Evaluation of the differences in characterization and classification of the rock mass quality

A comparison between pre-investigation, engineering geological forecast and tunnel mapping in the Northern Link project and the Cityline project

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Abstract

In the construction of a tunnel, the characterization of the rock mass is performed in three different steps, in the pre-investigations, in the engineering geological forecast and in the tunnel mapping during construction. There has in previous work been observed that discrepancies exist between the results from these different steps, with a tendency to assign poorer rock mass quality in the tunnel mapping than in the pre-investigations and in the engineering geological forecast. One example is the work done by Kjellström [1] on the Cityline where the divergence in rock mass quality was analyzed between the different steps. If a divergence exists between the engineering geological forecast and the actual conditions observed in the tunnel mapping, it will influence both planning and budget. It is therefore important that the engineering geological forecast is as close as possible to the actual rock mass conditions in the field.

The aim of this thesis was, using the case study of the Northern Link, to analyze those discrepancies in the rock mass quality estimated in the characterization and in the classification between the mapping of drill cores, the engineering geological forecast and the tunnel mapping thus complementing the work by Kjellström [1]. The aim was also identifying which parameters included in the Q-system that causes these discrepancies.

The analysis of the results showed that it is difficult to make the engineering geological forecast and the actual mapping match for every single meter, but that the overall correlation between them was good. The methodology used in the characterization and classification in the different phases (drill-core mapping, engineering geological forecast, tunnel mapping) may to some extent explain this divergence. The parameters Jr, Jn and Ja, included in the Q-system were the ones identified as having the largest influence on the discrepancies. In future work, it is recommended that focus is given on these parameters.

A way to improve future engineering geological forecast for tunnel contracts would be to have a better follow up of the engineering geological forecast and to have standardized guidelines on how to assess clearly the value of the Q parameters in each phase (for the drill cores as well as for the actual mapping). The reduction of those differences would then lead to a better planning and budget management in future tunnel projects in Sweden.

Key words: Characterization, classification, rock mass quality, engineering geological forecast, tunnel mapping, discontinuities, Q-system, Northern Link.
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1. Introduction

1.1. Background

The reason a tunnel project’s budget is exceeded can be for example that there is a divergence in the estimation of the rock mass quality. Peter Lundman’s PhD thesis “Cost Management for Underground Infrastructure Projects: A case study on cost increase and its causes” illustrates that [2].

The rock mass quality in a project is a key data. Several activities depend on it, especially during the tunneling phase. Activities and/or parameters that are influenced by the value of the rock mass quality are for example the amount of rock support, the excavation process (time, machines…) and the amount of grouting.

When it comes to constructing a tunnel, there are three steps that are done to characterize the quality of the rock mass, each step corresponding to a specific moment in the tunneling process (Figure 1-1):

1. Drill core mapping (pre investigation mapping).
2. Prevision of the rock mass quality (engineering geological forecast). Basis for design and tender process.
3. Actual tunnel mapping, which consists of having a geologist evaluating the rock mass quality while the tunneling is done.

*Figure 1-1 The three steps of the rock mass evaluation that can lead to deviations in the tunnel project if there are discrepancies between the steps.*

Any change on the value of the quality of the rock mass can have a consequence on the economy and on the planning if there is a divergence between the three steps.

The rock mass quality is e.g. assessed by estimating the Q-index in the planning phase. During this phase the amount of information available is limited, and estimating the Q-value gives a first idea of the reinforcement that are going to be necessary for the tunnel and allow the planner to do a first cost estimation.

Today, there is a large degree of freedom for discrepancies in the assessment of rock mass quality, which can lead to a divergence of the evaluated rock mass quality value between those three steps. A previous master thesis work studied the case of the Stockholm Cityline Project [1] with respect to divergences of the rock quality between the different steps. The engineering geological forecast and the outcome of the rock support were available, and a divergence between them was noticeable.
1.2. Aim

The aim of this master thesis is:

- To analyze if there exist discrepancies in the rock mass quality estimated in the characterization and in the classification between the mapping of drill cores, the engineering geological forecast and the tunnel mapping for the Northern Link project.
- If a discrepancy is observed, identify which parameters included in the Q-system that causes these discrepancies.

Furthermore, the aim is to identify if there may exist differences in the methodology in the characterization and classification in the different phases (drill-core mapping, engineering geological forecast, tunnel mapping) which can explain this divergence.

The results of the work may be used to improve future engineering geological forecast for tunnel contracts, and would complement the work by Kjellström [1]. The expectations are to get an increased understanding for how the mapping of drill cores should be carried out, the engineering geological forecast established and the tunnel mapping carried out with the aim to get less divergence between those steps. The reduction of those differences would then lead to a better planning and budget management in future tunnel projects in Sweden.

1.3. Outline of the thesis

In the first chapter, a presentation of the background and the aims of the work is introduced. In chapter two, a literature review is performed on characterization and classification of rock mass quality with respect to the three steps pre-investigation, engineering geological forecast and tunnel mapping. The means available to the geologist to perform the various steps (rock characterization and classification systems) are also described.

The third chapter contains a more specific description of the tunnel parts studied in the Northern Link as well as a description of the procedures followed by the geologists when performing the tunnel mapping in that project. In the fourth chapter, the methodology used to obtain the aims of the thesis is presented. In addition, the data available to do my study is presented. Moreover, assumptions about how the parameters included in the Q-system might change between mapping of drill cores, the engineering geological forecast and the tunnel mapping are listed. The results are presented in chapter five, presenting first the main trends of the rock mass quality distribution observed and then the influence on the discrepancies of each parameter in the Q-system used. Finally, chapter six and seven discuss the results and the conclusions and recommendations for future related work are presented.

1.4. Limitations

First, it is assumed that the results from mapping of the drill core could be assumed to represent a large scale sample of the rock mass that would be encountered in the tunnel. This constitutes a main uncertainty in the results. The assumption was judged acceptable because a large number of drill
cores were drilled to investigate the rock mass along the tunnel stretch. However, when the results are interpreted, this limitation should be kept in mind.

It is also assumed that the length of drill core is enough to represent an average value of the rock mass quality it goes through. Thus drill cores meters are compared with meters of the actually mapped tunnel.

Moreover, the rock mass quality is forecasted based on an interpretation of the information from the drill cores and from other geological observations. That’s why there are no values for each Q-parameter but rather indications of Q-groups according to the interpreted data.
2. Literature study

Assessing the rock mass quality is one of the most important tasks when planning an underground construction. Indeed, the estimated rock quality should be the closest to the actual one as any change on the value of the rock mass quality can have a consequence on the economy of the project.

The rock mass quality is assessed at three different occasions during a tunnel project, at the drill core mapping, at the engineering geological forecast and when the tunnel mapping is performed. As said earlier in the introduction, divergences between these three steps can occur. In this chapter, the work by the geologist is described in the different phases of the tunnel construction. The different system that are available to characterize and classify the rock mass are thereafter presented. The evolution of uncertainty throughout the tunnel construction phases are also illustrated.

2.1. Pre-investigation

The pre-investigation is performed before the tunnel excavation to have a better idea of how the rock looks like. Some of the investigations performed during the pre-investigation that lead to the establishment of the engineering geological forecast are: mapping of drill cores, BIPS images - Borehole Image Process System, mapping of outcrops (possible joints, strike and dip, shear zones, topography), analyses of satellite photographs, test-drilling and seismic investigations.

2.2. Engineering geological forecast

This material from the pre-investigations is then interpreted leading to an engineering geological forecast which describes the expected rock mass quality along the tunnel. Two different definitions of the rock mass forecast exist: the engineering geological forecast and the rock engineering forecast [3].

The engineering geologist performs the geological forecast. It consists of a report with appended drawings and contains the conditions interpreted by the engineering geologist. The engineering geological forecast constitutes a part of the construction documents. The engineering geological forecast is based, among other things, on the pre-investigation report. Note that this forecast isn’t normally included in the drawings/specification but constitutes the basis for planning.

In Figure 2-1 and 2-2, an example of such a forecast is available. As it’s noticeable in the zoomed part, the rock class, the reinforcement class, the sealing class and the geotechnical class are available for every tunnel meter.
The other rock mass forecast is the rock engineering forecast that consists of drawings and corresponding texts which account for the rock mechanic conditions in the system documents and/or construction document/Pre-investigation (FU). The text sections are presented in a technical description or the corresponding document. Since the rock engineering forecast is included in the construction documents/ Pre-investigation (FU), it also constitutes the basis for the execution of the tunnel and thus also a basis for tendering the bill. The rock engineering forecast is a "stripped down" and simplified / generalized variant of the engineering geological forecast but also contains information about the forecasted support and grouting operations.
2.3. Tunnel mapping

The tunnel mapping is the step that comes after the rock mass forecast. It is performed after excavation of the tunnel and is performed every three blast if the rock is good enough or every blast if there is a necessity to support the rock mass after each blast. This mapping gives the actual rock mass quality value that is going to be used to assess the rock support necessary to perform a safe excavation. A goal would be to have a perfect correlation between the rock mass forecast and the actual rock mass quality mapped in the tunnel after excavation.

2.4. Factors influencing the tunnel stability and how to evaluate them

Discontinuities are one of the factors influencing the tunnel stability and especially the rock mass quality as summarized in Figure 2-3 below [4]. Hudson and Harrison say that “In the engineering context here, the discontinuities can be the single most important factor governing the deformability, strength and permeability of the rock mass.” [4]. This emphasizes the importance of having a closer look at discontinuities when studying the rock mass quality.

Hudson and Harrison define a discontinuity as “any separation (plane or surface) in the rock continuum having effectively zero tensile strength and is used without any genetic connotation” (no information about how it was formed). Discontinuities can then be joints, faults, bedding planes, rock cleavage planes (foliation, schistocity) or weakness zones.

The geometrical properties of discontinuities are of primary importance. In the figure below, an illustration of the parameters used to describe the discontinuity characteristics in the rock mass is presented.

![Figure 2-3 Schematic of the primary geometrical properties of discontinuities in rock [4]](image-url)
The most important features of the discontinuities are spacing, persistence, discontinuity sets, block size, roughness and alteration. Those are described below and are included in the characterization and classification systems presented in the next section.

- Spacing and frequency
  Spacing ($x$) is the distance between adjacent discontinuity intersections with the measuring scanline while frequency ($\lambda$) (the number per unit distance) is the reciprocal of spacing (the mean of these intersection distances $\bar{x}$).

$$\lambda = \frac{1}{\bar{x}}$$

- Persistence, size and shape
  Persistence is the extent of the discontinuity in its own plane, including the associated characteristic dimensions and the factors such as the shape of the bounded plane.

- Orientation, dip direction/dip angle
  The dip direction is the compass bearing of the steepest line in the plane. The dip angle is the angle that this steepest line makes to the horizontal plane.

  Thus, as the discontinuity is assumed to be planar, the dip direction and the dip angle uniquely define the orientation of the discontinuity.

- Discontinuity sets
  If discontinuities exist, it is mainly due to mechanical reasons with a tendency to grouping that occurs around preferred orientations associated with the formation mechanisms. That’s why the concept of discontinuity set (parallel or sub-parallel discontinuities) is considered with their amount to characterize a rock mass geometry.

- Block size
  The presence of rock blocks is dependent on the characteristics described above (spacing, persistence, discontinuity sets). Having an idea of the mean block size and the block size distribution is important when performing the excavation and assessing the support needed. Hudson, compares the block size distribution to the particle size distribution used in soil mechanics [4].

- Roughness
  The surface of a discontinuity may be rough even though the discontinuities are assumed to be planar when considering the orientation and persistence analysis. This roughness is defined either by reference to standard charts as in the Q system or mathematically. See Appendix A for a description of the roughness by the joint roughness parameter Jr in the Q system.

- Alteration and filling material
  Weathering, hydro-thermal alteration and shearing cycles may occur through its geological history and affect the discontinuity surfaces. Two processes may be the source of filling material in the discontinuities, either shear movement leading to gouge material, or groundwater transporting material through open joints in the rock mass [5].
Aperture is the perpendicular distance between the adjacent rock surfaces of the discontinuity. For parallel and planar adjacent surfaces, it is constant while for non-parallel but planar adjacent surfaces it would be a linearly varying value and eventually for rough adjacent surfaces this value would be completely variable.

In addition, in situ rock stresses and ground water have an influence on the tunnel stability. That’s why they are represented in the rock mass classification systems as presented in section 2.5, see Figure 2-5 below. The factors influencing the tunnel stability are presented in the blue section in Figure 2-5. Note that the project related features such as geometry of the tunnel or its position have also an influence on the tunnel stability [7].
2.5. Rock mass classification systems

In this section, the main systems to classify and characterize the rock mass quality are described including RMR, GSI, RMi and the Q-system.

**RMR Rock Mass Rating**

The Rock Mass Rating (RMR) system is a geomechanical classification system for rocks, developed by Bieniawski between 1972 and 1973 [8, 9]. It combines the most significant geologic parameters of influence and represents them with one overall comprehensive index of rock mass quality, which is used for the design and construction of excavations in rock, such as tunnels and other underground structures.

The following six parameters are used to classify the rock mass using the RMR-system:

1. Uniaxial compressive strength of rock material
2. Rock quality designation (RQD), see explanations in the section Q-system below
3. Spacing of discontinuities
4. Condition of discontinuities
5. Groundwater conditions
6. Orientation of discontinuities

Each of the six parameters is assigned a value corresponding to the characteristics of the rock. These values are derived from field surveys and laboratory tests. The RMR value is the sum of the six parameters and ranges between 0 and 100.

