Preface

This technical report was carried out as a master’s thesis at the Royal institute of technology (KTH) during 2009. The report is meant to be an initial study to a larger research project at the department of building technology at the KTH. The report includes a summary of earlier research, a discussion about practical problems concerning shotcrete and a numerical study of an irregular shotcrete shell. It ends with a discussion about the findings and a few suggestions for further research on the topic that might be helpful.

The author would like to thank his supervisor Anders Ansell, who has been very supportive and have given me access to a lot of the necessary literature used in this technical report.

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Stockholm, November 2009

Anders Beijer-Lundberg
Summary

The shape of a tunnel wall after the task of blasting and drilling is typically irregular. After the tunnel wall is sprayed with shotcrete, the resulting shotcrete surface will also be irregular and have a big variation in thickness due to uneven shotcrete coverage. Today, most design models assume that the shotcrete surface behaves as a concrete slab. Research has proved that this is not the case and this report investigates the behaviour of an irregular shotcrete lining and is a pre-study to further research.

The existing literature on the subject is reviewed to gain an understanding of the important parameters that affect the shotcrete shell. In addition to the theoretical research, discussions have been held with different industry actors to gain an understanding of the practical problems and how shotcrete is presently done. The relevant parameters that influence the behaviour of a tunnel lining are discussed and the quantitative and qualitative importance of each parameter is estimated based on previous research.

Some of the practical problems that arise when a shotcrete lining is constructed are discussed. These problems complicate the design in many ways, and makes shotcrete design very different from ordinary structural design. There is also a brief review of how shotcrete design is carried out in Sweden.

The behaviour of a shotcrete rock support system subjected to several load combinations using different models that simulate irregularity and variation in thickness is analyzed. This is a further development of an existing concept and confirms some of these conclusions and poses additional questions.

Finally, the results from the earlier chapter are compared and discussed. The results show that the influence of the irregularity is highly dependent on the shotcrete thickness and shape and that the placement of the rock bolts is very important. Several other aspects are discussed as well. Finally suggestions for further research are given, especially the need for a 3D-model where the interface strength is included.
Sammanfattning

I de flesta stora infrastrukturprojekt där tunnlar igår är sprutbetong en del av bergförstärkningen. Sprutbetong är en blandning av cement, ballast, vatten och olika tillsatser såsom accelerator, som sprutas på till exempel en bergyta. Det går även att använda stålflibrer i denna blandning, vilket ger en betydligt större draghållfasthet. I detta fall brukar bergförstärkningen även bestå av bergbultar, vilket är bultar olika längd bland annat kan gjutas in i berget. Dessa kan sedan förses med brickor utanpå betongytan för att inte alltför stora spänningar ska ske vid bultarna.


Denna rapport är en förstudie till ett större forskningsprojekt om sprutbetong. Rapporten granskar den befintliga litteraturen på områden, försöker beskriva de praktiska problem som uppstår under byggnation, och använder en utveckling av den numeriska modell som tidigare har använts för att analysera samma problem i 3D.

Resultaten visar att de tidigare slutsatserna stämmer förhållandevis väl i 3D och att flera av de slutsatser som dragits av tidigare studie även stämmer med denna modell. Rapporten avslutas med förslag på fortsatt forskning och betonar vikten av att använd bergmassan som en del av bergförstärkningen i kommande modeller, där särskilt vidhäftningen som sker mellan sprutbetongen och berget är en viktig parameter eftersom antalet brott i sprutbetongen verkar samverka med denna parameter.
Notations

These notations are used throughout the report:

- $c$: cohesion strength
- $\delta_0$: deflection of the flat slab
- $\delta_i$: deflection of the irregular slab
- $E$: Young’s modulus of elasticity
- $E_g$: Young’s modulus of the surrounding ground
- $E_l$: Young’s modulus of the lining
- $EDZ$: excavation damaged zone
- $f_c$: compressive strength
- $f_t$: tensile strength
- $\phi$: internal friction angle
- $g$: gravity constant
- $\gamma$: unit weight of the ground
- $h$: shotcrete thickness
- $I_l$: moment of inertia of the lining
- $K$: horizontal pressure coefficient
- $k$: empirical factor depending on the steel fiber ratio
- $M$: lining moment
- $M_N$: normalized moment ratio
- $\mu_s$: steel fibre ratio
- $O$: circumference of the rock
- $q_i$: maximum load of the irregular slab
- $q_0$: maximum load of the flab slab
\( R \) radius of the lining

\( RMR \) rock mass rating

\( \rho_{\text{rock}} \) density of rock

\( \sigma_{\text{ad}} \) adhesion strength at the shotcrete-rock interface

\( \sigma_c \) compressive strength

\( \sigma_t \) tensile strength

\( \sigma' \) effective stress

\( \sigma_{su} \) yield stress of steel fibres

\( \sigma_H \) horizontal pressure

\( \sigma_V \) vertical pressure

\( \tau \) shear strength in interface

\( T \) lining thrust

\( T_N \) normalized thrust ratio

\( \nu_g \) Poisson’s ratio of the surrounding ground

\( \nu_l \) Poisson’s ratio of the lining

\( Z \) tunnel depth to the springline
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1. Introduction

Enhanced knowledge is needed in underground construction, since several of the present and future road and railway infrastructure project probably will be located underground, both in Stockholm and in other major cities. An important part of underground construction is the rock support. This is usually carried out using a combination of rock bolts and shotcrete, which is concrete this is sprayed onto surfaces instead of casted in forms. Shotcrete has usually been designed as a regular concrete slab which is different from how it works in-situ.

1.1 Background

Infrastructure projects have become more and more important in the modern society. The roads and railroads that connect the urban areas form the network of the modern economy. In Sweden, as in other parts of the developed world, the big and far-reaching infrastructure projects have been increasingly difficult to initiate and complete, depending on several factors that impact the development of urban infrastructure.

The most important factors in Sweden when big infrastructure project are initiated are laws and public opinion striving to prevent infrastructure development. The laws mostly concern the environment in certain areas, and can constrain the infrastructure development in a number of ways. An example is when certain rare species are found at a future construction site, the construction plans will have to be abandoned, since the government agencies for environment consider the preservation of the endangered species more important than the infrastructure, according to Jonas Sahlström (2009). On the other hand, the general public has always been ambivalent about urban infrastructure. At the same time as road and railway connections have been appreciated, there has always been a resistance to construction close to residential areas. This is sometimes called NIMBY, “not in my backyard”, and related to everything from residential development to highways and railways, and means that the good of the general public is often sacrificed in order to placate a minority, according to Frank Ouchterlony (2009).

These factors have meant that the infrastructure development has been very slow and complicated in Sweden and especially in Stockholm during the later part of the 20th century. After the completion of the Essingeleden in the late 1960s, no other large-scale road-based infrastructure project was completed until the Södra länken in the 1990s. This means that Stockholm had the same infrastructure for more than 30 years, apart from the underground system Tunnelbanan. At the present time the road system will be extended with the Norra länken, and the railroad infrastructure expanded by the Citybanan, which will be completed during the coming years. The common denominator about these three big projects, Södra länken, Norra länken, and Citybanan, is that they are underground networks of tunnels, situated in and
next to the city centre of Stockholm. The above mentioned factors, i.e. environmental laws and public disapproval of infrastructure projects, have made surface construction so difficult that most large-scale infrastructure projects probably will consist of tunnels in the future, both in Sweden and in other countries. A project that is discussed in Sweden today is the Österleden, located under the sea. The project will probably be very expensive, but would be impossible to complete above ground because of the political issues that impact urban development, according to Jonas Sahlström (2009).

This implies that underground construction technology will be very important in the future, both in Sweden as well as in other countries with similar kind of problems. The methods currently used will probably be improved to ensure a safer and faster way to create tunnels. The change from surface infrastructure to underground infrastructure will also be expensive, and it is important to develop cost-efficient methods to create underground openings. Because of these reasons, investigations of the properties of the parts involved in underground construction are very important, from the initial investigations to the construction of the shotcrete lining. The cost of the construction can be reduced when the knowledge is expanded, and society can gain significantly by these improvements.

1.2 Shotcrete as rock support

Shotcrete is concrete that is pneumatically sprayed on surfaces, instead of being casted in forms like ordinary concrete. It was developed in the beginning of the 20th century, originally to create animal models in an American museum, but the technique spread to the construction industry where it was recognized as a valuable tool, according to Holmgren (1987). The development of shotcrete happened at the same time as rapid development in tunnelling took place. Originally the rock support in tunnels had been wooden frames. These were later replaced by metal frames, but in the 1940s a new rock support technique called the New Austrian Tunnelling Method became known all over the world, which is described by Nord and Stille (1990). This method used the rock as a part of the rock support, and allowed certain deformations since this led to decreased rock stresses in the lining. These required detailed measurements of the lining strain, and a flexible lining that could allow a certain deformation. The preferred rock support for this was shotcrete combined with rock bolts. This method has become used all over the world.

Another technical development that increased the use of shotcrete was the introduction of steel fiber reinforcement in the shotcrete composite. This increases the tensile strength since the steel fibres bridges the cracks in the shotcrete mass and allow larger deformations than the brittle unreinforced shotcrete composite, according to Nord and Stille (1990). Even though the combination of shotcrete and rock bolts is widely used, several aspects about how the shotcrete works have not been described in detail. Traditionally, the design of a shotcrete shell combined with rock bolts has been estimated assuming a concrete slab with the same properties as the
shotcrete. Originally, only a rough estimation combined by experience was used, according to Nord and Stille (1990). In reality, the shotcrete shell is not very similar to a concrete slab, since the geometry is very much different from the slab.

In Sweden, as well as in other places in the world, the most commonly used way to penetrate the rock is by drilling and blasting, as described by Borio and Peila (2009). This leads to a surface that is highly irregular, as is shown in Fig. 1.1. The irregularity of the surface depends on several parameters, but the rock quality is very important, according to Nord and Stille (1990).

![Fig. 1.1: Irregular rock surface in the tunnel ceiling, photograph by Johan Spross.](image)

The knowledge about the structural properties of this irregular shotcrete shell is quite limited. Nilsson (2003) uses numerical models made in ABAQUS, a general finite element program, Abaqus (2009), to estimate the load bearing capacity of an irregular shotcrete shell. It is found that for a slender and thin shotcrete shell, an irregular form has a significantly higher load bearing capacity than a regular one. Borio and Peila (2009) use a 2D-model to estimate the stresses in an irregular shotcrete shell, which also has a variation in the thickness, and finds high stress concentrations at the rock-shotcrete interface of the shotcrete lining. Malmgren and Nordlund (2007) also use a 2D-model to estimate the influence of the irregularity and find that
surface irregularity leads to a higher number of rock-shotcrete interface failures, and to a reduced number of lining failures.

Since shotcrete and rock bolts are commonly used in large infrastructure projects, it is important to properly describe the structural properties of the rock support. This report tries to answer some of the additional questions about the influence of the irregular shotcrete lining and how a more accurate design model can be implemented.

1.3 Aim and goals

The aims and goals of the report are to prepare information for a larger research study. A literature review discusses the relevance of using different parameters in numerical models of the rock support. The practical aspects of the construction of a lining are discussed, and a numerical model is used to verify the results from Nilsson (2003) in 3D, and well as to compare the results to other, previous research. The results from the numerical models are discussed and possible conclusions and further suggestions for future research are stated.

