tunnels excavated by drill and blast.

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Appendix

Appendix A: File attachment. Spreadsheets for rock mass classification

See attached zip folder, under the following file name:

"Rock mass classification.xlsx"

Appendix B: File attachment. Geological mapping of headrace tunnel

See attached zip folder, under the following name:

"Geological mapping - Headrace TBM.pdf"

Appendix C: File attachment. Geological mapping of portal tunnel

See attached zip folder, under the following name:

"Geological mapping - Portal Drill and Blast.pdf"

Appendix D: Risk assessment for geological mapping

NTNU	Sikker jobb-analyse (SJA) - generell	utarbeidet av	Nummer	Dato
		HMS-avd.	HMSRV2606	29.03.11
		Godkjent av	side	Erstatter
HMS		Rektor	1 av 2	

SJA tittel: Feltarbeid, kartlegging i Kon Tum

Dato: 12.3.18 Sted: Kon Tum, Vietnam

Kryss av for utfylt sjekkliste: X

Deltakere:

Ola Hobbelstad		
Egil Johansen		
SJA-ansvarlig:		

Arbeidsbeskrivelse: (Hva og hvordan?)

Kartlegging av geologi ifm masteroppgave, ved tilløpstunnel til Øvre Kon Tum vannkraftverk.

Risiko forbundet med arbeidet:

Høy temperatur
Tog kjører inn og ut av tunnelen
Tap av strømtilførsel: ventilasjon og lys forsvinner inntil nødstrømtilførsel aktiveres.
Snuble risiko

Beskyttelse/sikring: (tiltaksplan, se neste side)

Konklusjon/kommentar:

* Størst fare tilknyttet kjørende tog. Årvåkenhet er viktig, høre og se etter når tog kommer.
* Størst sannsynlighet for snubleskader, men lav konsekvens. Se ned der man går er viktig.

Anbefaling/godkjenning:	**Dato/Signatur:**	**Anbefaling/godkjenning:**	**Dato/Signatur:**
SJA-ansvarlig:		Områdeansvarlig:	
Ansvarlig for utføring:		Annen (stilling):	

NTNU HMS	Sikker jobb-analyse (SJA) - generell	utarbeidet av	Nummer	Dato
		HMS-avd.	HMSRV2606	29.03.11
		Godkjent av	side	Erstatter
		Rektor	2 av 2	

HMS aspekt	Ja	Nei	Ikke aktuelt	Kommentar / tiltak	Ansv.
Dokumentasjon, erfaring, kompetanse					
Kjent arbeidsoperasjon?		X		Sikkerhetsinstruksjon	
Kjennskap til erfaringer/uønskede hendelser fra tilsvarende operasjoner?		X		— " —	
Nødvendig personell?	X				
Kommunikasjon og koordinering					
Mulig konflikt med andre operasjoner?	X			Annet arbeid foregår / Kommunikasjon	
Håndtering av en evt. hendelse (alarm, evakuering)?	X				
Behov for ekstra vakt?		X			
Arbeidsstedet					
Uvante arbeidsstillinger?	X			Man ser mye opp i tak	
Arbeid i tanker, kummer el.lignende?			X		
Arbeid i grøfter eller sjakter?			X		
Rent og ryddig?	X				
Verneutstyr ut over det personlige?		X			
Vær, vind, sikt, belysning, ventilasjon?	X				
Bruk av stillaser/lift/seler/stropper?		X			
Arbeid i høyden?		X			
Ioniserende stråling?		X			
Rømningsveier OK?	X				
Kjemiske farer					
Bruk av helseskadelige/giftige/etsende kjemikalier?		X			
Bruk av brannfarlige eller eksplosjonsfarlige kjemikalier?		X			
Må kjemikaliene godkjennes?			X		
Biologisk materiale?		X			
Støv/asbest?	X			Støv / støvmaske	
Mekaniske farer					
Stabilitet/styrke/spenning?	X			Nødvendig sikring er utført	
Klem/kutt/slag?	X			Alarm signaliserer oppstart.	
Støy/trykk/temperatur?	X			Hørselvern + alltid tilgang til vann	
Behandling av avfall?			X		
Behov for spesialverktøy?		X			
Elektriske farer					
Strøm/spenning/over 1000V?	X			Isolerte ledninger	
Støt/krypstrøm?			X		
Tap av strømtilførsel?	X			Nød generator på batteri	

NTNU	Sikker jobb-analyse (SJA) - generell	utarbeidet av	Nummer	Dato
		HMS-avd.	HMSRV2606	29.03.11
		Godkjent av	side	Erstatter
HMS		Rektor	3 av 2	

Området					
Behov for befaring?			X		
Merking/skilting/avsperring?			X		
Miljømessige konsekvenser?		X			
Sentrale fysiske sikkerhetssystemer					
Arbeid på sikkerhetssystemer?		X			
Frakobling av sikkerhetssystemer?		X			
Annet					

Appendix E: Engineering geological profile of Upper Kon Tum HPP