**GSI system (Geological Strength Index)**

The GSI system was introduced by Hoek in 1994 [10] and is a system used to assess the rock mass strength in combination with the Hoek Brown failure criterion. It is constructed on data from field observations where parameters such as blocks and cracks are observed (see Figure 2-6 below).
Figure 2-6 Evolution of GSI against the block size (From Hoek 1994 [10]).

RMi Rock Mass index

The rock mass index, RMi, was first presented by Palmström in 1996 [11] and has been further developed and presented in several papers. It is a general strength characterization of the rock mass taking into account only its inherent features.

The RMi combines the compressive strength of intact rock $\sigma_i$ and a jointing parameter JP composed of 4 jointing characteristics (block volume, roughness, degree of alteration, length of the joint) combined by empirical relations. The 4 jointing parameters have been combined to express the reducing effect the joints penetrating the rock mass have on the strength of intact rock. RMi can be used for assessing the rock support in tunnels. The RMi value is e.g. applied as input for estimating rock support.

The RMi system has some input parameters similar to those of the Q-system, such as the joint features. The input parameters used can be determined by commonly used field observations and measurements, however, it requires more calculations than the RMR and the Q system. Still spreadsheets have been developed (see www.rockmass.net) from which the RMi value and the types and amount of rock support can be found directly.
**Q system**

The Q-system for rock mass classification is developed by Barton, Lien, and Lunde [12]. It expresses the quality of the rock mass in the so-called Q-value, on which they have suggested support recommendations for underground excavations.

The Q-value is determined with the following formula

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

Where:
- RQD the rock quality designation
- Jn the joint set number,
- Jr the joint roughness number for critically oriented joint set,
- ja the joint alteration number for critically oriented joint set,
- Jw the joint water reduction factor,
- SRF is the stress reduction factor used to consider in-situ stresses according to the observed tunneling conditions.

A multiplication of the three terms results in the Q-value, which can range between 0.001 for an exceptionally poor to 1000 for an exceptionally good rock mass. The numerical values of the class boundaries for the different rock mass qualities are subdivisions of the Q range on a logarithmic scale. See the charts in Appendix C: Q charts [13] and in Figure 2-7.
Figure 2-7 Permanent support recommendations based on Q-values and span/ESR

One of the applications of the Q-system is to evaluate the support requirements.

Barton suggested a relation between the Q-value and the permanent support based on case histories, which can be used to forecast the needs in new tunnel projects.

The chart giving the support opposes what is called the Equivalent dimension in combination with the Q value given in a logarithmic scale. The Equivalent dimension is defined as:
The Equivalent dimension $= \frac{\text{Span or height in m}}{\text{ESR}}$

With ESR being the “Excavation Support Ratio” which expresses safety requirements depending on the use of the excavation. The lower it is, the higher the level of safety will be and the other way around. See Appendix D.

The longer the span or the height of the tunnel will be, the more supports will be needed.
As the Q system is the one that had been used during the whole Northern Link project, the parameters of the Q system are presented hereafter in detail.

**Signification of the quotes in the Q system**

The Q value is influenced by three factors that are expressed by the following quotes in the Q-formula [13]:

- \( \frac{RQD}{J_n} \) represents the size of the intact rock blocks in the rock mass (degree of jointing),
- \( J_r \) represents the shear strength along the discontinuity planes (Joint frictions),
- and \( \frac{J_w}{SRF} \) represents the stress environment on the intact rock blocks and discontinuities around the underground excavation.

**Degree of jointing**

The degree of jointing is determined by the joint pattern (joint orientation and joint spacing). A joint set is formed by near parallel joints (see discontinuity sets in section 2.4). The joint spacing may be reduced considerably along fracture zones.

The rock mass quality will decrease when joint spacing decreases and the number of joint sets increases.

**Joint friction**

Deformations will occur as shear displacement occurs along joints in hard rocks. That’s why this is a significant factor for the rock mass quality and the stability of tunnels. This factor is dependent on joint roughness, thickness and type of mineral fillings.

The more friction there is in the joints, the higher the rock mass quality will be with a better stability of the tunnels. Thus very rough joints, joints with no filling or joints with only a thin, hard mineral filling will have a better stability than a smooth surface and/or a thick filling of a soft mineral (lower friction).

**Active Stress**

The vertical stress in a rock mass commonly depends on the depth below the surface. But in some areas, tectonic stresses and anisotropic stresses due to topography might be more influential. The stability of the tunnel will generally depend on the stress magnitude in relation to the rock strength.

According to NGI 2015 [13], moderate stress is favorable for stability while low stresses are often unfavorable for stability. When zones of weak mineral fillings (clay, crushed rock) intersect rock masses, the stress situation may vary a lot within relatively small areas.
**Detailed description of the Q system parameters**

RQD was introduced by Deere in 1962 [14] and is a modified percent core recovery that incorporates only sound pieces of core that are 10 cm or greater in length along the core axis. See Figure 2-8 for an example of RQD on a drill core.

\[
RQD = \frac{\text{sum of core pieces } \geq 10\text{ cm}}{\text{total drill run}} \times 100, \%
\]

![Figure 2-8 Procedure for measurement and calculation of rock quality designation (RQD) [15]](image)

The rating of Jn is approximately equal to the square of the number of joint sets.

Jr (the joint roughness number) and Ja (degree of alteration of joint walls or filling material) are parameters that should be obtained for the weakest critical joint set or clay-filled discontinuity in a given zone. If the discontinuity with the minimum value of Jr/Ja is favorably oriented for stability, then a second less favorably oriented discontinuity may be of greater significance and its value Jr/Ja should be used when evaluating Q.

Jw is a measure of water pressure which has an adverse effect on the shear strength of joints. This is due to the reduction in the effective normal stress across joints. Jw should correspond to the future groundwater condition where seepage erosion or leaching of chemicals can alter the permeability of the rock mass significantly.
SRF is a measure of 1) Loosening pressure during an excavation through shear zones and clay-bearing rocks; 2) Rock stress in competent rocks; and 3) Squeezing pressure in plastic incompetent rocks, which can be regarded as a total stress parameter.

When using the Q-system, it is difficult to select a single rating for a particular parameter for beginners and even experimented geologists. Therefore, it is recommended that the ratings for different parameters should be given a range in preference to a single value to describe the variation in the rock mass quality. You can find a geotechnical chart in Figure 2-9 proposed by NGI to overcome the problem of selecting a representative rating of various parameters [first version]. By doing histograms from the estimated various quality of each observed parameter (10% poorest, 60% most typical, 30% best or maximum value for example), a weighted value of each parameter is calculated and a corresponding Q-value is determined. In Table 2-1 below, you can find an example of how to apply this method to get a weighted Q-value.

According to information obtained through personal communication with Fredrik Bengtsson, coordinating geologist on the Northern Link, he would do the same but in the end choose to combine the values of the parameters that would lead to the most representative value according to his geologist expertise [second version]. The idea when mapping is to consider that every part of a section has given parameters, leading to a patchwork of different combinations of parameters from which the geologist can then select the ones that would qualify the section the best.

There are two ways of considering the variability of the Q parameters, but we can already say that the second version that has been used in the Northern Link is more prone to be affected by human factors.
2 Literature study - 2.5 Rock mass classification systems

Figure 2-9 Data sheet for recording Q parameters [16]

Table 2-1 Weighted average method for obtaining the Q-Value [16]
2.6. Classification and characterization

The statements presented under are recommendations from Banverket’s guidelines. Those guidelines were originally from Swedpower’s project on rock mass characterization and classification. They are based on tests on the reproducibility of the rock mass quality assessment conducted by a large number of geologists and engineers [17].

When characterizing the rock mass, the RMR- or the GSI-system is recommended. The Q-system can work for a general characterization but it is not recommended when producing strength parameters.

When classifying the rock mass, the RMR- or the Q system are recommended. Not the GSI system. One system is judged not enough to describe a rock mass, two systems should be used and can have a chance to complete each other [17].

Characterization of the rock mass takes into account the rock mass features only. Classification means putting the rock mass into predefined classes. When characterization is performed, only the characteristics of the rock mass is considered in the Q-system by setting the parameters representing the last quote to 1. When characterizing, \( Q = Q_{base} \).

2.7. General Procedure for tunnel mapping

2.7.1. Tunnel mapping

In this part some complements from the document "Guidelines for mapping of tunnels, clarification (Riktlinjer för kartering av tunnlar, förtydligande)" are presented [19]. Even though they were elaborated for the Cityline project which was started after the Northern Link, they had similar procedures which have been confirmed by geologists who worked on the Northern Link. They help to understand better how the mapping was done during the Northern Link project.

- The work is to be performed for longer continuous tunnel stages if the geology and the contractor’s planning permits it. A stage of 20-30 meters is recommended instead of mapping after each round.
- If the work is carried out on longer mapping stages, the geologist must be continuously updated on the rock mass quality at the tunnel front, which means that it may be appropriate for frequent inspections of the tunnel front even if no mapping is necessary for the moment."

Concerning the rock mass:

- "By mapping tunnel stages longer than 1-2 salves (for example stretches of 20-30 m), Q parameters should be assessed in 5 meter intervals to obtain systematic classification.
- The point adjustment for the parameters in the Q-system should be carried out according to the instructions of the system but the geologist should also be aware of the system’s limitation with respect to proposals for how support are derived.
- The documented Q-values should be related to the current stress situation in the form of rock cover (SRF) and the account of crossing geometries (Jn).
• The SRF should be set to 1 except where the rock cover is less than half the width of the tunnel or when the rock cover is less than 5 m independent of the tunnel width. In this case the SRF is set to 2.5. Those corrections are done for the Q-value (classification).

• Jn is corrected in level crossings and junctions in which a tunnel is tangent to another (a tunnel cut through the top or bottom of another) and in rising-positions (ramp connecting room).

For tunnel intersections (crossroad) the correction of Jn is Jn x 3 while for portals it is Jn x 2.

Small escape routes and other similar small tunnels that connect to the track or service tunnel (three-way intersection) imply no correction of Jn except for the wide and high ones.

• "Double Effect" of low parameters values in the Q-index should be avoided; Jn and SRF in some cases cover up the same features that will not be described and graded in both parameters.

• RQD can be estimated by mapping imaginary cores along the drill pipes in the contour or along section lines across the tunnel contour. By doing estimations along such "lines", the geologist should be careful of how different orientations in the tunnel affect the perception of the frequency of the discontinuities and modify it if necessary. Another way to estimate the RQD is by selecting a typical area of the contour to represent the rock mass in a tunnel section (or a mapping interval) within which the number of discontinuities are added together and converted to a RQD value.

• There are two ways of considering the numeric parameters as presented in section 2.5. In the case of the Northern Link, numeric parameter values were not to be interpolated when doing the classification with Q-index. That’s why, in our case, non-tabulated parameter values are avoided even though the geologist thinks that the rock mass gets a fairer final Q-value with "suited" parameter values.

• The mapping geologist is recommended to document the variation of each parameter within the classification system used. Barton’s method to plot histograms to illustrate the variation within the individual parameters of the Q-index is a way to work but a suggestion is to note the value of the parameter such as Jn = 9 (6-12), which means that the characteristic parameter value is 9, but the rock mass exhibits a variation between 6 and 12 for the parameter in question.

The main reason to document the variation is to make determinations more easily about the appropriate support when the rock mass is characterized, on the border between two types of support classes. For the Northern Link, this does not appear in the excel sheet containing the data from the mapping. It was most probably done in the head of the geologist, so it’s not possible to have a grasp of that for the Northern Link’s project.

2.7.2. Uncertainties

Figure 2-10 illustrates the different phases an underground project goes through and their corresponding uncertainties; from a high level of uncertainty to a low one and from a minimum knowledge of the rock quality to a maximum knowledge.
The different stages of the rock mass quality assessment show a similar evolution: from pre-investigation (minimum knowledge through drill cores and other localized studies) through engineering geological forecast (extrapolation of the rock quality based on the pre-investigation) to the actual mapping (actual rock quality obtained).
3. The Northern Link project

3.1. Description of the studied case

Tunnel contracts in the Northern Link

As said earlier, among the 5 km of motorway which form the Northern link, 4 km are tunnels and together with the southern link (Södra länken) and Essingeleden, they constitute a part of an even longer incomplete ring road around Stockholm’s inner city.

The part of the Northern Link which is between Karlberg and Norrtull was already initiated in 1991 (the part on the left of NL12) but it was only in 2006 that the construction work of the remaining part between Norrtull and Värtan/Värtahamnen started. The whole project was estimated to be finished by 2015. When finished, it will allow Valhallavägen to be relieved from traffic congestion and the road between Björnnäsvägen and Lill-Jansskogen to be closed to car traffic. The project consists of many different construction contracts which were tendered by Vägverket (Trafikverket nowadays). This master thesis is going to focus on the rock tunnels which are mainly located under Nationalstadsparken. They consist of the four contracts NL22, NL33, NL34 and NL35, which lie under Bellevue, Albano, Teknikhöjden and Värtan (See Figure 3-1) [20].