1.4 Contents of report

Chapter 2 contain a literature review that describes the numerical models that have been used in previous research, the parameters that have been used, the numerical results and the conclusions drawn from the results. Finally a summary of how the different parameters affect the shotcrete is made and the effect on the lining is discussed.

In Chapter 3 the practical problems that arise in the design and construction of a shotcrete lining are investigated. The reasons for overbreak, the practical organization of the construction of lining, the calculations sometimes used, and a summary of the problems are detailed and discussed to get a picture of the practicalities of the construction and design.

Chapter 4 describes the use of a numerical model to investigate the behaviour of a shotcrete lining will rock bolts in 3D. The model is a development of the model used in Nilsson (2003). The model is used with several different load cases and the influence of the irregularity, the variation in thickness and the placement of the rock bolts on a shotcrete shell is investigated.

In the last chapter, the findings of the report are discussed, especially the problem of getting the theoretical findings to work in construction. There are many practical obstacles that must be incorporated in the design process. Brief advice for practical construction is given and some suggestions for future research are discussed.
2. Literature review of previous investigations

The effect of the irregularity on the behaviour of the shotcrete shell has been studied in a limited number of scientific papers. These scientific articles have also studied the influence of other parameters that can affect the shotcrete shell, often in combined with the irregularity. In this chapter, some of these papers will be reviewed. Both the numerical models that have been used as well as the results of the simulations will be related, which is needed to grasp the difference between these models and the model described in chapter 4 of this report. Finally a summary will be made that describes the different parameters as well as a quantitative and qualitative estimation of the influence of these factors. This chapter is also meant to form a short scientific background for further research on this topic.

2.1 Fiber reinforced sprayed concrete anchored in rock

The model developed by Nilsson (2003), is the basis of the numerical modelling presented in this report, which is based on the same geometrical and material properties. The numerical simulations of interest in this report consist of a parameter study of an irregular shotcrete slab. The parameters investigated were the irregularity of the slab, the thickness of the slab, the position of the rock bolts and the boundary conditions used in the simulations. The stiffness of the slab was indirectly changed as the thickness and shape was changed.

Two different irregular slabs were used. The first was quadratic and 4 x 4 m$^2$, shown in Fig. 2.1, with a rock bolt placed in the middle of the slab. The other was 6 x 6 m$^2$, shown in Fig. 2.2, with rock bolts placed either at the peaks of the slab or at the depressions of the slab. Similar simulations were also performed on a flat slab to compare the effects of the irregularity. Two different thicknesses on the shotcrete slab were used, 40 mm and 80 mm, both for the irregular slabs and the flat slabs. The load from the rock mass was simulated by a uniformly distributed load on the surface of the slab. Two different boundary conditions were used, simply supported ends and fixed ends. Nilsson (2008) believes that fixed ends are the most realistic assumption.

Parameters investigated

- Irregularity
- Thickness
- Rock bolt placement
- Boundary conditions
- Stiffness (indirectly)
Fig. 2.1: The 4 x 4 m$^2$ shotcrete slab. From Nilsson (2003).

Fig. 2.2: The 6 x 6 m$^2$ shotcrete slab. From Nilsson (2003).
**Stiffness analysis**

The influence of the irregularity on the stiffness of the shotcrete slab was the first parameter investigated. This was done by comparing the deflection of the irregular slab compared to that of a flat slab, with the same properties as the irregular slab. The relative stiffness is defined as the deflection of the flat slab $\delta_0$, divided by the deflection of the irregular slab, $\delta_i$. The result is shown in Fig 2.3, and indicates that the stiffness of the irregular slab is larger than for the regular slab. The difference in stiffness is also much more influenced by the irregularity when the thickness of the shotcrete lining is thin. The reason behind this is that the moment of inertia is affected more by the irregularity when the lining is thin, according to Nilsson (2003).

![Graph showing relative deflection of the irregular shotcrete slab compared to the regular slab.](image)

*Fig. 2.3: Relative deflection of the irregular shotcrete slab compared to the regular slab. From Nilsson (2003).*

**Load bearing capacity**

Nilsson (2003) performs an investigation of the influence from the following parameters on the load bearing capacity: the position of the rock bolts, the boundary conditions of the slab and the slab thickness. The influence was determined by using an automated load increment procedure provided by ABAQUS, Abaqus (2009). The rock bolts are placed first on the peaks of the shotcrete surface and then on the depressions of the shotcrete surface, thereby using two different rock bolting patterns. The boundary conditions are either fixed ends or simply supported, and fixed ends imply that neither movement nor rotation is possible, and simply supported imply that rotation is allowed but no movement. The load bearing capacity of the irregular slabs is
expressed as relative load bearing capacity. This is defined as the ratio between the maximum load of the irregular slab \( q_i \), and the maximum load of the flab slab, \( q_0 \), thereby comparing the load bearing capacity of slabs with different shape but with the same shotcrete thickness. Some of the results are given in Figs. 2.4-2.7.

As can be seen from Figs. 2.4-2.7, the position of the rock bolts is a very important parameter. The reason for this is that this is that if the rock bolts are placed on the peaks, they create a stress distribution around the rock bolts where the load is carried by compressed domes, according to Nilsson (2003). If the rock bolts are placed at the depressions, they will create a stress distribution in the vicinity of the rock bolts that is dominated by tensile stresses. Since the concrete is more sensitive to tensile stresses, this is clearly unfavourable. Nilsson (2003) also notes that this increases the risk for tensile failure in the rock bolts of punching failure at the washers, but that these problems can easily be solved by using rock bolts with higher strength or increasing the size of the washers.

Another conclusion is that the boundary conditions are also very important. As Fig. 2.4 and Fig. 2.6 shows, the boundary conditions change the load bearing capacity completely, which becomes much larger when the slab is simply supported than with fixed ends. Since what is measured is the relative load bearing capacity, the influence of the boundary conditions can be explained by the fact that the flat slab will have a higher degree of compressive arch action than the irregular one. This is because the effect of the compressive arch effect, i.e. the formation of pressure arches inside the concrete structure which help the structure to resist high forces, decreases when the irregularity increases, due to the continuous internal space needed to form compressive arches is smaller.

The influence of the irregularity depends on other parameters as well. As Fig. 2.4 shows, the load bearing capacity can increase by 100 % when the irregularity is 400 mm, the rock bolts are placed at the peaks and the thickness is 40 mm. However, when the rock bolts are placed at the depressions when the thickness is 80 mm and the irregularity is 80 mm, the effect will be negative (Fig. 2.7) and the irregular slab will have a lower load bearing capacity compared to the regular slab. This shows that the effect from an irregularity in the shotcrete shell can be completely different depending on the position of the rock bolts and the lining thickness for a given irregularity. The effect of the irregularity is also much smaller when the thickness of the shotcrete lining is 80 mm instead of 40 mm, as can be seen in Fig. 2.4 and Fig. 2.6.

The stiffness of the lining, or the slab, is indirectly changed between the models since the irregularity and the thickness of the shotcrete changes, which can be noted. In the Son-Cording (2006) scientific article that will be discussed later, the stiffness of the lining is actively changed both by changing the properties of the lining but also the properties of the surrounding ground. The shotcrete slab described by Nilsson (2003) does not interact with the surrounding rock mass, but if it does, the stiffness is probably one of the more interesting parameters that influence the behaviour of the lining.
Fig. 2.4: Load bearing capacity for slabs with shotcrete thickness 40 mm, simply supported boundary conditions used. From Nilsson (2003).

Fig. 2.5: Load bearing capacity for slabs with shotcrete thickness 80 mm, simply supported boundary conditions used. From Nilsson (2003).
Fig. 2.6: Load bearing capacity for slabs with shotcrete thickness 40 mm, fixed ends boundary conditions used. From Nilsson (2003).

Fig. 2.7: Load bearing capacity for slabs with shotcrete thickness 80 mm, fixed ends boundary conditions used. From Nilsson (2003).
The visualizations of the stress distribution in the slabs reveal the influence of the placement of the rock bolts. In Fig. 2.8 the bolts have been placed at the peaks of the shotcrete surface. As has already been stated, this creates local areas of high compressive stresses around the bolts, which can be seen in the visualizations. Fig 2.9 shows the opposite when the rock bolts have been placed at the depressions instead. This creates a local stress distribution dominated by high tensile stresses around the rock bolts, which leads to more tensile failures in the lining, according to Nilsson (2003).

Fig. 2.8: Rock bolts placed at the peaks of the shotcrete surface which results in high compressive stresses in the vicinity of the rock bolts. From Nilsson (2003).
2.2 Interaction between shotcrete, rock and rock bolts

The model by Nordlund and Malmgren (2007) includes more parameters than the model by Nilsson (2003) but uses a 2D model in plane strain representation instead. One of the most interesting features is that it incorporates the rock into the model where it acts both as a load and as rock support. This is a far more realistic setting than the one used in Nilsson (2003), since this is how the tunnel lining usually work, according to Nord and Stille (1990). The tunnel design philosophy called the New Austrian tunnel method (NATM) also stresses the importance of the interaction between the rock support and the rock, according to Hoek and Brown (1980). It is therefore very interesting to see how the two mediums work together. This is in some ways done in more detail in Son and Cording (2006), where the lining is stiffer, which means that the interaction becomes more interesting. In this model the shotcrete does not cover the whole tunnel section, which decreases the stiffness of the lining. Since most of the Swedish rock mass is hard compared to soft ground, this should not be that important but it can be significant in some cases, something that is describes in some detail in Malmgren and Nordlund (2007).

The incorporation of the rock mass is done by adding the rock-shotcrete interface to the model. The interface strength was also used as a parameter, where the tensile strength had the values 0.3, 0.6 and 1.2 MPa while the bond strength ($\sigma_{ad}$) had the same values. The shear strength in the
The rock mass itself is also a parameter. Its properties is described by different values of the
RMR, $\sigma_c$, $\sigma_t$, $\tan(\phi)$ and $c$ of the rock mass, respectively, as well as using different values for the
Young’s modulus. The EDZ (excavation damaged zone), i.e. the part of the rock mass closest to
the blast that has other properties than the rest of the rock mass, was simulated by using a
reduced value of the Young’s modulus for the rock next to the blast zone. The rock properties are
usually different from the rest of the rock mass at the EDZ because of the damage caused by the
explosion, which of course depends on a number of parameters, including rock strength and
workmanship, according to Malmgren and Nordlund (2007). Two different placements of the
rock bolts are used, at the peaks of the lining or at the depressions of the lining. This is done in a
similar way as in the model used in Nilsson (2003).

The irregularity was simulated by using four different geometric models, where three of them
had geometric irregularities. The amplitude was changed from 0 mm in the first model, to 80
mm, 150 mm and 300 mm in the irregular models. This corresponds to an irregularity, which is
defined as the double amplitude by Nilsson (2003), of 160, 300 and 600 mm. The irregularities
are sharper than in the Nilsson model, where the irregularities were simulated by smooth curves.
This leads to rather rough edges, something that is simulated in the models by Borio and Peila
(2009), and Son and Cording (2006) too. This is perhaps a more realistic assumption about the
geometry of the rock surface than the one made in the Nilsson (2003) model, since the real rock
edges are quite sharp, which can be seen in Fig. 1.1.