*Figure 3-1 The Northern Link section and the project’s contracts [17].*
Table 3-1 *Contracts studied and the different parties involved.*

<table>
<thead>
<tr>
<th>Contract</th>
<th>NL 22</th>
<th>NL 31</th>
<th>NL 33</th>
<th>NL 34</th>
<th>NL 35</th>
</tr>
</thead>
<tbody>
<tr>
<td>Type</td>
<td>Concrete and Rock tunnel</td>
<td>Concrete and Rock tunnel</td>
<td>Rock tunnel</td>
<td>Rock tunnel</td>
<td>Rock tunnel</td>
</tr>
<tr>
<td>Location</td>
<td>Bellevue</td>
<td>Bellevue</td>
<td>Albano</td>
<td>Teknikhöjden</td>
<td>Värnan</td>
</tr>
<tr>
<td>Contractor</td>
<td>Bilfinger &amp; Berger</td>
<td>Consortium Züblin &amp; Pihl</td>
<td>Veidekke</td>
<td>Veidekke</td>
<td>Consortium Hochtief &amp; Oden</td>
</tr>
<tr>
<td>Design</td>
<td></td>
<td>SWECO</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Geology</td>
<td></td>
<td>SWECO</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

In the Northern Link, even though the contractors were different, the design and the mapping was performed by SWECO (See Table 3-1). The team in charge of doing the mapping consisted of 5 geologists and a coordinating geologist, Fredrik Bengtsson, who had to ensure that all the geologists were following the same method to map the tunnel (See Figure 3-2).

![Figure 3-2 The mapping team in the Northern Link](image)

It should be noted that, in comparison, there were geologists from different companies involved in the Cityline to perform the mapping of the tunnel. Note also that for the Cityline, there was also different designers, engineers and geologists involved from different companies while for the Northern Link only the team of 6 geologists from SWECO was involved. Thus for the Cityline, a “rock workgroup” was launched to define the guidelines to follow for the engineering geological forecast. This led to the documents [3].

**History of the Northern Link**

The Northern Link (the parts of Norrtull, Roslagstull, Frescati and Värnan) was designed between 1994 and 1997. Two parts of the tunnel work were intended to be started in 1996 before the project was put on hold in the spring 1997. The project was by that time divided in two parts, Norra Länken 1 (NL1), which contained the Norrtull part and Norra Länken 2 (NL2), which consisted of the parts of Roslagstull, Frescati and Värnan. The different main parts (NL1 and NL2) were designed by two different consulting groups. The re-design of the Northern Link was started during the spring 2004. The part K3 of the Northern Link consists essentially of the eastern part of the “old” NL1 (east to the
The Northern Link project - 3.2 Mapping

3.2. Mapping

3.2.1. Mapping of drill cores

The drill cores from the Northern Link have been mapped differently depending on which part they belonged to. The ones from the eastern part (K3) have been classified by estimating the parameters of the Q-system for every meter of the drill core while those from the western part (K1) have been classified according to what we can call “fast mapping”, meaning that sections longer than one meter having the same characteristics are given the same parameters. See figure 3-3 to identify K1.
The mapping of every meter has the advantage to make the data already comparable from one drill core to another and maybe avoid the differences of interpretation from one geologist to another.

Figure 3-3 K1 and K3

Note that it’s classification that we are talking about here as the SRF-value is not set to 1 by default.

The permeability of the rock mass is then obtained for every three meters based on water pressure tests. Finally, the drill core is mapped in geological terms (rock type, minerals etc.). Picture on one of the boxes can be seen in Figure 3-4.

For four drill cores, BIPS images were also provided, which may help the geologist to make his assessment of the rock quality.

Charts to assess the Q-value can be found in Appendix C.

Figure 3-4 Drill core from NL34 (K3KBH7)
3.2.2. Engineering geological forecast

The way it was obtained for this project is missing however it is an interpretation of all the geological data and results in a forecast of the ground conditions and Q-value along the tunnel alignment as described in section 2.2.

3.2.3. Tunnel mapping

The Tunnel mapping is the third and final assessment of the characterization and classification of the rock mass quality. It is also the last step in the process that is evaluated in this thesis. After interviewing three geologists that were involved in the mapping of the Northern Link tunnel and by using the document written by Fredrik Bengtsson, “arbetsgång - systematisk kartering och förstärkningsavrop” [21] (process – systematic mapping and reinforcement suborder) the following description of the method used to perform the tunnel mapping in the Northern Link is presented below.

The recommendations for the Cityline available in the document “Guidelines for mapping of tunnels - Basis for design of construction documents” (“Riktlinjer för kartering av tunnlar – Underlag för projektering av bygghandling” dokument nr 9564-13-025-016 [22]) were also used as foundation to understand how the geologist proceeded to perform the mapping of the rock quality in the Northern Link.

After studying this document with the geologists (3 out of 5) who mapped the rock tunnel contracts of the Northern Link, a description of how the work had been done is presented below:

- Mapping and classification shall be performed during construction by an engineering geologist with experience from previous tunnel mapping. The engineering geologist establishes proposals for permanent support. Based on the conducted mapping and support proposals, he then shows the client the chosen permanent support.

- Mapping is normally done after each blast round (this was followed in the end of the mapping for the contract NL35), but at least after every third blast round (was followed in the beginning of the mapping for NL33). The excavated rock for one blast round being approximately 5 to 6 m. Though, doing it every third blast round (from 15 to 30m) is recommended to have a good overview and rational excavation.

- Before the mapping starts, a base for the mapping that is appropriate to the present tunnel section is established as well as a plan with unfolded walls. On the template, the tunnel front and potentially the tunnel walls, the ceiling and the initial springing should be distinguishable.

- Discontinuities and structures are drawn. The degree of water is also indicated, from moisture to flowing. The structures are numbered (continuously in each mapping sheet) and
the structures are accounted for in detail regarding distance between discontinuities, filling material and filling thickness.

- The mapping begins with a visual assessment of the rock properties. The discontinuity surfaces’ properties and potential mineral filling are examined by scraping the surface with a knife or with the nails. Just as in the Guidelines for core mapping, the following properties are documented: 1. location of discontinuity 2. Discontinuity distance 3. Discontinuity roughness 4. Discontinuity width 5. Discontinuity filling 6. Water flow 7. Number of discontinuity groups.

- Observed discontinuities are drawn into a plan. Discontinuity mapping is updated after each blast round. All discontinuity mapping of a blast round is made taking into account the previous round being mapped. If small cracks form a wedge with each other or if many short cracks constitute a zone, they should be mapped.

See Appendix F for images describing this work.

From personal communication with Fredrik Bengtsson, coordinating geologist for the Northern Link, he points out the necessity to have the 15m closest to the tunnel front to be clear (from work vehicles or blasted rock for example) so that the engineer geologist can perform the mapping [21].

In addition, some more recommendations formulated in “arbetsgång - systematisk kartering och förstärkningsavrop” [21] were:

“If you are not completely sure, avoid to reply on strengthening class until you have analyzed the mapping and Q classification in a quiet place.”

“Study the mapping of the discontinuities and compare it with Q-values for a comprehensive view, and keep in mind that the Q value does not say everything.”

3.3. Comparison how individual parameters are assessed in the Q-system for the pre-investigation, the engineering geological forecast and the tunnel mapping

In order to create an overview of how the different parameters were assessed in the establishment of the engineering geologic forecast (where mapping of drill cores is an essential part of the investigated materials) and tunnel mapping, the recommendations from the following manuals “Guidelines for core logging and preparation of engineering geological and rock engineering forecast - Basis for planning of construction document” [3] and” Guidelines for mapping of tunnels - Basis for design of construction documents ” [22] have been compiled below and validated by a geologist that was part of the mapping team:
3 The Northern Link project - 3.3 Comparison how individual parameters are assessed in the Q-system for the pre-investigation, the engineering geological forecast and the tunnel mapping

Table 3-3 Comparison of how the parameters are assessed for the establishment of the engineering geological forecast (when mapping borehole is a form of pre-investigation) and the tunnel mapping for the Q system

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Drill core Mapping/ Engineering geological forecast</th>
<th>Tunnel mapping</th>
<th>Comment by the geologist</th>
</tr>
</thead>
<tbody>
<tr>
<td>RQD</td>
<td>Mapping according to ISRM. Calculations regarding Mean fracture number/meter</td>
<td>Estimated along an imaginary bore pipe along the tunnel</td>
<td></td>
</tr>
<tr>
<td>Jn</td>
<td>The number of visible fracture groups is counted. Polepoint analysis if necessary</td>
<td>Based on assessment of visible fracture groups</td>
<td></td>
</tr>
<tr>
<td>Jr</td>
<td>Roughness description according to figure. Based on the fracture with the lowest shear strength within the mapped interval (1m)</td>
<td>Based on the worst fractures regarding the tunnel stability</td>
<td>average/meter. Can be quite wrong depending on the scale</td>
</tr>
<tr>
<td>Ja</td>
<td>Fit of the core pieces. Based on the fracture with the lowest shear strength within the mapped interval (1m)</td>
<td>Based on the worst visible fracture regarding the tunnel stability</td>
<td>problem, filling material could have been washed away when drilling drill cores</td>
</tr>
<tr>
<td>SRF</td>
<td>Depending on the specific rock condition (consistency = 1 when doing characterization)</td>
<td>Set to 1 except when the rock cover is lower than 5m or half tunnel width.</td>
<td></td>
</tr>
<tr>
<td>Jn korr</td>
<td>See Jn above. (consistency = 1 when doing characterization)</td>
<td>Is corrected in intersections with tangent tunnels or ramp</td>
<td></td>
</tr>
<tr>
<td>Jw</td>
<td>Based on water loss measurements (consistency = 1 when doing characterization)</td>
<td>Estimated leakage after the resulting injection</td>
<td></td>
</tr>
</tbody>
</table>
4. Methodology

4.1. Introduction

This chapter describes the methods that have been used to identify possible systematic discrepancies between the pre-investigation, geological forecast and the tunnel mapping when classifying and characterizing the rock mass quality.

As previously described, the aim of the thesis was to investigate the following questions with respect to the process described above:

- Are there systematic discrepancies when establishing the characterization and classification of the rock mass in the different steps?
- If there are any, what do they depend on?

This work is a continuation of the work done by Ingrid Kjellström on the Stockholm Cityline Project [1] and for this reason also has similar research questions and methodology. Due to this, the results from both projects can be comparable. The methodology contains the following main steps:

1. Description of expected discrepancies between pre-investigation, geological forecast and the tunnel mapping
2. Analysis of available data in the pre-investigation, geological forecast and the tunnel mapping
3. Comparison of results between pre-investigation, geological forecast and the tunnel mapping in general with respect to the Q-value and for each parameter in the Q-system.
4. Discussion of the results

In addition to this, it should be observed that it is assumed that the large amount of data from the tunnel mapping and from the drill core mapping has created a statistically representative view for describing the average value of the rock mass quality in the project. The results presented in this thesis are based on this assumption being correct.

4.2. Expected variation of the Q system parameters between the pre-investigation/geological forecast and the actual mapping.

Given the used routines to assess the rock mass quality in the different steps, the parameters in the Q-system are expected to vary to some extent between the drill core mapping and the mapping performed in tunnels. Below it is described how each parameter in the Q-system are assessed when drill core mapping and tunnel mapping is performed.

- **RQD:** Performed on every meter when doing the drill core mapping/forecast work. In the tunnel mapping, the same evaluation is performed but is more subjective. The mapping in the tunnel by the engineering geologist is performed along “an imaginary drill core”.

This parameter can be expected to be both higher or lower in the tunnel mapping.

The parameter can get a lower value when doing the tunnel mapping, since blasting can create an increased number of discontinuities on the rock surface.
Another reason that could explain why the rock mass could be evaluated differently for this parameter is that a poor quality rock can get a larger influence than a good quality rock in the tunnel scale compared with the pre-investigation of the drill core which is evaluated each meter – the so called scale effect.

However, it may also get a higher value in the tunnel mapping, when considering how the drill cores are treated once extracted. Indeed, they might get broken during transportation to the location where the mapping is to be performed, resulting in a lower RQD value when considering drill cores than the actual RQD value. However, an experimented geologist should be able to notice that a crack is due to transport. The same reasoning is also valid if the cores are broken while drilling, which could cause an imaginary high degree of discontinuities if this is not recognized.

Considering the two tendencies of the evolution of RQD, it can be assumed that they tend to compensate one another in one way or the other, resulting in a small positive or negative deviation of the RQD from the pre-investigation to the actual mapping.

- Jn – Joint set number: The parameter depends on the number of joint sets of the rock mass and this can be assumed to raise from the engineering geological forecast to the outcome, i.e. that the rock is evaluated to be worse as the number of joint sets rises. This could be because of the risk that more discontinuities can appear after blasting than when looking at drill cores. Also, it might be difficult to identify different joint sets from a drill core only if a structural analysis of the joint sets is not performed over the geological domain, resulting in a too low number in the drill core mapping.

- Jn kor – Corrected joint set number: A correction is applied to Jn given the geometry of the tunnel section analyzed. Jn is then multiplied by a factor 2 or 3 in those sections.

- Jr – Joint roughness number: The parameter depends on how rough the surfaces of the discontinuities are. The discontinuities can be assumed to appear smoother in the tunnel scale – presenting a lower degree of roughness when the tunnel mapping is performed compared to the mapping of drill cores. Jr is split in two scales: a large scale “waviness” and a small scale “roughness” which are very difficult to estimate in small borehole cores as the large scale waviness cannot be observed.

- Ja – Joint alteration number: The parameter depends on the alteration of the discontinuity surface or the filling material. When mapping drill cores, an average value for each meter is calculated while when mapping the tunnel, a bigger area in the sections is considered and the lowest value is considered. Filling material is also believed to be more difficult to identify in drill core mapping, since the filling might be washed out when drilling, which may lead to that the parameter is assumed to obtain a higher value at the tunnel mapping.

- Jw – Joint water reduction factor: Is assumed not to change from the engineering geological forecast to the tunnel mapping. Though, it cannot be properly compared between the steps of pre-investigation (mapping of the drill cores) and the tunnel mapping because it constantly gets a rating of 1 when mapping the drill cores and characterizing the rock mass.

- SRF – Stress Reduction Factor: Assumed to be reduced from engineering geological forecast to the outcome as the environment is properly taken into account in the outcome (note that it cannot be compared between the pre-investigation of the drill cores and the tunnel mapping because it constantly gets a rating of 1 when mapping the drill cores/characterizing the rock mass).
4.3. Available data

The work started by gathering the data from the Northern Link for the different steps of the planning, including data from drill cores from the pre-investigations and data from the engineering geological forecast. Data was also gathered from the tunnel mapping. The data were collected from the following contracts NL22, NL31, NL33, NL34, NL35 (contracts corresponding to the part K3) and from some old drill cores made in the part done at an earlier stage (K1). The table 4-1 below presents the amount of data that was available for each tunnel and stage.

Table 4-1 Summary of the investigated meters of borehole and tunnel meters forecasted and mapped for each contract.

<table>
<thead>
<tr>
<th>Contract</th>
<th>Mapped (m)</th>
<th>Forecasted (m)</th>
<th>Drill core (m)</th>
<th>Proportion of meters of drill cores per forecasted meter (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Old contracts (K1)</td>
<td></td>
<td></td>
<td>356,15</td>
<td></td>
</tr>
<tr>
<td>NL22</td>
<td>81</td>
<td></td>
<td>-</td>
<td></td>
</tr>
<tr>
<td>NL31</td>
<td>98</td>
<td></td>
<td>-</td>
<td></td>
</tr>
<tr>
<td>NL33</td>
<td>2292</td>
<td>2292</td>
<td>110</td>
<td>4,8</td>
</tr>
<tr>
<td>NL34</td>
<td>897</td>
<td>897</td>
<td>97</td>
<td>10,8</td>
</tr>
<tr>
<td>NL35</td>
<td>1932</td>
<td>1932</td>
<td>653</td>
<td>33,8</td>
</tr>
</tbody>
</table>

4.3.1. Available information from the drill cores

The characterization and classification of the drill cores is based on 329 and 863 meters of drill cores from K1 and K3 respectively. For the old contracts the Q-value was not assessed every meter; instead it was assessed when there was a change in the quality of the borehole. However, for the other contracts they were evaluated for every meter.

SWECO performed the pre-investigation work before the construction started. The core logs of the old (K1) part were analyzed by VBB VIAK (SWECO) and BBK AB. See Appendix G for an example of the drill core mapping.

The following information was available:

- Length of drill core (m)
- Q parameters: RQD, Jn, Jr, Ja, Jw, SRF
- Q-value and Qbase
- Position of drilling
- Orientation of the drilled holes
4.3.2. Available information from the engineering geological forecast

The engineering geological forecast for K3 was performed for 5234 m of the Northern Link contracts (K3) and were performed by both SWECO and Nitroconsult.

For the forecast, only the final Q-value was directly given without the parameters that led to it. This was also the case on the drawings of the engineering geological forecast.

The following information were available from the forecast:

- Tunnel parts and location
- Length (m)
- Rock type
- Q-value

4.3.3. Available information from the tunnel mapping

Data from the tunnel mapping were available for K3 for a total length of 5234 m. The tunnel mapping was performed by SWECO’s geologists.

The following information were available:

- Date when the mapping was performed
- Tunnel parts and location
- Start section and end section giving the length (m)
- Rock type
- Q parameters: RQD, Jn, Jr, Ja, Jw, SRF
- Exact Q-value
- Qbase
- Q-group (for example the rock mass belongs to the Q group 1-4 if Q is included in a range of 1 to 4)
- Remarks
- Rock cover
- Ordered support
- Forecasted support
- Quantity and type of support

4.4. Handling of the input data

Once all the material had been gathered, the individual Q parameters were compared to their corresponding values in each step (drill holes, forecast, tunnel mapping) with the goal to find which parameters that differed between the different steps.
Normalization

The first step was to make the different steps comparable with each other. The tunnel mapping was done every 5m, while some of the drill core mapping was performed every meter. Thus it was chosen to normalize all the measurement zones to a meter scale.

Parameter values

Every time the Qbase was evaluated (for the drill cores and for the tunnel mapping-characterization) the parameters Jw and SRF were set to 1.

Line diagrams

For every step, the Q values were plotted in a histogram to get an idea of how the dispersion evolves between the different steps. The classification system with the Q-value was split into four rock classes, each leading to a corresponding support class. The same ones have been kept when plotting the Q-value (Berg Klass 1 - BK1 > 10, BK2 = 4-10, BK3 = 1-4, BK4 < 1).

Statistical toolbox

The most common measure of central tendency used in engineering geology are the mean value and the mode, while one of the most common measure of central dispersions is the standard deviation of the frequency distribution.

The mean is an arithmetic average of a set of data and also is a center of gravity of the probability distribution along the x-axis. The mean ($\bar{x}$) of the set of “n” data ($x = x_1, x_2,...,x_n$) is given by:

$$\bar{x} = \frac{1}{n} \sum_{i=1}^{n} x_i$$

The mode is the most common value or most likely value of data sets.

The term standard deviation ($\sigma$), which is very common in statistical analysis, is the root-mean-square (rms) of the difference between a particular data within the set of data and their mean, and is expressed as:

$$\sigma = \sqrt{\frac{\sum_{i=1}^{n}(x_i - \bar{x})^2}{n-1}}$$

Theoretically, the denominator for the calculation of the standard deviation should be (n) rather than (n-1). However, (n-1) is generally used for the finite number of samples to correct the statistical bias [23]. Accordingly, it is logical and valid to use this expression in rock engineering.

The variance or second moment about the mean ($\bar{x}$) of the set of data is the square of standard deviation ($\sigma$) and is given by: $\sigma^2$.

The coefficient of variation (COV) of a set of data is defined as the standard deviation ($\sigma$) divided by the mean ($\bar{x}$):
\[ COV = 100 \times \frac{\sigma}{\bar{x}} \]

COV is dimensionless and is particularly useful to measure variation or uncertainty of a parameter. A small value of the COV represents a small level of variation or uncertainty.

**Most likely values - modes**

Some geologists follow the rule when doing the tunnel mapping that one should never take the average of the parameters on a section but to choose the most likely value of it as seen earlier in section 2.5. This was the case in the Northern Link.

The comparison of the average values between different steps are good to have in order to obtain an idea of how the parameters end up evolving between the mapping of drill cores and the mapping in the tunnels. However, to be able to draw conclusion about the values and interpret them from a geological perspective, modes are also going to be used.

Note that when dealing with the three quotes in the Q-system, they are first calculated for every single meter and then the mode of the quote is taken. The mode of the quote is not necessarily the quote of the modes.

**4.5. Comparison between the different steps**

The different values of the parameters for each step - drill core mapping (Pre-investigation), engineering geological forecast and tunnel mapping - are compared to one another considering their mean value, mode (most likely value) and standard deviation.

The values of these parameters were calculated with the aim to find where the possible differences occur.

The work was mainly divided in two steps:

- First, some general trends of the variation of the Q-value between the engineering geological forecast and the mapping are drawn by establishing the rock class frequency distribution, the accuracy and over/underestimation of the rock mass quality, and by estimating the repartition of the forecasted rock classes when mapped (what will be the rock class obtained when mapping the section forecasted to have a rock class equal to BK1?). See Appendix H for a description of the normal distribution.
- In the next step, it was investigated how the parameters in the Q-system vary between the pre-investigation and the outcome from the tunnel mapping within the Northern Link and between the Northern Link and the expectations formulated in the section 4.2. See Appendix I for the methodology used to treat the Q-values.
5. Results

In this chapter, the results of the analysis are presented. First, general trends are presented, i.e. if any discrepancies between the different steps in the analysis can be seen. Thereafter, the observed differences are presented for each parameter in the Q-system.

5.1. First step of the analysis

5.1.1. Trend for each step

In the results, it can be seen that the Q value forecasted is more optimistic than the Q value actually mapped, see Figure 5-1. The mapped tunnel presents more low quality rock than expected.

![Comparison distribution of Q value from forecast and mapping](image)

*Figure 5-1 Comparison of the distribution of the Q value from the engineering geological forecast and the mapping (Approx Mapping means that each exact Q value calculated was linked to the Q-group it belonged to. For example, if Q = 2.2 then the Q-group would be 1 - 4) Total number of samples for each data set: 5219 (m) for the forecast, 5234 (m) for the mapping.*

The rock classes Q<1 and Q>10 had been overestimated 2% and 9,2% respectively, while the rock classes Q = 1 – 4 and Q = 4 – 10 had been underestimated 9,1 and 2,3% respectively, as pointed out in Figure 5-1.
5 Results - 5.1 First step of the analysis

The frequency here is the percentage of tunnel meter belonging to one class on the whole tunnel length available.

The rock quality forecasted has a tendency to be “worse” than the one assessed for the boreholes. The orange arrow in Figure 5-2 shows that there may be a tendency of the geologist to be conservative when expressing the forecast even though 70% of the drill cores were assessed to be of very good rock. That may be due to the other factors taken into account when assessing the engineering geological forecast from all the pre-investigation results. The blue arrow in Figure 5-2 shows the consequence of this conservative behavior. The meters of tunnel that are not very good rock are then either of rock class $Q=1-4$ or $Q=4-10$. As can be observed in Figure 5-2, the actual mapping is quite close to the forecasted one, even though it’s still optimistic.

5.1.2. Total accuracy in percent

In this section, every single meter forecasted and mapped is compared to one another. The result is the estimation of the total accuracy of the forecast (in %) obtained by summing the number of meters of tunnel that are well forecasted and dividing it with the total number of forecasted meter.

An overestimated rock is a rock forecasted to be of good quality (BK1 for example) but end up in the actual mapping to be less good than expected (BK3 for example). Underestimating it, is on the contrary expecting it to be bad but it ends up to be better than expected.

Table 5-1 Accuracy, overestimation and underestimation in % of total tunnel length.

<p>| | |</p>
<table>
<thead>
<tr>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Accuracy (%)</strong></td>
<td>40,7</td>
</tr>
<tr>
<td><strong>Overestimated (%)</strong></td>
<td>36,8</td>
</tr>
<tr>
<td><strong>Underestimated (%)</strong></td>
<td>22,5</td>
</tr>
</tbody>
</table>
The overestimated rock is 36.8% of the total rock mass. It’s relatively high when compared to the results in “Osäkerheter i bergprognoser vid utförande av infrastrukturtunnlar” trafikverkets publ nr 2012:213 [24] for which the accuracy was 53% and 58% for the two studied cases. This points out the difficulty to forecast the geographic occurrence of the different Rock classes.

5.1.3. Forecast uncertainty for the whole project

Comparison of $\log_{10}Q_{\text{Progn}}$ and $\log_{10}Q_{\text{Mapp}}$ for the whole project – Average and standard deviation.

In Appendix M, the values of the average and standard deviation of the $\log_{10}$ of $Q_{\text{mean}}$ for both the actual mapping and the engineering geological forecast for each section and for the whole Northern Link project are listed. Having the mean value and the standard deviation for a section makes it possible to generate normal distribution graphs. This makes the comparison between different cases easier (See in Appendix H for some remarks about the normal distribution).

In table 5-2, values of the average and of the standard deviation of the $\log_{10}$ of the $Q_{\text{mean}}$ for both the mapping and the engineering geological forecast for the whole project are presented.

Table 5-2 Values of the average and of the standard deviation of the $\log_{10}$ of the $Q_{\text{mean}}$ for both the mapping and the forecast for the whole project

<table>
<thead>
<tr>
<th>Section</th>
<th>Average Mapping $\log_{10}Q_{\text{mean}}$</th>
<th>Standard deviation</th>
<th>Average Forecast $\log_{10}Q_{\text{mean}}$</th>
<th>Standard deviation</th>
</tr>
</thead>
<tbody>
<tr>
<td>The whole project</td>
<td>0.77</td>
<td>0.44</td>
<td>0.83</td>
<td>0.47</td>
</tr>
</tbody>
</table>

To understand the significance of the $\log_{10}Q$ better, Table 5-3 gives the correspondance between the $Q$ value and its $\log_{10}$, as well as the correspondance between the rock class (BK) and the reinforcement class. The information of this table was extracted from “justering av förstärkningklasser” [25] and helps to understand what the reinforcement classes correspond to. In our case, the tunnels are included in the category “span of 16-23m”.
5 Results - 5.1 First step of the analysis

Table 5-3 Correspondence table of rock class and reinforcement class

<table>
<thead>
<tr>
<th>Q</th>
<th>Log(_{10})Q</th>
<th>Q</th>
<th>Rock class (BK)</th>
<th>Reinforcement class</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.1</td>
<td>-1</td>
<td>&lt;1</td>
<td>BK4</td>
<td>VD</td>
</tr>
<tr>
<td>1</td>
<td>0</td>
<td>1-4</td>
<td>BK3</td>
<td>VC</td>
</tr>
<tr>
<td>4</td>
<td>0.6</td>
<td>4-10</td>
<td>BK2</td>
<td>VB</td>
</tr>
<tr>
<td>10</td>
<td>1</td>
<td>&gt;10</td>
<td>BK1</td>
<td>VA</td>
</tr>
<tr>
<td>40</td>
<td>1.6</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Figure 5-3 Comparison of Log\(_{10}\)Qprogn and Log\(_{10}\)QMapp for the whole project

Studying the Log\(_{10}\) of Q for the whole project, see Figure 5-3, the results show that the engineering geological forecast and the mapping of the Q value are quite close (the relative difference of the average and the standard deviation between the engineering geological forecast and the mapping are smaller than 10%). So the same kind of result as in figure 5-1 is obtained, as expected.
5.1 First step of the analysis

**Log\(_{10}\)QProgn - Log\(_{10}\)QMapp for the whole project**

To have an idea of how good the forecast of the rock quality is, and to have a result comparable to the one produced by [26] Log\(_{10}\)Qprognos – Log\(_{10}\)Qmapping for the whole project has been computed.