The analysis was performed in the finite element program UDEC (Universal distinct element
code), a plane strain representation numerical analysis tool, UDEC (2009), using the model
shown in Fig. 2.10, where the irregularity was changed when it was used as a parameter. Since
the research was carried out in the Swedish Kiirunavaara mine, two different load patterns were
used to simulate changes in stress distribution when mines are opened. In the first the horizontal
stress changed from 55 MPa to 65 MPa while the vertical stress was 27 MPa all the time. In the
second one the horizontal stress changed from 55 MPa to 45 MPa while the vertical stress was
27 MPa all the time. The load conditions are quite different from the one normally encountered
in normal infrastructure projects, but should be a good approximation of the conditions in the
Kiirunavaara mine, according to Malmgren and Nordlund (2007).
The simulations were carried out as a sensitivity study where each parameter was changed at a time. The article defines a base case, which had “normal properties” compared to the in-situ case, i.e. irregularity 300 mm and rock strength was approximately the same as the averages tests that had been performed in the mine, according to Malmgren and Nordlund (2007).

**Parameters investigated**

- Irregularity
- EDZ (excavation damaged zone)
- Load effect, two different sequences
- Rock strength
- Shotcrete thickness
- Rock bolt placement
- Discontinuities (one and many)
- Rock-shotcrete interface

**Results**

The use of the rock mass as a component of the rock support changes the results quite significantly in some aspects. For example, there is no big difference in displacement between
unsupported rock and supported rock until the irregularity becomes about 600 mm, due to the fact that the rock also contributes to the load carrying system and seems to be able to withstand the pressures if the irregularity is limited. The results show that the irregularities, the rock strength and the Young’s modulus of the rock have quite a significant influence on the displacements. The strength of the interface, the shotcrete thickness or the use of rock bolts did however, not have a significant influence, according to Malmgren and Nordlund (2007).

The influence of the irregularity on the stress distributions is apparent when the stress in the lining is plotted along the tunnel section. This can be seen in Fig. 2.11 and Fig. 2.12. Compressive stresses are depicted outside the tunnel lining and tensile stresses inside the lining. When irregularities on the tunnel surface increases, the large compressive stresses that are clearly visible in Fig. 2.11 will decrease. It is also worth remarking that the irregularity in Fig. 2.12 seems to increase the tensile stresses at the tunnel walls and ceiling, compared to Fig. 2.11. Since the tensile stresses are more damaging for the concrete, this might have a considerable effect on the load capacity of the tunnel lining. This is also stated in Hoek and Brown (1980), who find empirical evidence of the higher tensile stresses at the irregular edges.

**Fig. 2.11: Stresses in lining, regular surface From Malmgren and Nordlund (2007).**

**Fig. 2.12: Stresses in lining, irregular surface. From Malmgren and Nordlund (2007).**
The number of failures in the lining and in the shotcrete-rock interface is also investigated. The number of tensile interface failures with the number of tensile lining failures is compared for the different models. Some of the results can be seen in Fig. 2.13 and Fig. 2.14. It seems like there are significantly more failures in the interface for the regular model shown in Fig. 2.13, compared to the number of failures in the interface, shown in Fig. 2.14. There are just two lining failures for the regular model. The numbers of failures of the irregular model are different. There are fewer interface failures for the irregular compared to the regular model, which can be seen in Fig. 2.13, but much more failures in the lining, as can be seen in Fig. 2.14.

Fig. 2.13: Tensile failures at the interface. From Malmgren and Nordlund (2007).

Fig. 2.14: Tensile failures in the lining. From Malmgren and Nordlund (2007).

The results indicate that there is a relation between the irregularity and the number of interface failures and lining failures. The results also indicate that increased interface strength can result in more lining failures. There is also a possible connection between the stress distribution and the
failures in the interface and the lining, but this is not investigated here since only one parameter was changed at a time. The same relation can be shown in Fig. 2.15 and Fig. 2.16, that shows the number of failures in the interface and in the lining when the irregularity is varied. When the irregularity increases, the number of interface failures decrease but the number of lining failures increases, even when the irregularity of the lining is small compared to the lining thickness.

Fig. 2.15: Distribution of tensile interface failures. From Malmgren and Nordlund (2007).

Fig. 2.16: Distribution of tensile lining failures. From Malmgren and Nordlund (2007).
The report also investigated the effect of the rock bolt placement. Two different rock bolt patterns were used, one were the bolts were placed at the depressions of the surface (pattern 1), and the other where the rock bolts were placed at the peaks of the surface (pattern 2). The results can be seen in Fig. 2.17, where it is obvious that it is preferable to place the rock bolts at the peaks of the shotcrete surface, and that it can even in some cases be better not to use rock bolts than to place these at the depressions of the surface, even though this effect seems to be quite insignificant. The results are similar to that of Nilsson (2003) who also showed the importance of the placement of the rock bolts on the shotcrete surface.

Fig. 2.17: Failures depending on the placement pattern of the rock bolts. From Malmgren and Nordlund (2007).

Fig. 2.18: Tensile failures in the interface and in the lining with varying discontinuities. From Malmgren and Nordlund (2007).
The discontinuities also affect the lining and the interface. Fig. 2.18 shows the number of tensile failures in the interface and the number of tensile failures in the lining when the discontinuities are changed. The stress distribution will also change when the number of discontinuities increases, but there is no big difference when there is only one discontinuity, as Fig. 2.19 shows. The importance of the discontinuities depends on the rock quality. If there are few discontinuities, these will not matter much, since the interaction between the shotcrete lining and the rock mass will be different. It also affects the flexibility ratio, something that is discussed by Son and Cording (2006). It is also probable that the loads will probably change when there are many discontinuities, since there will be more joints and fracture zones that can disconnect from the rock mass and affect the lining.

![Fig. 2.19: Stress distribution in the lining. From Malmgren and Nordlund (2007).](image)

### 2.3 Influence of tunnel shape on lining stresses

Borio and Peila (2009) investigate the behaviour of a shotcrete lining subjected to uniform pressure (Fig 2.20). The model is created in the FE-program 2D FEM Phase, a 2D elasto-plastic finite element program, 2D FEM Phase\textsuperscript{2D} (2009), and the model is quite similar to the one used by Malmgren and Nordlund (2007). One of the similarities between the models is that that the lining is only sprayed on the ceiling and walls, not on the ground, which decreases the lining stiffness and thereby increases the flexibility ratio compared to a closed lining. The flexibility ratio will be discussed further on with reference to the Son and Cording (2006) article. There is one big difference between this model and the one used by Malmgren and Nordlund (2007) in that the irregular shape is only seen in the rock surface. The shotcrete lining is supposed to fill the irregularity which will result in a smooth surface on the inside of the shotcrete lining, where the surface is flat. To a certain degree, this is true in-situ since the depressions of the rock surface tend to be filled with more shotcrete compared to the shotcrete coverage of the peaks, which means that the inside of the shotcrete shell is more regular than the outside. But when the
irregularities are large, the inside of the shotcrete surface tends to be quite irregular too. The shotcrete thickness thus varies when the surrounding rock surface changes.

Note that no rock bolts are used in the analysis so that the shotcrete is used as the only rock support. The shotcrete lining is also thicker than the one used in Nilsson (2003) or Malmgren and Nordlund (2007). This affects the stiffness of the shotcrete shell and also influences the deflections around the lining. It also means that there will probably be higher stresses in the lining since it is not very flexible, even though has an open shape.

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**Fig. 2.20:** Geometric properties of the model, note the smooth inside of the tunnel and the sharp edges of the irregularities. From Borio and Peila (2009).
Parameters analyzed in the model

The irregularity is included as a parameter in the model by changing the amplitude of the tooth-shaped rock edges while keeping the minimum shotcrete coverage to 200 mm. The irregularity is varied between three different values: 0, 200 mm and 400 mm. The wavelength is 2 m.

The rock horizontal pressure coefficient $K$, which describes the horizontal pressure according to Eq. (2.3), is also used as a parameter, and has a variation from 0.5 to 1. This changes the stress distribution around the lining and influences both the deflections around the lining and the stresses in the lining.

$$\sigma_H = K \cdot \sigma_V \quad (2.3)$$

Where

- $\sigma_H$ is the horizontal pressure
- $\sigma_V$ is the vertical pressure
- $K$ is the horizontal pressure coefficient

The cohesion of the rock is also used a parameter, and has a variation from 0 to 0.2 MPa, thereby simulating different ground conditions.

Parameters investigated

- Irregularity: 0, 200 and 400 mm
- Shotcrete thickness
- Pressure coefficient $K$, 0.5 or 1
- Cohesion of the rock

Results

The results show the difference between the resulting stresses when the lining is regular and irregular. Fig. 2.21 shows the high tensile stresses around one of the edges of the irregular lining as well as the stress distribution for the flat slab. It is obvious that the tensile stresses are much higher near the rock edge.

Fig. 2.22 shows the stresses at the parts of the shotcrete lining that are specified in Fig. 2.23. It is obvious that the stresses increase when the irregularity increases. It is interesting to note the difference between this model and the one used by Nilsson (2003), which had smooth edges, and this one has sharp edges that are probably more realistic. This means that there will be local
stresses at the edges that can damage the shotcrete. The same is true for the model used by Malmgren and Nordlund (2007).

Fig. 2.21: Maximum principals stresses in the lining. From Borio and Peila (2009).

Fig. 2.22: Stresses in lining. From Borio and Peila (2009).
2.4 Ground-lining interaction in rock tunnelling

The Son and Cording (2006) article consists of a numerical treatment of the influence of irregularity on shotcrete linings and develops a model to analyze the effects from different stress states in soft ground. The model is analyzed in the same program as the Malmgren and Nordlund (2007) model, i.e. UDEC (2009), and is a 2D model with plane strain set-up. The model focuses on the interaction between the ground and the lining, especially by using the concept of the flexibility ratio. It assumes that the lining is closed, which is not the case in Malmgren and Nordlund (2007) or Borio and Peila (2009), where the floor is not covered with shotcrete, which reduces the stiffness. A lining is said to be flexible if there are no large bending moments in any part of the lining, and rigid if the deflection will be insignificant while the bending moments are larger, according to Peck et al. (1972). Most tunnels obviously have a stiffness that is somewhere in between rigid and flexible, since there always it is impossible to build completely rigid or flexible linings. The difference between the flexible and rigid tunnel depends on the stiffness of the lining and ground, but also on the diameter of the tunnel, which influences how the lining will be affected by the pressure from the ground. The tunnel used in Son and Cording (2006) has a circular cross-section. The tunnel is assumed to be at a depth of 30 meter and have a diameter of 10 m, as can be seen in Fig. 2.24.
Since there is double symmetry in the tunnel, around the horizontal and vertical axes, only one quarter of the tunnel area is used in the modelling. The boundary conditions are modelled by using roller supports at the boundaries (Fig. 2.25). The pressure is distributed along the side of the model and horizontal pressure is varied to study the influence of the pressure coefficient on the stress distribution of the model.