*Table 5-4 Average and standard deviation of Log\(_{10}\)Qprogn - Log\(_{10}\)Qmapp*

<p>| | |</p>
<table>
<thead>
<tr>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Average</strong></td>
<td>0.06</td>
</tr>
<tr>
<td><strong>Standard deviation</strong></td>
<td>0.53</td>
</tr>
</tbody>
</table>

*Figure 5-4  Normal distribution of Log\(_{10}\)Qprogn - Log\(_{10}\)Qmapp*

The figure above shows that the average value of the difference is very close to 0 (0.06), which means that in average the estimation of Q is quite accurate (the closest the difference Log\(_{10}\)QProgn – Log\(_{10}\)Qmapping is to zero, the better the forecast is) but the standard deviation is quite big – 0.532. The result is the same as the one obtained in Figure 5-3, but gives an additional information about the spreading of the data.

68% of the values Log\(_{10}\)Qp – Log\(_{10}\)Qm = Log\(_{10}\)(Qp/Qm) are included in the range Mean - Standard deviation and Mean + Standard Deviation.

\[-0.4722 < \log_{10}\left(\frac{Q_p}{Q_m}\right) < 0.5918\]

By applying 10\(^\wedge\) to the relation,

\[0.34 < \frac{Q_p}{Q_m} < 3.91\]

it results in:

\[0.34 \times Q_m < Q_p < 3.91 \times Q_m\]

valid for 68% of the whole tunnel.
Thus there is 68% chance that a rock section that has been mapped with a quality $Q_m = 6$ (BK2) would be obtained for a section that was forecasted with a rock quality $2 < Q_p < 23.46$. This point out the fact that a rock mapped with a rock class BK2 could actually be forecasted to be part of three different classes (BK1, BK2 and BK3). This is also observable in section 5.1.4.

In “Förundersökningar vid Undermarksprojekt - Osäkerheter och deras hantering” [26] the authors have defined a “good” and a “bad foundation”. A “good foundation” in opposition to a bad one has a difference value close to 0. Moreover, the smaller the value of the standard deviation is, the closer to each other are the forecast and mapping of the Q value. Indeed, if the amount of pre-investigation is good enough, the difference between the mean values mapped and forecasted should be negligible (difference close to 0) and there should also be less deviation from the mean value of the difference.

Figure 5-5 Comparison between forecasts with respectively "good" and "uncertain" foundation [26].

5.1.4. Mapped rock mass quality per forecasted rock mass quality

This section presents the way the mapped rock mass qualities are divided for each forecasted rock mass quality. This has been obtained by calculating the proportion of each mapped rock class for a given forecasted rock class. See Table 5-5 and Figure 5-6 below for the results concerning the Northern Link.

The 4th row of the table 5-5 below indicates that among the rock mass that has been forecasted with a rock class (BK) $x = 1$ for example, 43.7% has been mapped with a rock class (BK) $y = 1$. This table was used to generate the figure 5-6.
5 Results - 5.1 First step of the analysis

Table 5-5 percentage distribution of the mapped rock types for different forecasted rock types for the Northern Link

<table>
<thead>
<tr>
<th>If prog x</th>
<th>If prog 1</th>
<th>If prog 2</th>
<th>If prog 3</th>
<th>If prog 4</th>
</tr>
</thead>
<tbody>
<tr>
<td>Mapp BKy</td>
<td>BK1</td>
<td>BK2</td>
<td>BK3</td>
<td>BK4</td>
</tr>
<tr>
<td>Total (m) prog x and mapp BKy</td>
<td>900</td>
<td>882</td>
<td>278</td>
<td>0</td>
</tr>
<tr>
<td>% prog x and mapp BKy</td>
<td>43.7</td>
<td>42.8</td>
<td>13.5</td>
<td>0.0</td>
</tr>
<tr>
<td>Total prog x</td>
<td>2060</td>
<td>1773</td>
<td>1169</td>
<td>232</td>
</tr>
<tr>
<td>Total length</td>
<td>5234</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Figure 5-6 Percentage distribution of the mapped rock types for different forecasted rock types for the Northern Link

The graphs in Figure 5-6 confirm that it is difficult to assess a forecasted value of the Q-value that is correlated with its actual mapped value. The rock class is always between at least two rock classes if it has been forecasted to be of a given class. In the previous section, the result showed that a rock mapped with a certain class could have been forecasted with three different classes. Now the result
is the following: a rock that has been forecasted from a given class (for example BK1), could end up being mapped in three different classes (43.7% in BK1, 42.8% in BK2 and 13.5% in BK3 in our case).

A first conclusion from the first step of our analysis can be drawn: Managing to make the engineering geological forecast and the actual mapping match for every single meter is difficult but we can still acknowledge that the overall correlation between the engineering geological forecast and the actual mapping is quite good for the Northern Link project.

5.2. Second step of the analysis

5.2.1. Influence of the last quote – comparison of QBase and Q

As presented in section 2.5, \( Q = Q_{\text{base}} \times (Jw/SRF) \). Basically, the difference between Q and Qbase is that the last quote has been taken into account for Q (classifying), while it has not been taken into account for Qbase (characterizing).

As can be observed in Figure 5-7, there is a significant difference between the mapped value of Qbase and the one from the boreholes (difference of one class).

![Distribution of QBas](image)

*Figure 5-7 Distribution of Qbase when mapping the actual tunnel and the drill cores*

Then, when the last quote is taken into account, the difference between both types of rock quality assessment (Pre-investigation and mapping) is reduced, see Figure 5-8.
The quote (Jw/SRF) and the geometrical correction for Jn don’t have a large effect on the final Q-value when mapping is concerned as we can conclude from Figure 5-9.

The small influence of the last quote when assessing the rock quality of the boreholes in the case of K1, can be seen in Figure 5-10. However, according to Figure 5-10 and 5-11, there have been two ways of assessing the rock mass quality.
5 Results - 5.2 Second step of the analysis

As seen in Figure 5-11, there is a larger influence of the last quote when the assessment of Q has been done for the drill cores of the following contracts: NL33, NL34, NL35 (K3).

This difference between the drill cores of K1 and K3 can be explained by the methodology used to assess the rock quality Q in both cases, that are different from one another. Indeed, K1 was done by VBB VIAK (SWECO) and BBK AB, while the rest was done by SWECO.
5.2.2. Influence of each single parameter

After generating the difference of the parameters and quotes in the Q-system between the actual mapping and the drill core mapping, the following graph have been generated for the Northern Link to study the difference for each single parameter.

![Graph showing the difference in percentage for each Q parameter and quote, contract by contract for NL between the actual mapping and the drill core mapping.](image)

**Figure 5-12** Difference in percentage for each Q parameter and quote, contract by contract for NL between the actual mapping and the drill core mapping.

\[
\text{Diff mean [\%]} = 100 \times \frac{\text{Mapping} - \text{drill core}}{\text{drill core}}
\]

The differences of both quotes, RQD/Jn and Jr/Ja, are negative which means that they have a tendency to be decreased from the borehole mapping to the actual mapping. The Q value is downgraded from the drill cores to the tunnel mapping.

The quote Jr/Ja evolves differently in the case of NL34 in comparison to the other contracts but it can be due to the small amount of core log meters available in that case (approximately 90m), which are from quite bad rock.

The difference for each parameter and quote is studied more closely in the discussion chapter.
6. Discussion

6.1. Influence of single parameters

Here are some comments on each single parameter that can be drawn by analyzing the values of Table 6-2.

- **RQD** gets a higher value under tunnel mapping. A possible reason could be that the drill cores can be in a worse state than when drilled (handling of the drill cores may affect their quality) and might present more or less cracks than the actual rock, depending on the moment the assessment of the Q value is done on the drill cores. It could be also that the drill cores were used to investigate poor quality broken rock associated with weakness zones.

- **Jn** has a higher value in tunnel mapping. This could mean that the tunnels encountered a rock mass with more fracture sets in the tunnel mapping than in the pre-investigation. However, it could also be linked to the fact that it’s difficult to estimate Jn on a drill core, and so there would be a tendency to underestimate this value when doing the pre-investigation. The drilling direction of the borehole also influences the number of joints that are intersected by the borehole. Joints sub-parallel to the borehole will be underrepresented in the cores, and this will give too high RQD-values and too low Jn-values.

The mode of Jn(Drill cores) is 3 or 6 (respectively 1 joint set or two joint sets and random joints) while the mode of Jn(Mapping) is either 9 or 6 (respectively three joint sets or two joint sets and random joints). While Jn = 6 (Two joint sets plus random joints) was expected. The existence of a Jnkorr=2 or 3 * Jn, that is calculated during mapping at tunnel intersections might be another factor that affects the correlation between the engineering geological forecast and the actual mapping.

- **Jr** has a lower value in tunnel mapping. 1.5 in mapping (Slickenslided, undulating or rough, irregular, planar) against 2 or 3 in the drill cores (Smooth, undulating or Rough, irregular, undulating), see Figures 6-1 and 6-2 below.

- The quote RQD/Jn shows a reduction from the drill cores to the tunnel mapping (73.1% for NL35, 26.2% for NL34 and 77.3% for NL33). The relative block size in the rock masses mapped in the tunnel tends to be smaller than the ones expected from the drill cores.
6 Discussion - 6.1 Influence of single parameters

Mapping

Figure 6-1 Extract of the Jr scale 1/2 [13]

Drill cores

Figure 6-2 Extract of the Jr scale 2/2 [13]

The roughness of a discontinuity could be characterized at two scales, a large one (dm to m) and a small one (cm to mm). When looking at a 56mm diameter core log, it’s hard to determine if the joint surface is planar or undulating. The method suggested by NGI to estimate the roughness consists in laying a 1m long ruler on the joint surface. The scale effect is then to be taken into account for the large scale roughness.

The small scale roughness is also questionable when studied on core log for the same reasons. In the case of a core log, the surface on which the roughness is evaluated is quite small and may differ if studied along the whole joint surface.

- Ja is lower for NL34 in tunnel mapping. A possible reason may be that only 97m of drill core were available against 897m of mapped tunnel, so this result is not necessarily representative of the situation of the whole NL34 section. While Ja is quite the same for NL33 and NL35 in tunnel mapping. The geologist did a matching estimation in average of Ja in the Northern Link section NL33 and NL35.

- The quote Jr/Ja is lower in tunnel mapping except for NL34. It should be noted that the value of Jr/Ja is collected for the critical joint set, that is, the joint set most unfavorable for the stability of the tunnel. As the section analyzed is bigger in the tunnel than in the drill core, we can expect to find a worse critical joint set when looking at the tunnel than when looking at 1 m of drill core. Even the way the mapping of the drill core is made has an influence on that. A value for Jr and Ja is evaluated every single meter for the drill cores mapping, while for the actual mapping the only joint considered is the critical one in a wider area.
6 Discussion - 6.1 Influence of single parameters

- \( J_w \) doesn’t change between the steps

- A large influence was noticed on \( J_w/\text{SRF} \) on the \( Q \) value when \( \text{SRF} \) is lower in the mapping than expected. Note that there were two ways of assessing \( \text{SRF} \) in the various areas of the Northern Link. Indeed, in the part K1 it was set to 1 in the mapping of the drill cores done by BBK AB and VBB VIAK (SWECO) while in the part K3 the mapping of the drill cores was done by SWECO.

In Table 6-1, a summary of what have been said earlier is presented.