The interface strength is modelled with Eq. (2.1), the Mohr-Coulomb linear assumption. After interface failure, which happens when the compressive stress, tensile stress or shear stress is higher than the interface resistance, the contact will only have a frictional shear resistance. The
tensile, compressive and shear strengths have been defined based on earlier tests and references in literature, according to Son and Cording (2006). The thickness and the shape of the lining are parameters in the model. This means that the variation in thickness will also affect the irregularity of the lining. This is quite realistic, since the depressions of the surface tend to get filled with more shotcrete than the protrusions, thereby creating a variation in thickness, as seen in Fig. 2.26. The geometry of the lining is similar to the one used in Borio and Peila (2009).

Fig. 2.26: Variation in shotcrete thickness along the irregular rock surface. From Son and Cording (2006).

Parameters investigated

- Irregularity
- Stiffness of surrounding ground
- Horizontal pressure
- Flexibility ratio

The flexibility ratio depends on both the properties of the ground and the lining and is defined in Eq. (2.4). It is a concept first introduced by Peck et al. (1972). It is derived by dividing the flexural stiffness of the ground by the flexural stiffness of the liner, thereby making an expression that depends of their interaction.
\[ F = \frac{E_g}{6E_l I_l} \frac{(1 + \nu_g)}{(1 - \nu_l^2)} \frac{1}{R^3} \]  

(2.4)

Where

- \( E_g \) is the Young's modulus of the surrounding ground
- \( \nu_g \) is the Poisson’s ratio of the surrounding ground
- \( E_l \) is the Young’s modulus of the lining
- \( \nu_l \) is the Poisson’s ratio of the lining
- \( I_l \) is the moment of inertia of the lining
- \( R \) is the radius of the lining

For each change of parameter in the model, the normalized moment ratio, Eq. (2.5), and the normalized thrust ratio, Eq. (2.6), were measures to account for the effect in the lining. These parameters were used for the ordinate in Fig. 2.27 a-c and 2.28 a-c, while the abscissa describes the flexibility ratio of interaction between the ground and the lining. The results show that both irregularity and the flexibility ratio, i.e. the relationship between the stiffness of the lining and the stiffness of the surrounding ground, are very important factors that influence the behaviour of the tunnel and should be taken into account in design.

\[ M_N = \frac{M}{\gamma Z R^2} \]  

(2.5)

\[ T_N = \frac{T}{\gamma Z R} \]  

(2.6)

Where

- \( M_N \) is the normalized moment ratio
- \( M \) is the lining moment
- \( \gamma \) is the unit weight of the ground
Z is the tunnel depth to the springline

R is the radius of the lining

\( T_N \) is the normalized thrust ratio

\( T \) is the lining thrust

Fig. 2.27 a: Normalized moment ratio at tunnel crown, \( K=0 \). From Son and Cording (2006).
Fig. 2.27 b: Normalized moment ratio at tunnel crown, $K=1$. From Son and Cording (2006).

Fig. 2.27 c: Normalized moment ratio at tunnel crown, $K=2$. From Son and Cording (2006).
Fig. 2.28 a: Normalized thrust ratio at tunnel crown, $K=0$. From Son and Cording (2006).

Fig. 2.28 b: Normalized thrust ratio at tunnel crown, $K=1$. From Son and Cording (2006).
The first conclusion from the results is that the flexibility ratio is very important and affects the results quite a lot. When the flexibility ratio increases, i.e. the stiffness decreases, the stresses in the shotcrete lining increases drastically. This is especially true if the horizontal pressure and the pressure coefficient are large.

The irregularities also have a very big effect on the stress distribution in the shotcrete lining. The most interesting parameter seems to be the irregularity height to liner thickness ratio, according to Son and Cording (2006). It also seems that if the shotcrete lining can be constructed to increase the flexibility ratio, the irregularities might not be important. This indicates that it would be valuable to increase the flexibility of the lining with damaging the effectiveness of the rock support. Another thing that can be noted is that the height to thickness ratio of the lining also influences the flexibility ratio, something that Nilsson (2003) also notes for the stiffness of the shotcrete slab which increased as the irregularity increased, while the thickness was uniform.

### 2.5 Summary of important parameters

The main results from previous research are collected in this section. This forms the basis for the numerical analysis described in chapter 4, where some of the parameters discussed here are used in the numerical model. The results from the numerical analysis are discussed and compared to the results from previous research and practical aspects of construction in chapter 5.
**Rock bolt position**

The placement of the rock bolts are of great importance for the load bearing capacity. The research made by Nilsson (2003) and Malmgren and Nordlund (2007) indicates that this is the case and that the effect is significant. In Nilsson (2003), the effect of a rock bolting pattern at the peaks is a 25 % higher load capacity, both when the shotcrete thickness is 40 mm and 80 mm. In Malmgren and Nordlund (2007), the effect is slightly smaller but seems to affect both the lining and the interface. These conclusions are based on the assumption that the irregularity is regular, which is probably not the case in-situ, and this assumption influences the ability to place the rock bolts at the optimal positions. The articles by Borio and Peila (2009) and Son and Cording (2006) do not include this parameter, but it is likely that it would yield the same result.

**Boundary conditions**

The boundary conditions influence the load bearing capacity in a major way since it influences the way the stresses are developed in the shotcrete lining and how they are redistributed along the lining. Nilsson (2003) is the only one that models the shotcrete in 3D, which makes it difficult to conclude anything from the other articles which use a plane strain setup.

**Shotcrete thickness**

The thickness of the shotcrete is very important, both in itself and when the shotcrete lining is irregular. The effects of the irregularity are much more noticeable when the shotcrete lining is 40 mm than when 80 mm as shown by Nilsson (2003). If the lining thickness is increased, this will also increase the lining stiffness, which decreases the flexibility ration, which will increase the moment and thrust in the lining, according to Son and Cording (2006).

**Variation in shotcrete thickness**

Son and Coring (2006) demonstrates that the irregularity height to liner thickness ratio is very important since the moment and thrust in the liner is very dependent on these parameters. Nilsson (2003) and Malmgren and Nordlund (2007) uses a uniform shotcrete thickness which means that the impact of this parameter cannot be estimated. It could be possible that the variation in shotcrete thickness will results in an unfavourable stress distribution, but it is difficult to estimate the effect. Both Borio and Peila (2009) and Son and Cording (2006) use a variation in shotcrete thickness but does not camper the influence of this parameter compared to the stress distribution of the regular lining.
**Stiffness**

The stiffness has a great influence and the shotcrete linings used in Borio and Peila (2009) and Son and Cording (2006) had a much higher stiffness than the models used in Nilsson (2003) and Malmgren and Nordlund (2007). The stiffness of the model is influenced by the irregularity and shotcrete thickness, according to Nilsson (2003), if the lining is closed at the floor, according to Malmgren and Nordlund (2007), and the properties of the ground, as well as the radius of the lining, according to Son and Cording (2006). The stiffness will probably influence the stress distribution of the lining, according to the results from Son and Cording (2006). The results from Nilsson (2003) indicate that the irregular shotcrete slab has a higher stiffness than the regular slab when the shotcrete thickness is uniform. Son and Cording (2006) also investigate the effect of changes in rock mass properties, which influences the stiffness of the surrounding mass. The stiffness of both the rock mass and the lining, which is described by the flexibility ratio, seems to influence the behaviour of the lining significantly and seems to be of great interest.

**Rock strength**

The rock strength has a large impact on the number of failures in the interface, as Malmgren and Nordlund (2007) shows. This influence is hard to estimate quantitatively, but it is probably also affects the interface, which the results from Malmgren and Nordlund (2007) indicates. The rock mass properties obviously also influence the possible load effects and the stiffness of the lining, which makes it important to estimate the influence of the changes in its properties.

**Interface strength**

The interface strength has a large impact on the number of failures in the interface, according to Malmgren and Nordlund (2007), who showed that the influence of the interface strength also was influenced by the irregularity of the lining. The interface strength was not used as a parameter in Borio and Peila (2009) and Son and Cording (2008) and only the lining stresses were collected from the model. Since the interface strength also impacts the stresses in the lining when the shotcrete thickness changes, fewer failures in the lining can mean more failures in the interface. The number of failures in the lining and in the interface seems to correspond. High interface strength is favourably in all cases, as Malmgren and Nordlund (2007) has discovered.

**EDZ, excavation damaged zone**

The effect from the EDZ is marginal, and decreases with the irregularity, according to Malmgren and Nordlund (2008). This is also influenced by the definition of the EDZ. Malmgren and
Nordlund (2007) used decreased rock strength and Young’s modulus to estimate the effect of the blast but this could also by simulated by and increased number of fractures near the tunnel lining, or an impact in any other material parameter, which makes it difficult to estimate the influence of the EDZ, according to Malmgren and Nordlund (2007).

**Irregularity**

The irregularity of the rock surface is one of the most important parameters that influence the stress distribution. Malmgren and Nordlund (2007) showed that even small irregularities influence that lining in a major way. There are, however, uncertainties about how the irregularity influences the shotcrete and the interconnection between the irregularity and the number of failures in the interface and in the lining, since these parameters seems both seems to be affected by the irregularity. Nilsson (2003) shows that the irregularity has a significant impact on the load bearing capacity of an irregular shotcrete slab compared to a regular slab. The irregularity of the rock surface also influences the stiffness and variation in thickness of the shotcrete lining.

**Reduced/increases loads**

The different load cases used by Malmgren and Nordlund (2007) shows that the case where the horizontal load decreased was the most dangerous. This load case may, however, be more typical in mining excavation than in ordinary underground excavation and may not be of interest in typical civil engineering excavations.
3. Practical aspects of the shotcrete lining

In this chapter some of the practical problems that affect shotcrete design are discussed and the difficulties that are present in construction are explained. In construction several parameters are very difficult to measure and cost is a large practical constraint. First an explanation of the overbreak is supplied and it is explained why it cannot be reduced beyond a certain level. It is followed by a discussion of how larger underground projects that contains shotcrete as rock support is organized in Sweden, from the government authority that controls the projects, and presents it to the government who finally makes the decision, to the designer and construction firm. The designer usually makes the shotcrete design as well as controls and investigates several parameters during construction.

This chapter is based on material from Swedish tunnel projects and discussions with people professionally involved in the construction and design of these kinds of projects. They have experience of practical problems that affect the quality of the construction and design and have stressed that the reality is very complex and difficult. Some of the problems that make tunnelling different from other civil engineering projects are related to, and why experience and active risk management is very important. Many of the parameters that design of rock supports is based on are difficult to measure and the knowledge about these is at best uncertain. This means that experience from earlier projects and empirical methods will be very important even though there are clear disadvantages of using this in design. This will also show the contrast between the theory used to predict stresses in the shotcrete lining presented in chapter 2 and the more complex reality. These two parts are further discussed in the final chapter.

3.1 Reasons for overbreak

There are several reasons for the irregularity of the rock surface and this is influenced by many parameters that are difficult to forecast and control during construction. Among the factors that influence the irregularity, or overbreak which is a technical term commonly used, are the geological properties of the rock mass, the quality of the workmanship, the blasting scheme and the mucking and scaling of the blasted surface. There are also economic reasons for having at least some overbreak which results in an irregular rock surface.

As has previously been stated, the geological properties of the rock mass are very difficult and also expensive to measure and estimate. Even though there is the possibility of changing paths or change the geometry in the ground if the properties of the rock are known, in most cases the geological quality must be accepted as a parameter that cannot be changed, according to Frank Ouchterlony (2009).
The geological properties depend on a number of parameters, but the both quality of the rock and the orientation of the joints are very important. If the rock is severely fractured it is quite evident that this will influence the irregularity of the rock surface after the blast. The orientation of the joints will also influence the post-blast shape, especially when the joints are oriented in the same direction as the tunnel. This means that it will be much easier for the loose rocks to detach from the rock surface and fall into the underground opening, something that is described by Nord and Stille (1990).

Drilling and blasting during underground work is naturally a very important parameter considering the irregularity of the rock surface. The most important part of the blast scheme is the peripheral blasting charges that will directly affect the rock wall, usually being the last blasts in the blasting sequence. The skill of the workmen is naturally very important. The shape of the rock surface also depends on the distance between each blast since it influences the time the rock will stay without rock support and also the ability to blast through heterogeneous rock. This is also an economical and organizational questions, since the length of each round naturally influence the time and cost of the construction. Another factor is the ability of the workmen to adequately muck and scale the tunnel after the blast, but his has much less influence than the other factors, though it is still important.

The economic parameters also influence the shape of the tunnel. It is often much more expensive to remove the underbreak, i.e. the rock that reaches into the tunnel space, the opposite of the overbreak, than to fill it. This means that there is an incentive to at least allow some overbreak, which can in some cases be larger than what is needed from a design point of view. Borio and Peila (2009) present studies where the average thickness of the overbreak varies from 10 % of the tunnel diameter in normal rock to 25 % in severely fractured rocks.

### 3.2 Design and construction of a shotcrete lining

The construction and design of a shotcrete lining for a tunnel is in many aspects very different from the ordinary design of other structural items. Since the design process involves quantities that are unknown, or known with a very low degree of certainty, an iterative way of designing the shotcrete lining has to be used. The construction also involves several partners that have to communicate throughout the process and who all have different interests and knowledge of the construction, as demonstrated in Fig. 3.1. Risk management is very important and several empirical methods are usually used to confirm that the rock support will be good enough to ensure safety. The following part of the report describes how this process works in a major Swedish infrastructure project.
Fig. 3.1: Organizational structure of a larger tunnel project.

Pre-plan geological investigation

The decision to launch a large scale infrastructure project is a political decision, but is usually made based on suggestions by experts working for government agencies. In Sweden, the biggest agencies are Vägverket (The Swedish Road and transport authority) and Banverket (The Swedish railroad authority). These agencies continuously develop new plans for roads and railroads to improve the infrastructure of the country. They use traffic analysts to evaluate and develop proposals for new roads and railroads in Sweden.

There will also be a technical evaluation of the proposals. The agencies enlist in-house or outsourced expert that perform geological tests for tunnel to get an initial idea of the costs. The first budget is based on these estimations. In the example of the Norra länken, the design office used was SWECO AB, according to Dan Palmgren (2009) at Vägverket, who performed the geological tests for Vägverket. Based on the results from these tests, different road or railroad corridors are planned, and several alternatives are prepared of the final decision. The prioritized projects will then be proposed for the government if they are large or by the agencies themselves if they are smaller.
Refined geological investigation and lining design by empirical methods

A more refined geological investigation is then performed on selected parts of the road corridor. These tests will decide the parameters for the shotcrete design, together with the experience from earlier projects. This design is often done by the same design office that investigates the geological condition, in the case of Norra länken SWECO, according to Dan Palmgren (2009).

The design is usually based on empirical methods, which means that the experience from earlier projects is used to give an estimation of what will be a reasonable rock support. For any typical Q-value or RMR values (rock mass rating) or any other empirical parameter (explained below) will give a “typical rock support”, based on experience. The design work will give the construction firm three design values can be changed depending on the conditions of the rock surface after the drilling and blasting. The Q-value or RMR-values depend on the measuring method. They construction firm will be given quite detailed instruction for each part of the tunnel. The specified design factors that are:

- Shotcrete lining thickness
- Shotcrete lining strength
- Rock bolt distance

Other factors, such as the strength of the rock bolts, the size and form of the bolt washers, specific placement of the bolts, etc, can be used. Normally this is not used in the design, mainly because of practical reasons. Sometimes unreinforced shotcrete is used and sometimes special solutions are needed, according to Frank Ouchterlony (2009). An example of this could be a supporting arch, where reinforcement bars are used together with shotcrete to create underground reinforcement that resists much larger loads than what is possible with reinforced shotcrete and rock bolts.

Safety is one of the biggest issues in the design process, and the design will be very different if the underground space is going to be used by people or simply as an underground storage facility, or similar use, according to Nord and Stille (1990). The experiences of the safety from earlier projects are also used together with control measures to ensure the safety during the lifetime of the shotcrete lining.

The design will also be based on requirements from the government agency. This concerns the quality of the material used (shotcrete and rock bolts) but also specifies the necessary shotcrete thickness, shotcrete quality, and rock bolt distance. These values are based on the BKR, the Swedish building code, according to Nord and Stille (1990).

The design office then prepares a list of different rock supports that can be used depending on the conditions of the rock after the blasts are done. It will describe the best lining design based on the empirical estimation of the rock after the blast. This value can be the Q-value or RMR or any other method that is proved to be useful in the conditions that exist in the specified
geological area. The Q-value is an empirical value based on several geological factors, such as joint number, water penetration, rock quality designation, etc. It is an indicator of the rock support needed for the specific part of the shotcrete lining and it has been used in the Norra länken to measure the quality of the rock mass.

For more advanced parts of a tunnel, the design will be based on calculations here specified later in the text. In Sweden these are mainly based on the weight of loose rock in the ceiling and walls. In other countries the loads can be dramatically different based on the ground conditions and other geological or practical factors.

**Construction**

The shotcrete lining is just one part of the construction phase which consists of a large organization with many people with different knowledge of the different parameters that affect the ground. They workplace tends be limited in volume and communication is very difficult, according to Nord and Stille (1990). Because of all these factors it is very important to have simple blueprint and construction directions. More specified information can easily be lost in the information flow, according to Jonas Holmgren (2009). The time is also very important since the opening date for the tunnel is usually decided, and the whole project will be influenced by this. Time is often more important than cost when the economic benefits will be quite small. This means that robust and easy solutions will always be appreciated, according to discussions made with Frank Ouchterlony (2009).

The construction firm will start spraying the shotcrete lining after the drill and blast has been done. The drill and blast procedure includes:

- Pre-grouting
- Drilling
- Charging
- Blasting
- Scaling
- Mucking

After the mucking has been done, a geologist (in Norra länken from SWECO) will investigate the empirical design value (Q-value or other empirical value) of the rock. This is often done by the design firm that originally did the shotcrete design, but in some cases by an independent firm. The investigation and classification of the Q-value (or any other value) is done by visual observations. The Q-value will then give the suggested shotcrete lining based on the calculations by the designer. The geologists also investigate the joint patterns and order special rock bolts if it is needed. This means that there is a flow of information and that the new information from the
construction can be used for control of how the lining performs, according to the interview with Jonas Holmgren (2009).

The shotcrete will then be sprayed on the tunnel wall and ceiling and the rock bolts will be placed over the shotcrete shell. In many parts of Norra länken, only the ceiling is covered with shotcrete. The deformations of the rock surface and the shotcrete lining will be measured to have control of the behaviour of the rock support.

The geometry of the underground opening is scanned before and after the shotcrete is sprayed to control that the thickness is conforming to the design throughout the shell. There are also several controls of the performance of the shotcrete and of possible joint the shotcrete lining, during the construction and when the structure is finished.

### 3.3 Examples from Swedish shotcrete design

An example of the calculations used in shotcrete design in Sweden is described in this section. These types of calculations are done when the load effects are considered to be so serious and unusual that the “typical support” based on empirical methods is not sufficient to ensure that the safety of the underground opening is achieved. In Sweden, the rock mass is usually hard and the most common loads are sections of loose rocks in the roof of the sides of the tunnel wall. This is different from in soft rock, where there often exist a uniform pressure on the tunnel lining, which means that it is important to consider the stiffness of the whole lining, which is done in Son and Cording (2006). In Sweden, this is normally not the case, although there have been some noteworthy exceptions, some of them discussed in Nord and Stille (1990). Since the most common problem in Sweden is larger blocks, in some cases these can be fastened to the rock mass with rock bolts which is necessary if they are very big and it is obvious that they might damage the construction or affect the safety of the underground opening, according to Jonas Holmgren (2009). The blocks can, however, also be secured by using either unreinforced shotcrete, which has been done since the beginning of the 1960s, or by using reinforced shotcrete combined with rock bolts to resist the moment and shear forces in the shotcrete.

The guidelines briefly discussed in this part have been used in the design of “Södra länken”, a large Swedish road tunnel project in the Stockholm area. Calculations are usually carried out to assess the risk from the following failure modes:

- Adhesion failure
- Moment induced failure
- Moment induced failure at the rock bolt washers

In the calculations methods used in Sweden, the safety factor \( F_s = 1.5 \), has been prescribed in the Swedish construction code (BKR), according to Fredriksson (1992). It also depends on the safety
level of the underground opening. It is obvious that underground openings where people normally are located have much more strict standards than other facilities. It is also based on an assumption about the difficulty of the project, which is called in Sweden called geotechnical class, where the most difficult projects are in class 3, GK3. The geotechnical class also influences the organization of the project and it is in some cases required to have an independent adviser to limit the risks, according to Frank Ouchterlony (2009).

**Adhesion failure**

Adhesion failure refers to a failure in the ability of the shotcrete to resist the local loads directed against the surface from single rocks positioned between the rock bolts. The load effect from a single rock can be calculated using geometrical assumptions about the rock. Some different geometry that is possible in Swedish conditions is shown in Fig. 3.2. The engineering geologist that performs the investigation of the rock surface between every drill and blast session will also assess the risk of large blocks falling down. In some cases the risk is negligible, but in other cases the roof can consist of one large rock that may or not fall down, which is enough to take precautions, according to Frank Ouchterlony (2009).

The geometric assumptions are used both for reinforced shotcrete combined with rock bolts, which is the most commonly used rock support in Sweden, and also for unreinforced shotcrete. The unreinforced shotcrete was used widely in Sweden from the 1960s and onwards during the time that the method was further developed. If the loads are not very large, which is sometimes the case in Sweden where there is often hard rock with only minor risks of rock fallout, unreinforced shotcrete is possible to use. There are also other varieties, with reinforcement bars, but these are much more difficult to construct and take much more time both for the construction firm and the designer, according to Jonas Sahlström (2009).

The loads that affect the shotcrete shell can be calculated if assumptions about the geometrics of the possible loads are used. In reality different assumptions are utilized to test the sensitivity of the assumptions and their effects on the result to find the most dangerous load case. There is also an uncertainty about how the volume of the loose rock, something that is very hard to get a realistic idea about. The maximum angle of the rock against the wall or ceiling can vary between 45º and 60º, but 45º is more likely in Swedish conditions. This was e.g. used for the calculations in Södra länken, as well as in other Swedish infrastructure projects, and has been chosen by experience. This is also true for the other assumptions, since the parameters are difficult to measure and control.
Fig. 3.2: Different possible geometries that can affect the shotcrete shell.