**Table 6-1 Possible explanation to variation of each parameters’ value from the mapping of the tunnel to the mapping of the drill cores**

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Higher (+) or Lower (-) value under tunnel mapping in comparison to drill core mapping</th>
<th>Possible explanation of this behaviour</th>
</tr>
</thead>
<tbody>
<tr>
<td>RQD</td>
<td>+</td>
<td>Drill cores harmed during transportation. Drill core performed in weak zones.</td>
</tr>
<tr>
<td>( J_n )</td>
<td>+</td>
<td>Difficult to estimate ( J_n ) on a drill core : scale effect. Correction not applied when mapping the drill cores. Drill cores 1 joint set to two joint sets and random joints while mapping two joint set and random joints to three joint sets. Drill cores = 1D directional sample while tunnel mapping is in 3D so it’s easier to identify fracture sets then.</td>
</tr>
<tr>
<td>( J_r )</td>
<td>-</td>
<td>56mm diameter drill cores : scale effect. For mapping : critically oriented joint set. For drill core mapping : every meter.</td>
</tr>
<tr>
<td>( J_a )</td>
<td>=</td>
<td>Good estimation of ( J_a ) in NL. Lower for NL34 and Cityline : possible filling material washed away from the drill cores. For mapping : critically oriented joint set. For drill core mapping : every meter.</td>
</tr>
<tr>
<td>( J_w )</td>
<td>=</td>
<td>( J_w = 1 ) all the time</td>
</tr>
<tr>
<td>SRF</td>
<td>-</td>
<td>Two ways of core logging. SRF generally=1 for tunnel mapping.</td>
</tr>
</tbody>
</table>
A comparison for each NL contract between the mean value for the actual mapping, the mapping of the drill cores and their difference in % are presented in Table 6.2. In the table, the following legends were used:

Green: tendency to strongly raise the Q value from the drill core to the actual mapping in average
Light green: tendency to raise the Q value from the drill core to the actual mapping in average
Red: tendency to strongly lower the Q value from the drill core to the actual mapping in average
Light red: tendency to lower the Q value from the drill core to the actual mapping in average
[] most likely value

Note that Diff mean [%] = 100 * \[\frac{\text{Mapping} - \text{drill core}}{\text{drill core}}\]
## 6 Discussion - 6.1 Influence of single parameters

### Table 6-2 Comparison for each NL contract between the mean value for the actual mapping, the mapping of the drill cores and their difference in %

<table>
<thead>
<tr>
<th>Parameters of the whole project</th>
<th>Parameters NL35</th>
<th>Parameters NL34</th>
<th>Parameters NL33</th>
<th>Parameters old contracts and NL22</th>
</tr>
</thead>
<tbody>
<tr>
<td>Mean borehole</td>
<td>Mean mapping</td>
<td>Mean borehole</td>
<td>Mean mapping</td>
<td>Diff mean [%]</td>
</tr>
<tr>
<td>RQD</td>
<td>86.53 [90]</td>
<td>88.11 [90]</td>
<td>87.27 [95]</td>
<td>7,6</td>
</tr>
<tr>
<td>Jw/SRF</td>
<td>0.77 [1]</td>
<td>142,5</td>
<td>0.93 [1]</td>
<td>0.39 [0.4]</td>
</tr>
<tr>
<td>Length (m)</td>
<td>356.15</td>
<td>1932</td>
<td>2292</td>
<td>110</td>
</tr>
</tbody>
</table>
6 Discussion - 6.2 Comparison between the expactions and the results from the Northern Link and the Cityline.

In Figure 6-3, the assessment of the different parameters is compared for every Northern Link contract.

**Figure 6-3 Comparison of the assessment of the different parameters for NL**

6.2. Comparison between the expactions and the results from the Northern Link and the Cityline.

In this section, the results from Ingrid Kjellström’s master thesis [1] for the Cityline are compared to the ones obtained in this master thesis for the Northern Link and to the expected results.

**Figure 6-4 Comparison of the differences in the parameters leading to Q between the Northern Link and the Cityline**

Mapping - Borehole Northern Link
Mapping - Borehole Cityline (Norrström)
Mapping - Borehole Cityline (Norrmalm)
A first comment when comparing Table 6-2 and Figure 6-3 is that even though some parameters do not evolve the same way when taken separately, the quotes do evolve in the same direction with different intensities for all the tunnels considered.

Table 6-3 Comparison of the signs of the differences between drill core mapping and actual mapping for each parameter and quote of the Q system (Expectations, results for NL and the Cityline)

<table>
<thead>
<tr>
<th>Parameters</th>
<th>Expected</th>
<th>The Cityline</th>
<th>The Northern Link</th>
<th>Comment</th>
</tr>
</thead>
<tbody>
<tr>
<td>RQD</td>
<td>0</td>
<td>-</td>
<td>+</td>
<td>In all the cases, it has a low order of magnitude and seems to have a low effect on the quote where it intervenes</td>
</tr>
<tr>
<td>Jn</td>
<td>+</td>
<td>+</td>
<td>+</td>
<td>Its order of magnitude leads the one of the quote</td>
</tr>
<tr>
<td>Jr</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>Similar results between the Northern Link and the Cityline. The divergence between the drill core mapping and the actual mapping represents 35% approximately, so it’s quite big</td>
</tr>
<tr>
<td>Ja</td>
<td>+</td>
<td>+</td>
<td>-</td>
<td>Has less influence than Jr but tends to either go along or against it.</td>
</tr>
<tr>
<td>Jw</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td></td>
</tr>
<tr>
<td>SRF</td>
<td>-</td>
<td>no info</td>
<td>-</td>
<td></td>
</tr>
<tr>
<td>RQD/Jn</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>The order of magnitude of the quote’s discrepancies between the two kinds of mapping is led by the one of the parameter Jn</td>
</tr>
<tr>
<td>Jr/Ja</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>The order of magnitude of the quote’s discrepancies between the two kinds of mapping is led by the one of the parameter Jr and accentuated or reduced by the one of the parameter Ja</td>
</tr>
<tr>
<td>Jw/SRF</td>
<td>no info</td>
<td>+</td>
<td>-</td>
<td></td>
</tr>
<tr>
<td>Q</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td></td>
</tr>
</tbody>
</table>

In Table 6-3, it is compared how the difference of the mean value between the drill cores and the actual mapping (in %) of each parameter evolves between the expectations for the Stockholm Cityline project and the Northern Link.

“+” being a positive percentage (higher value when the tunnel is actually mapped than when the drill core is mapped) while “−” is a negative one.

The table above shows that the tendencies observed for the Northern Link and the Cityline are quite correlated to the ones expected, apart from Ja’s behavior that is different from the one expected. These discrepancies result in a smaller Q-value in the actual mapping than when mapping the drill cores.
The studies performed above suggest that it is the parameters $J_r$, $J_n$, $J_a$ and $SRF$ that should be studied in more detail, since they seem to have the largest influence on the variations observed between the drill core mapping and the actual mapping in the tunnel.

**Limitations**

The length of the drill cores represents 53,3% and 24,9% of the forecasted length of the tunnel for Norrström and for Norrmalm in the Stockholm Cityline, while they represent only 4,8%, 10,8% and 33,8% for the contracts NL313, NL34 and respectively NL35 (see table 6-4). This could indicate that it is more likely that the drill cores for the Stockholm Cityline better represent the average rock mass quality compared to the Northern Link.

**Table 6-4 Summary of investigated meters of borehole and tunnel meters forecasted and mapped for each contract.**

<table>
<thead>
<tr>
<th>Contract</th>
<th>Mapped (m)</th>
<th>Forecasted (m)</th>
<th>Drill core (m)</th>
<th>Proportion of meters of drill cores per forecasted meter (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Old contracts (K1)</td>
<td></td>
<td></td>
<td>356,15</td>
<td></td>
</tr>
<tr>
<td>NL22</td>
<td>81</td>
<td>-</td>
<td></td>
<td></td>
</tr>
<tr>
<td>NL31</td>
<td>98</td>
<td>-</td>
<td></td>
<td></td>
</tr>
<tr>
<td>NL33</td>
<td>2292</td>
<td>2292</td>
<td>110</td>
<td>4,8</td>
</tr>
<tr>
<td>NL34</td>
<td>897</td>
<td>897</td>
<td>97</td>
<td>10,8</td>
</tr>
<tr>
<td>NL35</td>
<td>1932</td>
<td>1932</td>
<td>653</td>
<td>33,8</td>
</tr>
<tr>
<td>Norrström (Cityline)</td>
<td>4596</td>
<td>3996</td>
<td>2131</td>
<td>53,3</td>
</tr>
<tr>
<td>Norrmalm (Cityline)</td>
<td>2557</td>
<td>2735</td>
<td>682</td>
<td>24,9</td>
</tr>
</tbody>
</table>

The rock mass quality is forecasted based on an interpretation of the information from the drill cores and from other geological observations. Though, it’s good to notice that for the Northern Link, data about how exactly the engineering geological forecast was carried out is missing. The traceability of the engineering geological forecast is really poor in this study and is something Trafikverket and their design consultants should be working on in the future.
6.3. The influence from the human factor

The human factor is most likely an important factor in the case studied. Below are some examples when the human factor could affect the results:

- The lighting conditions may affect the observation of the geologist during the actual mapping.
- The time available to do the work might affect the quality of the work.
- There can be different focus behind the establishment of the engineering geological forecast of the rock mass quality. Indeed, if the geologist chooses to have an overall forecast in agreement with the actual mapping in average rather than a forecast that is in good agreement with the actual mapping for every single meter. Having a more precise forecast allows the geologist to identify better the weakness zones.
- The use of only one classification system might result in that some criteria are not considered. In the literature, it is advised to use two classification systems so that when one doesn’t give enough information to take a decision, the second one can add some additional information.
- Mapping of the drill cores lacking common guidelines.
- Mapping of the rock mass quality lacking common guidelines.
- The scale effect highlighted in the previous section and that might make the link between the drill cores and the actual tunnel mapping.
- The values of the geological features are dependent on the geologist. But in large projects like the Northern Link or the Cityline, there is often more than one geologist (Turnover, mapping team in large projects). There should be guidelines in how to assess the value of a parameter.
- The education of every single geologist affects the results. And that the geologist has experience has also a big influence on the results.

In Figure 6-5 is an illustration of how the subjective judgement from each geologist may affect the final result.

![Figure 6-5: Position of joints recorded by different observers along the same scanline. From a test in the Kielder aqueduct tunnels (slightly modified from Ewan et al. 1983 [27])]
The position of the joints along the scanline are mainly the same but from one observer to another, some joints are considered or not.

Some additional remarks are:

- The classification systems are made for mapping the actual tunnel. Resulting in difficulties in assessing the rock quality of drill cores for example, which is a significant part of the pre-investigation.
- There should be more connections between the planning of the pre-investigation and the aim of the engineering geological forecast. Sometimes, the pre-investigation may be performed without first thinking about what the result is going to be used for.
- There is a dissymmetry of the interesting geological aspect when considering the construction phase from when considering the establishment of the engineering geological forecast.
- How often the mapping is done has an influence on the quality of the results. Indeed, mapping every blast instead of every third blast, makes the mapping less continuous and thus resulting in miss interpretation of the rock mass actual quality.
- The Q system might also not be suited to all kind of ground behavior in tunnels as pointed out by Palmström [28] (see table 6-5).

Table 6-5 The fitness of the Q-system in various types of ground behavior in tunnels (simplified from Palmström and Stille, 2006 [29])

| Types of ground behaviour | Suitability *
<table>
<thead>
<tr>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Stable</td>
<td>2</td>
</tr>
<tr>
<td>Fall of block(s) or fragment(s)</td>
<td>1 - 2</td>
</tr>
<tr>
<td>Cave-in</td>
<td>3</td>
</tr>
<tr>
<td>Running ground</td>
<td>4</td>
</tr>
<tr>
<td>Buckling</td>
<td>3</td>
</tr>
<tr>
<td>Slabbing, spalling</td>
<td>2</td>
</tr>
<tr>
<td>Rock burst</td>
<td>3 - 4</td>
</tr>
<tr>
<td>Squeezing ground</td>
<td>3</td>
</tr>
<tr>
<td>Ravelling from slaking or friability</td>
<td>4</td>
</tr>
<tr>
<td>Swelling ground</td>
<td>3</td>
</tr>
<tr>
<td>Flowing ground</td>
<td>4</td>
</tr>
<tr>
<td>Water ingress</td>
<td>4</td>
</tr>
</tbody>
</table>

*) 1 Suitable; 2 Fair; 3 Poor; 4 Not applicable

6.4. Features not included in the Q-system

In « use and misuse of the Q system » [28], the authors have listed those important features that are not included in the Q system and might explain why the value obtained doesn’t perfectly reflect the quality of the rock. We can note that they correspond to the discontinuity features presented in section 2.5.
• “Joint orientation, which was not found to be an important, general parameter. Part of the reason for this may be that the orientations of many types of excavations can be, and normally are, adjusted to avoid the effect of unfavourably oriented joints. However, this choice is often not available. The parameters Jn, Jr and Ja were considered to play a more important role than orientation. If joint orientations had been included, the classification would have been less general, and its essential simplicity lost.

• Joint size. The fact is that larger joints have a markedly stronger impact on the behavior of a rock mass than smaller. According to Piteau (1973), the size of joints is essential in evaluations, since the strength reduction on a failure surface, which contains a discontinuity, is a function of the joint size. Also ISRM (1978) and Merritt and Baecher (1981) mention this. It is often a problem to observe or measure the joint lengths, a main reason is that the whole joint plane seldom can be seen in rock exposures. This may be a main reason why this feature is not used in the Q-system.

But practice shows that it is easy from observations to divide between cracks or small joints and large joints. Hudson and Priest (1983) recommend that a measure of joint length should only indicate the length/size interval of the joint, and the experience is that a characterization of joint length can be made in intervals of <1m, 1–3m, 3–10m, and >10m.

• Joint aperture. The opening of joints, which is very important for ground water movement and conductivity of the rock mass. This is of main importance when evaluating water inflow to underground excavations.

• Rock strength. The reason for not including rock strength in the Q is that it has little impact on the ground behavior in many cases, especially for jointed rock where instability is caused by block falls. For other types of ground behaviour, e.g. rock stress problems the compressive strength of the intact rock material has a significant influence. But for such cases the rock properties are used to evaluate the actual behavior (degree of bursting, degree of squeezing, etc.) for input into the SRF. But, when the Q system has been extended to cover also other fields than stability and rock support, the rock strength may play an important feature. Therefore, \( Q_c = Q(\sigma_c /100) \) has been introduced. But so far, the two authors have not found any recommendations to when \( Q_c \) should be used instead of the "old Q".

Palmström [28] qualify the SRF as “a sort of 'correction factor' or 'fine tuning factor', rather than a factor expressing 'active stresses' aiming at arriving at a Q value that gives appropriate rock support".
7. Conclusion and recommendation for future works

The conclusions from the performed work are:

- It's good to note that the forecasted and mapped rock mass quality (Q-values) were in good agreement in the Northern Link project. Though, there is a tendency to overestimate the rock quality in the engineering geological forecast in comparison with the mapping (estimating that it is better than what it actually is).

Possible explanation to the discrepancies observed when considering the total accuracy can be that the geologists that perform the pre-investigations and the engineering geological forecast had different goals. In our case, the forecast was in average relatively correct against the mapping but with significant discrepancies when considering every single forecasted section (40% accuracy only in the Northern Link case).

- What is required is not necessarily an increased focus on the discontinuities and the characterization of their properties in the pre-investigation but to better understand that some parameters might not represent the reality of the tunnel when establishing the engineering geological forecast from the pre-investigation. This is possible if the planner understands the physical phenomena involved (scale effect between the steps, drill cores are very local, guideline needed).

- Having an engineering geological forecast and a mapping that matches perfectly is probably difficult to obtain because the geologist would always put himself on the safe side. But having a realistic forecast is something one can aim for. Thanks to the study performed, we have a better understanding of which parameters that are more prone to change from the mapping of the drill cores to the actual mapping in the tunnel. This knowledge can lead to a more accurate engineering geological forecast.

- The parameters that one should focus on are: Jr, Jn, Ja and SRF.

Some recommendations for future work are:

- There is a need for the rock engineering community to provide guidelines in how parameters should be assessed when mapping drill cores.

- To make the engineering geological forecast and the actual mapping correlating better, there should be more emphasize put on communication between:
  - The designer and the mapper (geologist)
  - The client/purchaser and the contractor

To do so, the designer should put more efforts in making the forecast clear to anyone who would wonder why this section of the tunnel was forecasted with that value. The forecasts should present the input data they are based on.

- **The follow up of the engineering geological forecast and the mapping data** should be an important form of the experience feedback. This feedback is necessary to identify the...
measures that can improve the practice of pre-investigation, engineering geological forecast and tunnel mapping for new tunneling projects.
It’s not optimal that in the Northern Link project such a thing was missing because in the end the engineering geological forecast was quite close to the outcome but there were no means to know which parameters were considered for a given section.

- Perform at least one more case study to confirm the observations made about how the parameters of the Q system affect the correlation between the pre-investigation, the engineering geological forecast and the mapping.
9. References


[16] N. Barton, "Application of Q-system and index tests to estimate shear strength and deformability of rock masses." in In Workshop on Norwegian Method of Tunnelling (pp. 66–84)., New Delhi, India., 1993.


Appendices

Appendix A: Roughness according to NGI

From NGI, 2015 [13]
## Appendix B : Detailed results of comparisons

<table>
<thead>
<tr>
<th>Parameters drill cores</th>
<th>NL22</th>
<th>NL33</th>
<th>NL34</th>
<th>NL35</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Mean</td>
<td>Std Dev</td>
<td>COV</td>
<td>Mean</td>
</tr>
<tr>
<td>RQD</td>
<td>86,5</td>
<td>14,8</td>
<td>17,1</td>
<td>81,1</td>
</tr>
<tr>
<td>Jn</td>
<td>5,9</td>
<td>2,2</td>
<td>37,9</td>
<td>4,2</td>
</tr>
<tr>
<td>RQD/Jn</td>
<td>19,1</td>
<td>15,7</td>
<td>81,9</td>
<td>44,8</td>
</tr>
<tr>
<td>Jr</td>
<td>2,9</td>
<td>0,2</td>
<td>7,5</td>
<td>2,5</td>
</tr>
<tr>
<td>Ja</td>
<td>1,4</td>
<td>0,7</td>
<td>51,7</td>
<td>2,4</td>
</tr>
<tr>
<td>Jr/Ja</td>
<td>2,5</td>
<td>0,8</td>
<td>31,0</td>
<td>1,4</td>
</tr>
<tr>
<td>Jw</td>
<td>0,8</td>
<td>0,2</td>
<td>29,9</td>
<td>1,0</td>
</tr>
<tr>
<td>SRF</td>
<td>1,0</td>
<td>0,0</td>
<td>0,0</td>
<td>2,5</td>
</tr>
<tr>
<td>Jw/RF</td>
<td>0,8</td>
<td>0,2</td>
<td>29,9</td>
<td>0,4</td>
</tr>
<tr>
<td>Length (m)</td>
<td>356,15</td>
<td>110</td>
<td>97</td>
<td>653</td>
</tr>
</tbody>
</table>

*Table gathering the mean value, the standard deviation and the COV for each Q-parameter for 4 contracts*
The differences of the mean values (Diff mean) are expressed as percentages (%) while the ones of the standard deviation (Diff Std Dev) are barely differences.

Here is the relation between Diff mean and the mean values for mapping and drill cores that has been used for every single parameter:

\[
\text{Diff mean} = \frac{\text{Mean value for mapping} - \text{Mean value for drill cores}}{\text{Mean value for drill cores}} \times 100
\]

While the relation between Diff Std Dev and the standard deviation for mapping and drill cores is:

\[
\text{Diff Std Dev} = \text{Standard deviation for mapping} - \text{Standard deviation for drill cores}
\]

Thanks to those relations, it’s possible to get back to the values of the parameters for the mapping.

<table>
<thead>
<tr>
<th>Mapping - Drill core</th>
<th>NL22</th>
<th>NL33</th>
<th>NL34</th>
<th>NL35</th>
<th>Rest (NL33, NL34, NL35)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Diff mean</td>
<td>2.7</td>
<td>-8.1</td>
<td>7.6</td>
<td>-11.7</td>
<td>49.9</td>
</tr>
<tr>
<td>Diff Std Dev</td>
<td>2.9</td>
<td>-7.5</td>
<td>39.2</td>
<td>-35.7</td>
<td>30.5</td>
</tr>
<tr>
<td>Diff mean</td>
<td>68.6</td>
<td>2.0</td>
<td>182.6</td>
<td>2.9</td>
<td>61.1</td>
</tr>
<tr>
<td>Diff Std Dev</td>
<td>73.4</td>
<td>-0.1</td>
<td>-6.4</td>
<td>5.2</td>
<td>30.5</td>
</tr>
<tr>
<td>Diff mean</td>
<td>73.4</td>
<td>-0.1</td>
<td>-6.4</td>
<td>5.2</td>
<td>30.5</td>
</tr>
<tr>
<td>Diff Std Dev</td>
<td>47.8</td>
<td>-0.2</td>
<td>-77.3</td>
<td>0.0</td>
<td>-26.2</td>
</tr>
<tr>
<td>Diff mean</td>
<td>24.2</td>
<td>-0.1</td>
<td>132.7</td>
<td>0.2</td>
<td>142.5</td>
</tr>
<tr>
<td>Diff Std Dev</td>
<td>12.0</td>
<td>0.4</td>
<td>-39.3</td>
<td>0.5</td>
<td>7.8</td>
</tr>
<tr>
<td>Diff mean</td>
<td>24.2</td>
<td>-0.1</td>
<td>132.7</td>
<td>0.2</td>
<td>142.5</td>
</tr>
<tr>
<td>Diff Std Dev</td>
<td>12.0</td>
<td>0.4</td>
<td>-39.3</td>
<td>0.5</td>
<td>7.8</td>
</tr>
</tbody>
</table>
### Appendix C: Q charts from [13]

#### Table 1: RQD-values and volumetric jointing.

<table>
<thead>
<tr>
<th>RQD (Rock Quality Designation)</th>
<th>RQD</th>
</tr>
</thead>
<tbody>
<tr>
<td>A: Very poor</td>
<td>0.25</td>
</tr>
<tr>
<td>B: Poor</td>
<td>25-40</td>
</tr>
<tr>
<td>C: Fair</td>
<td>50-75</td>
</tr>
<tr>
<td>D: Good</td>
<td>75-90</td>
</tr>
<tr>
<td>E: Excellent</td>
<td>90-100</td>
</tr>
</tbody>
</table>

Note: 1) Where RQD is reported or measured as ≤ 10 (including 0) the value 10 is used to evaluate the G-value
2) RQD-intervals of 5, i.e. 100, 95, 90, etc., are sufficiently accurate.

#### Table 2: J<sub>n</sub> values.

<table>
<thead>
<tr>
<th>Joint set number</th>
<th>J&lt;sub&gt;n&lt;/sub&gt;</th>
</tr>
</thead>
<tbody>
<tr>
<td>A: Massive, no or few joints</td>
<td>0.5-1.0</td>
</tr>
<tr>
<td>B: One joint set</td>
<td>2</td>
</tr>
<tr>
<td>C: One joint set plus random joints</td>
<td>3</td>
</tr>
<tr>
<td>D: Two joint sets</td>
<td>4</td>
</tr>
<tr>
<td>E: Two joint sets plus random joints</td>
<td>5</td>
</tr>
<tr>
<td>F: Three joint sets</td>
<td>9</td>
</tr>
<tr>
<td>G: Three joint sets plus random joints</td>
<td>12</td>
</tr>
<tr>
<td>H: Four or more joint sets, random heavily jointed “sugar cubes”, etc</td>
<td>15</td>
</tr>
<tr>
<td>J: Crushed rock, earth like</td>
<td>20</td>
</tr>
</tbody>
</table>

Note: 1) For tunnel intersections, use 3 x J<sub>n</sub>
2) For portals, use 2 x J<sub>n</sub>
### Table 3  $J_r$ — values.

<table>
<thead>
<tr>
<th>Joint Roughness Number</th>
<th>$J_r$</th>
</tr>
</thead>
<tbody>
<tr>
<td>a) Rock-wall contact, and Rock-wall contact before 10 cm of shear movement</td>
<td></td>
</tr>
<tr>
<td>A Discontinuous joints</td>
<td>4</td>
</tr>
<tr>
<td>B Rough or irregular, undulating</td>
<td>3</td>
</tr>
<tr>
<td>C Smooth, undulating</td>
<td>2</td>
</tr>
<tr>
<td>D Slickensided, undulating</td>
<td>1.6</td>
</tr>
<tr>
<td>E Rough, irregular, planar</td>
<td>1.5</td>
</tr>
<tr>
<td>F Smooth, planar</td>
<td>1</td>
</tr>
<tr>
<td>G Slickensided, planar</td>
<td>0.5</td>
</tr>
</tbody>
</table>

Note: 1) Description refers to small scale features and intermediate scale features, in that order.

| c) No rock-wall contact when sheared |
| Zone containing clay minerals thick enough to prevent rock-wall contact when sheared | 1     |

Note: 2) Add 1 if the mean spacing of the relevant joint set is greater than 3 m (dependent on the size of the underground opening).

2) $J_r = 0.5$ can be used for planar slickensided joints having lineations, provided the lineations are oriented in the estimated sliding direction.
Table 4: \( J_0 \) values.

<table>
<thead>
<tr>
<th>Joint Alteration Number</th>
<th>( J_0 ) approx.</th>
<th>( J_0 )</th>
</tr>
</thead>
<tbody>
<tr>
<td>a) Rock-wall contact (no mineral fillings, only coatings)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>A</td>
<td>Tightly heated, hard, non-softening, impermeable filling, i.e., quartz or epidote.</td>
<td>0.75</td>
</tr>
<tr>
<td>B</td>
<td>Unaltered joint walls, surface staining only.</td>
<td>25-35'</td>
</tr>
<tr>
<td>C</td>
<td>Slightly altered joint walls. Non-softening mineral coatings; sandy particles, clay-free disintegrated rock, etc.</td>
<td>25-30'</td>
</tr>
<tr>
<td>D</td>
<td>Silty or sandy clay coatings; small clay fraction (non-softening).</td>
<td>20-25'</td>
</tr>
<tr>
<td>E</td>
<td>Softening or low friction clay mineral coatings, i.e., kaolinite or mica. Also chlorite, talc, gypsum, graphite, etc., and small quantities of swelling clays.</td>
<td>8-16'</td>
</tr>
<tr>
<td>b) Rock-wall contact before 10 cm shear (thin mineral fillings)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>F</td>
<td>Sandy particles, clay-free disintegrated rock, etc.</td>
<td>25-30'</td>
</tr>
<tr>
<td>G</td>
<td>Strongly over-consolidated, non-softening, clay mineral fillings (continuous, but &lt;5 mm thickness).</td>
<td>16-24'</td>
</tr>
<tr>
<td>H</td>
<td>Medium or low over-consolidation, softening, clay mineral fillings (continuous, but &lt;5 mm thickness).</td>
<td>13-16'</td>
</tr>
<tr>
<td>J</td>
<td>Swelling clay fillings, i.e., montmorillonite (continuous, but &lt;5 mm thickness). Value of ( J_0 ) depends on percent of swelling clay-size particles.</td>
<td>6-12'</td>
</tr>
<tr>
<td>c) No rock-wall contact when sheared (thick mineral fillings)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>K</td>
<td>Zones or bands of disintegrated or crushed rock. Strongly over-consolidated.</td>
<td>16-24'</td>
</tr>
<tr>
<td>L</td>
<td>Zones or bands of clay, disintegrated or crushed rock. Medium or low over-consolidation or softening fillings.</td>
<td>13-16'</td>
</tr>
<tr>
<td>M</td>
<td>Zones or bands of clay, disintegrated or crushed rock. Swelling clay. ( J_0 ) depends on percent of swelling clay-size particles.</td>
<td>6-12'</td>
</tr>
<tr>
<td>N</td>
<td>Thick continuous zones or bands of clay. Strongly over-consolidated.</td>
<td>12-16'</td>
</tr>
<tr>
<td>O</td>
<td>Thick, continuous zones or bands of clay. Medium to low over-consolidation.</td>
<td>12-16'</td>
</tr>
<tr>
<td>P</td>
<td>Thick, continuous zones or bands with clay. Swelling clay. ( J_0 ) depends on percent of swelling clay-size particles.</td>
<td>6-12'</td>
</tr>
</tbody>
</table>
Table 5  \(J_w\) – values.