The loads that affect the shotcrete shell can be calculated if assumptions about the geometrics of the possible loads are used. In reality different assumptions are utilized to test the sensitivity of the assumptions and their effects on the result to find the most dangerous load case. There is also an uncertainty about how the volume of the loose rock, something that is very hard to get a realistic idea about. The maximum angle of the rock against the wall or ceiling can vary between 45° and 60°, but 45° is more likely in Swedish conditions. This was e.g. used for the calculations in Södra länken, as well as in other Swedish infrastructure projects, and has been chosen by experience. This is also true for the other assumptions, since the parameters are difficult to measure and control.

The load effect can be described as:

\[ C_E = V_{\text{rock}} \cdot g \cdot \rho_{\text{rock}} \]  \hspace{1cm} (3.1)

Where

- \( V_R \) is the volume of the rock
- \( G \) is the gravity constant
- \( \rho_{\text{rock}} \) is the density of the rock
The load capacity a shotcrete layer can also be calculated using Eq. (3.1). It must be noted that the variables are assumptions, and that practical problems can affect the properties of the parameters. In chapter 2, the adhesion was described in simple terms, but it is more complicated in real tunnel surfaces and depends on a multitude of parameters, according to Hahn (1982).

\[ C_R = O \cdot \delta \cdot \sigma_{ad} \]  

(3.2)

Where

- \( C_R \) is the load capacity of the shotcrete layer
- \( O \) is the circumference of the rock
- \( \delta \) is the load bearing area, depending on the layer thickness
- \( \sigma_{ad} \) is the adhesion strength

The adhesion strength depends on the rock material, the surface roughness and the ability to clean the rock surface before concrete spraying is done, which may be difficult in-situ. The adhesion strength is normally between 1-2 MPa in Swedish conditions, but can get significantly lower in materials such as clay shale, according to Fredriksson (1994).

The load bearing area depends on the layer thickness of the shotcrete surface, where increased thickness increases the load bearing area, since more of the shotcrete can penetrate the cracks in the rock and create a larger area that can resist the load. It is an almost linear relationship, but is also affected by great uncertainties. The following equation can then be solved to find the load bearing area:

\[ C_E = \frac{C_R}{F_S} \]  

(3.3)

The combination of rock bolt distance and shotcrete thickness that satisfies this condition can then be found, giving the design values used in construction based on this failure mode.
**Bending moment failure**

Apart from the adhesion failure, there can also be a bending moment failure in the lining. This can be a possible failure mode when the shotcrete surface is subjected to a single load and the adhesion is bad and when the rock is severely damaged and pulverized, which creates a distributed load on the rock surface, as can be seen in Fig. 3.4. In these cases reinforced shotcrete combined with rock bolts are need to resist the high moments in the shotcrete.

There can also be a punching failure in the shotcrete around the rock bolts. Extensive tests performed in Sweden have shown that this is an unlikely failure mode when the rock bolt washer are normally large (around 160 x 160 mm²), and is for this reason not considered in the design model used in Sweden, according to Fredriksson (1994). The irregular shotcrete surface is approximated as a flat slab in the calculations presently used. This is the case both in Sweden and in other countries, which of course results in design values that are far from the real case. This can still be practically useful when the calculations are combined with empirical methods and it also depends on how irregular the shotcrete surface is.

If these assumptions are used and the interface between shotcrete and rock mass is assumed to be low, the resulting shotcrete slab work like an ordinary concrete slab supported by steel columns or concrete columns. There can either be single blocks that affects the shotcrete surface or a distributed load that affects the surface. In the calculation done for Södra länken, only the case where the load is distributed was considered. An illustration of that load is shown in Fig 3.3 a), where the rock bolts support the shotcrete as if it was an ordinary slab affected by a distributed load. Fig. 3.3 b) shows the assumed inner friction angle that affects the structure and the load distribution.

The moment resistance $M_R$ in the shotcrete slab can be described by Eq. (3.3), which is an empirical relationship used by Fredriksson and Stille (1992).

$$M_R = \frac{(h^2 \cdot \mu_s \cdot \sigma_{su})}{6 \cdot k} \quad (3.4)$$

Where

- $h$ is the shotcrete thickness
- $\mu_s$ is the steel fiber ratio
- $\sigma_{su}$ is the yield stress of the steel fibres
- $k$ is an empirical factor depending on the steel fiber ratio
If the load consists of a single block, the moment load effect, $M_E$, is calculated in the same way as for the adhesion failure, by using geometrical relationships. If the load effect consists of loose fractured rock fragments, an assumption can be made about the inner friction angle in the loose material, whereby the resulting load can be found. An example of the loose fragment load is shown in Fig. 6.5. Numerical models (finite element models) are used to calculate the bending moments in the flat slab to get $M_E$, defined as:

$$M_E = \frac{M_R}{F_S}$$

(3.5)

The combination of distances between rock bolts and shotcrete thickness that satisfies this condition can then be found and used as design values for the particular failure mode.
**Bending moment failure at the rock bolts washers**

By using Eq. (3.4), the load bearing resistance of the shotcrete can be calculated, depending on the steel fiber ratio and the thickness. It is then possible to calculate the load on the washer. Fig. 3.4 shows an assumption of the possible load distribution that is used in these calculations. The angle of the load can vary between 30-60 degrees. Using this assumption, the moment load effect can be calculated, using numerical models to simulate a simply supported slab subjected to a distributed load.

If it is realistic that the rock is relatively stiff and will interact with the shotcrete, the load on the washers can be reduced and the final load effect calculated, which eventually gives the necessary shotcrete thickness and rock bolts distance. This failure mode is typically the most dangerous one and usually determines the design values in construction.

**Final analysis**

When the necessary shotcrete thickness and rock bolts distance have been calculated, the values are compared and the most severe load effect will be used as the design load. The required shotcrete thickness and rock bolt distance will then be used in the design. As previously stated, failure at the washers is usually the most dangerous failure mode.

*Fig. 3.4: Bending moment at the washer subjected to load from fractured rock masses*
3.5 Construction of a shotcrete lining

There are several practical problems that can affect the shotcrete lining and the design of the shotcrete lining. Many of these problems are of an economical or organizational origin, difficult to change or are part of the construction process. This will influence the quality of the result and are very important to consider since any improvements in design methods easily can be offset by the practical problems that arise when new ideas are implemented at the construction site.

Adhesion strength

The adhesion strength at the shotcrete-rock interface is a very important parameter that probably increases the load bearing capacity with increasing adhesion strength. This parameter depends on many different factors, such as the rock quality and type, but the condition of the surface is also very important. The surface should be clean before the shotcrete is sprayed on the rock. This is done with a car mounted with a cleaning nozzle. In Norra länken this machine has, however, not been able to get very close to the rock wall since the tunnel section is so large, which has resulted in very low adhesion strengths. If this is the case in other places, it may not be safe to use specified adhesion strengths in design process, according to Frank Ouchterlony (2009).

Shotcrete thickness

It is difficult to produce a uniform shotcrete layer with a hydraulically operating machine, a so called shotcrete robot. As already remarked, depressions tends to be filled with more shotcrete than covers the peaks, and the result depends very much of the individual ability of the machine operator. This means that it will be very difficult to use specified thicknesses in design due to the large variation.

Communication

Since the construction of a shotcrete lining is only a part of the bigger tunnel construction, a large organization uses a very tiny space and the time aspect is always important. The motto “better safe than sorry” is often used, since the results of insufficient rock support can damage the whole construction and be very expensive as well as take a lot of time.
**Placement of rock bolts**

The rock bolts are usually placed in a grid-like pattern where no difference is made whether they are placed on peaks or depressions. The placement of the rock bolts is probably very important, but there are also practical problems when rock bolts must be placed on surfaces that are highly irregular, especially when both the placement at the peaks and the distance between the rock bolts must be considered. It could be difficult to find the optimal placement for an irregular shotcrete shell when no clear pattern in the irregularity.
4. Numerical experiments with shotcrete linings

In this chapter the numerical models are described in some detail. The load effects that the shotcrete shell is subjected to, as well as the different parameters that are changes in the different examples are given. In chapter 5 the results of the numerical simulations are presented together with discussion about the results and possible conclusions. The analysis has been performed in Abaqus (Abaqus, 2009), a general finite element program used in several areas related to construction, and the material model has been taken from Nilsson (2003).

4.1 Model geometry

The geometric properties of the model used in the numerical simulations have been created by combining examples presented by Nilsson (2003) with those by Malmgren and Nordlund (2007). The geometry is supposed to simulate a tunnel for a two-lane road in hard rock, which is quite common in Sweden.

The regular model is 10 meters wide, 5.5 meter high and 8 meters long. The shape is shown in Fig. 4.1 and Fig 4.2. One version of the model will have the same height, length and depth, but also have an irregularity that is simulated in the same way as the model used by Nilsson (2003). The wavelength of the irregular will be 2 m and the irregularity 400 mm. The irregularity is defined as the double amplitude of the irregularities, as defined by Nilsson (2003).

Fig 4.1: Cross section of studied tunnel.
The shotcrete thickness will be varied in example 4 of the model to more closely simulate a “real” shotcrete shell where the depressions of the shell will have a larger shotcrete thickness than the peaks. Three thickness variations are used in these models. These are based on the assumption that the minimum thickness is 25 mm and the maximum thickness changes between 75 mm, 95 mm and 125 mm. The mean value of the shotcrete thickness is 50 mm, 60 mm and 75 mm. The variation in thickness is simulated by a sinus-curve function between the maximum and minimum value. An example is shown in Fig. 4.3 where the thickness is 25 mm in the red parts of the model and 75 mm in the blue parts of the model. The model was meshed using a quad-dominated mesh, Fig. 4.4.

(a)                                                                      (b)

Fig 4.3: Variation of thickness in the shotcrete lining (a). Darker spots indicates a 25 mm thickness, lighter spots a 75 mm thickness. Variation in shotcrete thickness across the length of the tunnel model (b).
Three different variations of the model are used: the regular model, the irregular model with the rock bolts placed at the peaks of the irregularities, and the irregular model with the rock bolts placed at the depressions of the irregularities. The same variations are made in Nilsson (2003) but for a shotcrete slab instead of the whole lining.

The rock bolts are simulated by specifying boundary conditions for rectangular elements on the shotcrete lining. These elements have a 160 x 160 mm\(^2\) area and have the boundary condition encastré, which means that they are not permitted to move or rotate. The rock bolts are simulated in the same way in Nilsson (2003). The sizes of the rock bolt washers have been specified based on Nord and Stille (1990).

4.2 Material model

The material model is the same as used by Nilsson (2003). It is based on numerical test performed on beams with the same material properties as the shotcrete used in the analysis. The material properties of the shotcrete are:

- Compressive strength \( f_c = 50 \text{ MPa} \)
- Elastic (Young’s) modulus \( E = 34 \text{ GPa} \)
- Tensile strength \( f_t = 4.5 \text{ MPa} \)
- Poisson’s ratio \( \nu = 0.2 \)
The behaviour of the tensile response has been created by fracture mechanics, where the energy required to open the crack has been defined. The stress-displacement relationship used in the model, where the softening part is assumed to be linear, is shown in Fig 4.5.