<table>
<thead>
<tr>
<th>Joint Water Reduction Factor</th>
<th>(J_w)</th>
</tr>
</thead>
<tbody>
<tr>
<td>A  Dry excavations or minor inflow (humid or a few drips)</td>
<td>1.0</td>
</tr>
<tr>
<td>B  Medium inflow, occasional outwash of joint fillings (many drips/&quot;rain&quot;)</td>
<td>0.66</td>
</tr>
<tr>
<td>C  Jet inflow or high pressure in competent rock with unfilled joints</td>
<td>0.5</td>
</tr>
<tr>
<td>D  Large inflow or high pressure, considerable outwash of joint fillings</td>
<td>0.33</td>
</tr>
<tr>
<td>E  Exceptionally high inflow or water pressure decaying with time.</td>
<td>0.2–0.1</td>
</tr>
<tr>
<td>F  Exceptionally high inflow or water pressure continuing without noticeable decay.</td>
<td>0.1–0.05</td>
</tr>
</tbody>
</table>

Note: 1) Factors C to F are crude estimates. Increase \(J_w\) if the rock is drained or grouting is carried out.

2) Special problems caused by ice formation are not considered.
### Table 6: SRF-values.

<table>
<thead>
<tr>
<th>SRF</th>
<th>6 Stress Reduction Factor</th>
</tr>
</thead>
<tbody>
<tr>
<td>a) Weak zones intersecting the underground opening, which may cause loosening of rock mass</td>
<td></td>
</tr>
<tr>
<td>A</td>
<td>Multiple occurrences of weak zones within a short section containing clay or chemically di-integrated, very loose surrounding rock (any depth), or long sections with incompetent (weak) rock (any depth). For squeezing, see also and 6x</td>
</tr>
<tr>
<td>B</td>
<td>Multiple shear zones within a short section in competent clay-free rock with loose surrounding rock (any depth)</td>
</tr>
<tr>
<td>C</td>
<td>Single weak zones with or without clay or chemical di-integrated rock (depth ≤ 50 m)</td>
</tr>
<tr>
<td>D</td>
<td>Loose, open joints or “sugar cubes”, etc. (any depth)</td>
</tr>
<tr>
<td>E</td>
<td>Single weak zones with or without clay or chemical di-integrated rock (depth &gt; 50 m)</td>
</tr>
</tbody>
</table>

#### Note: 1) Reduce these values of SRF by 25-50% if the weak zones are shallow and do not intersect the underground openings.

<table>
<thead>
<tr>
<th>SRF</th>
<th>b) Competent, mainly massive rock, stress problems</th>
</tr>
</thead>
<tbody>
<tr>
<td>a.1</td>
<td>a.2</td>
</tr>
<tr>
<td>F</td>
<td>Low stress, real surface, open joints</td>
</tr>
<tr>
<td>C</td>
<td>Medium stress, favourable stress condition</td>
</tr>
<tr>
<td>H</td>
<td>High stress, very tight structure, usually favourable to stability. May also be unfavorable to stability dependent on the orientation of stresses compared to jointing/weariness planes</td>
</tr>
<tr>
<td>J</td>
<td>Moderate spalling and/or slaking after &gt;1 hour in massive rock</td>
</tr>
<tr>
<td>K</td>
<td>Spalling or rock burst after a few minutes in massive rock</td>
</tr>
<tr>
<td>L</td>
<td>Heavy rock burst and immediate dynamic deformation in massive rock</td>
</tr>
</tbody>
</table>

#### Note: 1) For strongly anisotropic virgin stress field (if measured), when 5 ≤ σ1/σ2 ≤ 10, reduce σ3 to 0.75 σ3. When 10 ≤ σ1/σ2 > 10, reduce σ2 to 0.5 σ2, where σ1 = unconfined compression strength, and σ2 = maximum tangential stress (estimated from elastic theory). 2) When the depth of the crown below the surface is less than the span, support SRF increase from 2.5 to 5 for such cases (see Table 1). 3) When the depth of the crown below the surface is less than the span, support SRF increase from 2.5 to 5 for such cases (see Table 1).

<table>
<thead>
<tr>
<th>SRF</th>
<th>c) Squeezing rock: plastic deformation in incompetent rock under the influence of high pressure</th>
</tr>
</thead>
<tbody>
<tr>
<td>a.1</td>
<td>1-5</td>
</tr>
<tr>
<td>M</td>
<td>Mid squeezing rock pressure</td>
</tr>
<tr>
<td>N</td>
<td>Heavy squeezing rock pressure</td>
</tr>
</tbody>
</table>

#### Note: 1) Determination of squeezing rock conditions must be made according to the relevant literature (e.g., Singh et al., 1972 and Bhui and Ghiraldini, 1998).

<table>
<thead>
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Appendix D: Rock mass quality and rock support

Support categories

1. Unsupported or spot bolting
2. Spot bolting, SB
3. Systematic bolting, fibre reinforced sprayed concrete, 5-6 cm, Bi-Str
4. Fibre reinforced sprayed concrete and bolting, 6-9 cm, Str (E500)-A
5. Fibre reinforced sprayed concrete and bolting, 9-12 cm, Str (E700)-B
6. Fibre reinforced sprayed concrete and bolting, 12-15 cm, reinforced ribs of sprayed concrete and bolting, Str (E700)-RBS1+B
7. Fibre reinforced sprayed concrete of 15 cm reinforced ribs of sprayed concrete and bolting, Str (E1000)+RBS II+B
8. Cast concrete lining, CCA or Str (E1000)+RBS III+B
9. Special evaluation

Bolts spacing is mainly based on Ø20 mm
E = Energy absorption in fibre reinforced sprayed concrete
ESR = Excavation Support Ratio
Areas with dashed lines have no empirical data

RRS - spacing related to Q-value

| S30/6 | Ø16 - Ø20 (span 10m) |
| D40/6 x 2 | Ø16-20 (span 20m) |
| S65/6 | Ø16-20 (span 5m) |
| D45/6 + 2 | Ø16-20 (span 10m) |
| D55/6 + 4 | Ø20 (span 20m) |
| Special evaluation (span 20 m) |

S30/6 = Single layer of 6 rebars,
30 cm thickness of sprayed concrete
D = Double layer of rebars
Ø16 = Rebar diameter is 16 mm
C/c = RRS spacing, centre + centre

Permanent support recommendations based on Q-values and span/ESR
Appendix E: Mapped sections
Boreholes – rock quality
Appendix F: Example of mapped cracks and reinforcements
FÖRKLARINGAR

1. BERGARTER
   - Granit
   - Gnejs
   - Diabas
   - Pegmatit
   - Grönsten/Amfibolit

2. BERGMASSA
   - Spricka (flack)
   - Kraftig spricka (brant/vertikal)
   - Sprickfylnad:
     - Ka, Kl, Kv, fältspat och lera

Z1: Skivigt berg, skivornas tjocklek >20 cm
Z2: Tunnskivigt berg, skivornas tjocklek <20 cm
Z3: Blockigt berg, blockens kantlängd 20–60 cm
Z4: Delvis sönderkrossat berg, blockens kantlängd <20 cm
Z5: Sönderkrossat berg med lerinslag

3. SVAGHETSZON (bredd >10 cm)

4. VATTENFÖREKOMST
   - Fukt (redovisas inte)
   - Dropp
   - Flöde

5. ÖVRIGT
   - Utfall
   - Öppning (till förbindelsetunnel, nischer etc.)
Appendix F: Example of mapped cracks and reinforcements
Appendix G: Example of the mapping tables produced for the pre investigation and the actual mapping.

**1 Kärnborrhål K3KBH1**

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From [30]
Appendix H: Remarks about the normal distribution graphs from [24]

"The implementation of this work started from the idea that what goes in in a project also comes out of the same project. To be able to track the relationships between in and out, the data have been compared as statistic groups, a way to measure that is built on process theory. The procedure can be summarized as « Measure – Understand – Improve ».

In a number of cases the comparisons are reported as distribution functions which are assumed to be normal distributed. This has been important for the analysis done. The normal distribution is based on the idea that the most common occurring values lie in the middle of the curve, the average value, while the less common ones lie on varying distances from the middle.

The spreading is expressed with the help of the standard deviation, sigma, a measure that expresses how tight or spread a group is, in which case a bigger standard deviation indicates a more spread group than a little one. Altogether, both measurements mean that the probability for different outcomes can be estimated, see Figure 1.

![Figure 4 Probability that different measured values are encountered in a given statistic group of data, expressed as standard deviations.](image)

The accuracy in using the normal distribution to describe the variations of the rock mass quality in the rock mass forecast and tunnel mapping without knowing if they are distributed this way has not been considered to be a problem in this study. This is based on the numerous studies of the general question that indicates that those have very little meaning. Instead it is highlighted that it’s more important to make sure that there are variations rather than any mathematical function which describe it best.

That’s why for each forecast range, the average value of the actual forecast has been calculated with regards to the rock mass quality, together with the standard deviation, sigma. The result from the tunnel mappings has been handled in a similar way. Then, based on the obtained average values and spreads, the corresponding normal distributions have been calculated, plotted and compared. Those have then formed the base observations and conclusions."
Appendix I : More about the method of analysis from [26]

“The variations have been determined by calculating and comparing centers of gravity and
distributions for different Q-evaluations for the different tunnel parts. Those were expressed as
average value and standard deviation (σ). As the Q-values are presented in a logarithmic scale, see
figure..., the values that have been used in the calculations have been converted to be used as
calculation basis. For example:

Suppose that when mapping, a tunnel has been determined to have a Q-value of 1.1 on a 4-
meter stretch, a Q-value of 5.6 on the next following 2 meters and a Q-value of 11.2 on the last 4
meters. The most representative Q-value on the stretch, the average value, have been determined
in such a way:

1. Logarithms of the input Q-values. Which are 0.0414, 0.7482 and respectively 1.0492.
2. Determining the different Q-values’ importance (« weighting ») by calculating the relative
   lengths they represent. Those are 0.4, 0.2 and respectively 0.4.
3. Multiplication of the logarithmic Q-values and their weight and also summing them gives
   the Q-average value for the tunnel part, i.e. 0.4 x (0.0414), 0.2 x (0.7482) and respectively
   0.4 x (1.0492). Which gives 0.5859.

Note that the obtained value 0.5859 can be recalculated to a common Q-value by calculating
$10^{0.5859}$. This corresponds to an average Q-value of 3.9. Calculating an average value without
regards to the fact that Q-values are expressed logarithmically and that different values represent
different lengths gives an average Q value of 6, i.e. resulting in a little overestimation of the actual
rock properties.

The measurement of the spreading, the standard deviation σ, for the input Q-values have been
determined in a similar way, i.e. by taking the logarithm and weighting before calculating. In this
case $σ = 0.1677$.

Even those values can be recalculated to a common Q-value by calculating $10^{0.1677}$. Which becomes
1.5.

Altogether, it can be said that the Q-values vary around the average value 3.9 with a standard
deviation of 1.5.

The way of looking at the average values and the standard deviation has meant that the assessment
in the forecast and from the mappings has been treated as if they were normally distributed
probability function. This has meant that the reliability in different variations could be assessed,
something that has contributed to the quality of the conclusions. The question on which distribution
function is the best to describe the reality has been excluded on the basis that average + 3 x standard
deviations always contains more than 99% of the possible outcomes, regardless of the distribution
function.

The named approach and calculations has even made it possible to express the forecasts as
indicating only a rock class for a particular tunnel stretch in a way that makes it possible to compare
them with both another forecast as well as the mapping results. For example:
Suppose that a rock forecast indicates that 100% of the rock is placed between Q-values 1 and 4. The most representative Q-value on the stretch, average value, have then been determined in the following way:

1. Logarithms of the available Q-value borders, in this case 1 and respectively 4. Which gives 0 and respectively 0.6021.
2. Averaging in the same way as above of the logarithmic Q-values. Which gives 0.3010.

As earlier, this can be expressed as a « common » Q-value by calculating $10^{0.3010}$. Which gives 2.

The standard deviation $\sigma$ has been determined basing the calculation on the fact that the average value $+ 3\sigma$ describes almost 100% of the possible outcomes. This implies that the values which have been set as outer limits, i.e. 0 and respectively 0.6021, have been considered describe $6\sigma$. The standard deviation $\sigma$ has then been calculated by dividing the difference (0.6021 – 0) by 6, i.e. $\sigma = \frac{0.6021 - 0}{6} = 0.1004$. This corresponds to a Q-value of 1.3.

This means that a forecast that indicates that 100% of the rock is placed between the Q-values 1 and 4 has been expressed as one that has an average Q-value of 2 and a spreading measurement, expressed as $\sigma$, that is 1.3. If the forecast indicates a percentage distribution in different rock classes, the average value has been calculated by including those weights in the calculations.

Altogether, this has meant that all the input data concerning rock classes could have been expressed in a way that makes them comparable to each other, regardless how they were expressed in the beginning. This leads to the fact that the uncertainties in rock characterization and classification can be studied as time-dependent variables which can be measured and used as base for calculations and comparisons. Since the reduction of uncertainties, variations, lead to efficiency, those metrics can also be used as support for improvement.”
Appendix J: Values of the average and of the standard deviation of the Log10 of the Qmean for both the mapping and the forecast

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