![Stress-strain relationship of the shotcrete.](image)

Fig. 4.5: Stress-strain relationship of the shotcrete.

Since the numerical model does not include an analysis of any tensile or compressive failure the results from any failure cannot be detected. If the tensile or compressive principal stresses exceed the maximum compressive or tensile strengths of the shotcrete, a failure will probably occur if the affected area is big enough. If the affected area is small, the stresses will probably be redistributed along the shotcrete lining if failure occurs and the shotcrete lining would probably not be affected in a major way. If a larger part of the lining is subjected to compressive or tensile principal stresses that exceed the strength of the material, the result will probably be a failure of a larger part of the lining. For example, one of the rock bolts can be subjected to stresses that can exceed the tensile strength of the bolt, which will be defective.

### 4.3 Assumptions and simplifications used in the analysis

Since the interaction between shotcrete, rock and the rock bolts is so complicated, a number of simplifications and assumption have been used in this report. These assumptions and simplifications will obviously influence the results of the simulations. This makes it very important to describe the assumptions in detail:
• The bond strength between the shotcrete and rock surface has not been included in the numerical model. The interface strength will be described by the Mohr-Coulomb failure criterion \( \tau = c + \sigma' \cdot \tan\phi \). This relation has been deemed too difficult to include in the 3D-model.

• The rock bolts will not yield and are modelled as fixed elements on the surface. This is a somewhat realistic assumption since the concrete probably will fail before the bolts.

• The shotcrete quality is uniform, which is usually not the case in-situ, according to Hoek and Brown (1980).

• The maximum and minimum principal stresses have been used to estimate the effects of the shotcrete shell. These should be able to represent the stress states of the shotcrete lining, but there could be stress situations that does not reflects the overall stress distribution of the shell and where this methods could give faulty results.

• The shotcrete thickness will show a regular variation. In reality, the variation will be far more chaotic, which can be difficult to simulate.

• There are no sharp edges at the rock surface, the local geometry is “regular”

• The global geometry will be smooth and dominated by a variation dominated by sinus-waves, while the rock surface is dominated by chaotic variations.

• The loads on the shotcrete shell will be modelled by uniform pressure. In reality, the rocks that rest on the shotcrete shell will obviously have a stiffness, which allows the load to be redistributed on the shell, which can increase the load-bearing capacity of the shotcrete compared to the uniform pressure.
4.4 Example 1

In example 1, a load that simulates the force from a single rock is applied on top of the shotcrete lining. The load consists of uniform pressure. The position of the load is shown in Fig. 4.6.

![Diagram of load from a single rock on shotcrete lining]

*Fig. 4.6: The load from a single rock on the shotcrete lining. In this case, the cracks in the rock mass cannot restrain the rock, which instead will be carried by the shotcrete lining.*

The load will consist only of pressure, and hence will not have any stiffness. This is somewhat unrealistic, since a real rock has stiffness and could redistribute the load when the rock support is changed, for example if there is a failure in some part of the rock support that is subjected to the load from the rock, according to Jonas Holmgren (2009).

This is the most common load in Swedish conditions since the Swedish rock is quite hard. The loads are sometimes carried by the shotcrete lining, and in other cases where the rock formations are very big, one or more rock bolts will be applied at the surface of the single rock to fasten it against the rest of the rock mass, according to Nord and Stille (1990).

Three different variations of the model will be used: the regular model, the irregular one with the rock bolts at the peaks and the irregular one with the rock bolts at the depressions. The thickness of the shotcrete lining is changed from 25 mm to 125 mm for the three different models but is uniform for all three models, in order to study the influence of the shotcrete thickness.

The rock bolt that will be subjected to the highest stresses will represent the lining when the maximum principal compressive and tensile stresses and the influence length are calculated. The initial simulations showed that the maximum tensile and compressive principal stresses always appeared at the edges of the washers attached to the rock bolts. This confirms what Jonas Holmgren (2009) discussed in his interview when he described how the failures usually appears...
at the bolt washers, where the maximum stresses are located. In the first example where the load effect was simulated by a uniform pressure on the tunnel ceiling, the rock bolt washers closest to the area subjected to the load were obliviously most affected. The maximum principal stress has been defined as the highest stress that converges along the edge of the rock bolts washers. Since the washers are simulated by quadratic elements there is a theoretical divergence at the edge of the rock bolt. Instead the highest value that converges by the edge of the rock bolt washer has been used.

The deflection of the tunnel ceiling in the middle of the area that is subjected to the load will also be collected from the model. The influence of the high tensile principal stresses along the edges of the rock bolts have been estimated by using the “influence length” of the stress distribution. This has been estimated as the multiple of the rock bolt length that has been subjected to more than 80 % of the maximum tensile stress. An example is shown in Fig 4.7, where the influence length is 1.2.

![Image of Influence length](image)

*Fig. 4.7: Influence length of the maximum tensile principal stress along the edge of the bolt washer where the influence length is 1.2.*

The following results will be collected from the numerical simulations:

- Maximum principal tensile stress at the rock bolt subjected to the largest load.
- Maximum principal compressive stress at rock bolt subjected to the largest load.
- Influence length at rock bolt subjected to the largest load.
- Deflection of the tunnel ceiling relative to the vertical axes.
The aims of example 1 are to:

- Identify the differences between the stress distributions of the models.
- Estimate the possible damage of the models depending on the shotcrete thickness.
- Estimate the stiffness of the different models depending on the shotcrete thickness.
- Assess the difference between the rock bolt placement patterns

### 4.5 Example 2

In example 2, the influence of a uniform pressure from the ground is analyzed. The same variations of the models as in example 1 will be used. The load consists of a uniform pressure along the shotcrete lining. This simulates the uniform underground pressure when the pressure coefficient, i.e. the difference between the horizontal and vertical pressure, is 1. The thickness variation is the same as in example 1. The use of a uniform pressure means that there the load has no stiffness, which influences the results, as previously described.

In this example, the maximum principal stresses are collected from the rock bolt subjected to the largest load and from the two elements along the lining, in the ceiling and at the wall of the lining. These two elements are positioned according to Fig. 4.8. Both elements are as far away as possible from the rock bolts, i.e. they are in the middle of the rectangular grid pattern that the rock bolts create out of the shotcrete lining. The maximum principal stresses are collected from the inner side of the lining, since the highest tensile stresses will occur on that side of the lining.

![Fig. 4.8: Position of the elements on the shotcrete shell from where the principal stresses and the deflections are collected.](image-url)
The following results will be collected from the numerical simulations:

- Maximum principal stress at rock bolt subjected to the largest load.
- Maximum principal stress at the inner side of the ceiling element and wall element.
- Deflection of the ceiling relative to the vertical axes.
- Deflection of the wall relative to the horizontal axes.

The aims of example 2 are to:

- Estimate the changes in stress distributions between the regular model and the irregular models with different bolt patterns when the ground pressure is uniform.
- Assess the impact of the bolt placement pattern on the stress at the wall and ceiling of the shotcrete lining when the ground pressure is uniform.
- Estimate the changes in the stiffness of the different parts of the model when the irregularity increases, both for the wall and for the ceiling of the lining.

4.6 Example 3

Example 3 analyzes the effect of a change in the horizontal pressure coefficient, which makes it possible to compare the results to Borio and Peila (2009) and Son and Cording (2006). The shotcrete thickness will be uniformly 50 mm and the three above mentioned models are used. The parameter investigated is the influence of the stress coefficient $K$, the coefficient that specifies the horizontal stress in terms of a multiplier of the vertical stress. The coefficient $K$ will vary between 0, 0.5, 1 and 2. The difference in stress distribution when the stress coefficient is changed is simulated by increasing the uniform pressure on the lining walls. Since the rock-shotcrete interaction is not used in this report, this should be an accurate approach.

The following results will be collected:

- Maximum principal stress at the inner side of the ceiling element and wall element.
- Deflection of the ceiling relative to the vertical axes.
- Deflection of the wall relative to the horizontal axes.

The aims of example 3 are to:

- Assess the difference in stress distribution between the irregular and regular shotcrete shells when the pressure coefficient is changed.
- Estimate the influence of the placement of the rock bolts on the stress distribution when the horizontal pressure coefficient is changed.
- Estimate the influence of the shape and the placement of the rock bolts on the stiffness of the shotcrete lining when the horizontal pressure coefficient is changed.
4.7 Example 4

Example 4 analyses the influence of a variation shotcrete thickness. The three above mentioned models are used again. Three different shotcrete thickness variations are used, which simulate different degrees of shotcrete coverage. All have a minimum coverage of 25 mm, but the maximum coverage changes between 75, 95 and 125 mm. The mean coverage will be 50, 60 and 75 mm. The thickness distributions are described by Eqs. (4.1-4.3). The load consists of a uniform pressure, which is the same load as in example 2.

\[
t(x, y, z) = 0.050 + 0.025 \cdot \frac{\sin(2 \cdot x) + \sin(2 \cdot y) + \sin(2 \cdot z)}{3} \tag{4.1}
\]

\[
t(x, y, z) = 0.060 + 0.035 \cdot \frac{\sin(4 \cdot x) + \sin(4 \cdot y) + \sin(4 \cdot z)}{3} \tag{4.2}
\]

\[
t(x, y, z) = 0.075 + 0.050 \cdot \frac{\sin(4 \cdot x) + \sin(4 \cdot y) + \sin(4 \cdot z)}{3} \tag{4.3}
\]

The following results will be collected:

- Maximum principal stress at the rock bolt subjected to the largest load.
- Maximum principal stress at the inner side of the ceiling element and wall element.
- Deflection of the ceiling relative to the vertical axes.
- Deflection of the wall relative to the horizontal axes.

The aims of example 4 are to:

- Investigate the influence of the variation in thickness
- Determine if there is any difference between the regular and the irregular lining
- Assess the influence of the rock bolt placement.
5. Results and discussion

5.1 Results from example 1

The results from example 1 show that the shape of the shotcrete lining influences the stresses in the lining significantly. Fig. 5.1 shows the maximum stresses of the shotcrete structure for the regular model, the irregular model with the rock bolts placed at the depressions, and the irregular model with the rock bolts placed at the peak of the shotcrete shell. The numerical analysis showed that both the maximum tensile stresses and the maximum compressive stresses were located at the bolt washers. This is confirmed by the interview with Jonas Holmgren (2009).

The results in Fig. 5.1 confirm the results from Nilsson (2003), where the irregular shotcrete linings had a higher load bearing capacity compared to the regular lining. It also confirm the result from Nilsson (2003) that the rock bolt pattern where the bolts were placed at the peaks of the surface irregularities had a higher load bearing capacity compared to the pattern where the rock bolts were placed at the depressions of the surface irregularities. The difference between the irregular and the regular shotcrete linings seems to increase when the thickness increases, which means that the results are somewhat different compared to Nilsson (2003).

![Graph showing maximum tensile stresses at the edges of the bolt washers.](image)

*Fig. 5.1: Maximum tensile stresses at the edges of the bolt washers.*
The stresses were located along the edges of the bolt washers, but the exact location along the bolt washer depended greatly on the shotcrete thickness. Fig. 5.2 show the distribution of the maximum principals stresses along the edge of the bolt washers, where the stress converges to the edge closest to the load as the shotcrete thickness increases. The dark areas symbolize approximately 80% of the maximum stresses. In-situ, the bolt washer could probably move and rotate somewhat compared to the model where no movement or rotation was allowed along the surface of the bolt washer. This could reduce the maximum principal stresses at the washer.

![Fig. 5.2: The maximum principal stress along the bolt washer, the thickness is a: 25 mm, b: 35 mm, c: 50 mm, d: 60 mm.](image)

The minimum principal stresses along the edge of the bolt washer will behave quite differently, which is shown in Fig. 5.3. The results confirm the hypothesis put forward in Nilsson (2003) that the irregular shotcrete lining with rock bolts at the peaks, create a stress distribution around the rock bolts that are dominated by high compressive principal stresses. This pattern has noticeable higher compressive stresses at the rock bolts, especially when the shotcrete lining is thin, which can be seen in Fig. 5.3, but the difference decreases as the shotcrete lining thickness increase. As the thickness of the lining increases, it seems that the regular shotcrete lining will have higher compressive stresses compared to the irregular lining with any of the bolting pattern. This might suggest that the stress is re-distributed differently in the regular lining when the thickness increases than for the irregular lining with either bolting pattern.

As previously stated, the influence of the stresses on the load bearing capacity of the lining was also investigated by using a measurement of the stresses that can be called the influence length. This is the ratio between the lengths of the stress that surpass 80% of the maximum principal stresses along the bolt washer, compared to the length of the bolt washer. This should be a somewhat accurate measurement of the damage inflicted by the load. This ratio between the influence length and the bolt length are shown in Fig. 5.4.
Fig. 5.3: Minimum principal stress along the bolt washer.

Fig. 5.4: The influence length of the lining stresses.
Fig. 5.4 shows that the rock bolt pattern has a high influence on the load bearing capacity. It confirms the results from Nilsson (2003), which shows that when the rock bolts are placed at the depressions of the surface, in can decrease the load bearing capacity compared to the regular lining. It also confirms the results from Nilsson (2003) and Malmgren and Nordlund (2007) where it is shown that it is favourable to place the rock bolts at the peaks of the irregularities along the lining. This irregular model with the bolting pattern at the peaks of the irregularities is favourable compared to both the regular lining and the rock bolt pattern where the rock bolts are placed at the depressions of the irregularities.

Fig. 5.5 shows the difference in stress distribution around different bolts if the rock bolts are placed arbitrarily along the shotcrete lining. It is obvious that the rock bolts that are placed on the depressions of the shotcrete lining (in the middle of Fig. 5.5) give rise to higher maximum principal stresses than the rock bolts placed at the peaks of the shotcrete lining (shown in the left side of Fig. 5.5). This also confirms the conclusions put forward in Nilsson (2003) that the shotcrete around rock bolts placed at the peaks are dominated by compressive stress instead of tensile stress. For the rock bolts placed at the depressions of the shotcrete lining, the situation is reversed, and they are surrounded by high tensile stresses.

Fig. 5.5: Stress distribution for a shotcrete lining with rock bolts placed arbitrarily along the lining.
Fig. 5.6 shows the deflection in the lining ceiling for different shotcrete thicknesses. It is obvious that the regular shotcrete lining has a much higher flexibility compared to the irregular lining with any rock bolt pattern. This confirms the results from Nilsson (2003), which reaches the conclusion that the irregular shotcrete lining has a much higher stiffness than the regular lining. Nilsson (2003) also reaches the conclusion that the influence of the irregularity on the stiffness of the lining is increased when the thickness decreases, and this is also confirmed in Fig. 5.6.

The results also show that the rock bolt pattern with the rock bolts placed at the depressions results in a lining that has a higher stiffness than the rock bolt pattern with the rock bolts placed at the peaks of the irregularities. This effect also increases when the thickness of the shotcrete lining decreases.

![Deflections in the shotcrete lining ceiling](image)

**Fig. 5.6: Deflections in the shotcrete lining ceiling.**

### 5.2 Results from example 2

Example 2 involves a load distribution that is uniform along the whole shotcrete lining. This means that the pressure coefficient $K$ is 1. The maximum principal stress was situated at the rock bolts, which created a stress distribution similar to example 1. The maximum principal stresses at the bolt washer edges are shown in Fig. 5.7.
Fig. 5.7: Maximum principal stresses at the rock bolt when the load was uniformly distributed along the shotcrete lining.

![Graph showing stress vs. shotcrete thickness]

Fig. 5.8: Tensile stress in the lining ceiling.

![Graph showing stress vs. shotcrete thickness]

The results in Fig. 5.7 confirms the conclusions from Nilsson (2003), that the rock bolt pattern with the rock bolt at the peak is favourable. It also confirms that the rock bolting pattern with the rock bolts at the depressions of the irregularities can be damaging compared to the arbitrary placement on the regular lining, which is stated by Nilsson (2003) who also concludes that this
effect is larger when the shotcrete thickness increases, which is confirmed in Fig. 5.7 where the difference between the regular lining and the irregular lining with rock bolting pattern at the depression increases when the shotcrete thickness increases. The rock bolts that were subjected to the largest principal stress were situated at the lining wall, which creates a load that is similar to the slab Nilsson (2003) uses.

The tensile stresses at certain elements at the tunnel ceiling have also been collected. Fig. 5.8 shows the tensile stresses in the tunnel ceiling at approximately the same elements for the regular model, the irregular with rock bolting pattern at the depressions and the irregular model with the rock bolting pattern at the peaks.

The results show that the tensile stresses are lower for both the irregular models when the shotcrete thickness is small. This can probably be explained by the shape of the irregular lining, which is alternatively dominated by compressive and tensile stress when shape of the lining continuously alternated. This is shown in Fig. 5.9, where it is possible to see how the irregularities create an alternating stress distribution in each irregularity. This effect is probably most effective when the shotcrete thickness is small, something that is explained by Nilsson (2003) by the fact that the difference in moment of inertia between the regular shell and the irregular shell is largest when the shotcrete thickness is small.

Fig. 5.9: Stress distribution in the irregular shotcrete shell.
Fig. 5.10 shows the tensile stress in the lining wall. The difference between the regular lining and the irregular lining is much larger in the lining wall compared to the lining ceiling. This can probably be explained by the fact that the regular tunnel wall in the model simply was constructed as two flat slabs, which is similar to the model settings in Nilsson (2003), where a regular and an irregular slab were used. Since the tunnel ceiling than a natural arc, it is possible that the difference between the regular and irregular model was somewhat decreased in the tunnel ceiling. The rock bolting pattern at the peaks has lower tensile stresses when the shotcrete thickness is small, compared to the bolting pattern at the depressions, but the situation is changed when the thickness increases. Both the irregular models had much smaller tensile stresses than the regular model.

![Graph showing tensile stress vs. shotcrete thickness](image)

**Fig. 5.10: Tensile stress in the tunnel lining wall.**

The deflection of the lining ceiling and lining wall were also collected from the model. The deflections in the ceiling are shown in Fig. 5.11. This confirms the results from Nilsson (2003), that the irregular shotcrete lining is much stiffer compared to the regular lining. Fig. 4.9 also shows that the difference between the rock bolting pattern at the peaks and the rock bolting pattern at the depressions is quite small.

Fig. 5.12 shows the deflection of the lining wall. It is obvious that the differences in the deflection are much larger than in the ceiling. This could probably also be explained by the fact that the difference between the irregular shotcrete wall and the ceiling is higher due to the arch in the ceiling. The irregular wall will have a much higher stiffness than the regular wall, and this also decreases the tensile stress, as Fig 5.12 showed. As previously remarked, the walls are
similar to the shotcrete slabs used by Nilsson (2003), which means that the shows a strong similarity between them, both in a similar stiffness and load bearing capacity.

![Deflection in the lining ceiling](image1)

**Fig. 5.11: Deflection in the lining ceiling.**

![Deflection at the lining wall](image2)

**Fig. 5.12: Deflection at the lining wall.**
5.3 Results from example 3

In example 3 the horizontal pressure coefficient was changed to simulate different ground conditions. The pressure coefficient was changed between 0, 0.5, 1, 2, which corresponds to load distribution 1-4. The shotcrete thickness was 50 mm.

Fig. 5.13 shows the tensile stress in the ceiling. The tensile stress of the regular lining decreases as the horizontal pressure coefficient increases and the load on the tunnel walls increases. This is natural since the load is redistributed along the lining, towards the walls of the shotcrete lining instead of the ceiling, where the load is constant. For both the irregular linings the tensile stress is more or less the same when the pressure coefficient increases. This probably means that the irregular lining is less flexible and less able to redistribute load along the lining. Hoek and Brown (1980) observe that the irregular lining often has increased amounts of failures compares to parts of the tunnel where the lining is more regular. This could be an explanation, since the loads are not able to be redistributed in the same way as in the regular lining.

\[ \text{Fig. 5.13: Tensile stress in ceiling.} \]

The same seems to be true for the stress distribution at the lining wall, shown in Fig. 5.14. The irregular models have quite low tensile stresses at the lining wall, even as the pressure is increased along the lining wall. The regular model, in contrast, has increased lining wall tensile stresses, which increase almost in the same rate as the pressure coefficient, and also in a pattern that is nearly linear. The behaviour of the lining wall is similar to the results from Nilsson (2003), where the regular model had higher tensile stresses.
Fig. 5.14: Tensile stress in wall.

Fig. 5.15 shows the deflection in the lining wall. It is obvious that the regular lining has much higher deflection, and like stresses the increase almost linearly with the increased horizontal pressure coefficient. There is some difference between the irregular linings with different rock bolting patterns. It seems like the rock bolting pattern at the peaks have slightly higher deflection, something that also is shown in Fig. 5.4. The irregular lining walls have a much higher stiffness than the regular lining wall.

The difference between the deflection of the irregular models and the regular model is smaller for the deflection in the lining ceiling, shown in Fig. 5.16. The deflection in the ceiling of the regular model actually decreased when the horizontal pressure coefficient increases, which probably means that the lining is so flexible as to allow the redistribution of the stresses while the lining moves slightly when the stresses change. The irregular models both have small changes in the deflection of the tunnel ceiling, which probably means that the stiffness of these models a much higher compared to the regular model.
Fig. 5.15: Deflection in lining wall.

Fig. 5.16: Deflection in tunnel ceiling.
5.4 Results from example 4

Example 4 investigates the influence of a variation in the shotcrete thickness for the regular model and the irregular models with different rock bolting patterns. Three different variations in shotcrete thickness have been used, variation 1, 2, and 3, corresponding to Eqs. 4.1, 4.2 and 4.3, respectively. The maximum principal stresses along the edges of the rock bolt washer are shown in Fig. 5.17.

The results show that the irregular model with the rock bolting pattern at the peaks has the lowest maximum principal stress. The results are similar to the Fig. 5.4, where the difference between the irregular and regular models increased when the shotcrete thickness increases, which seems to be the case when the mean value of the shotcrete changes. It also seems like the mean value of the shotcrete thickness is more important than the distribution of the variation, and it is possible that the stresses can be redistributed along the shotcrete lining when the thickness changes.

Fig. 5.18 shows the stress in the lining ceiling. The stresses seem to converge when the shotcrete thickness increases and there is no big difference between the irregular models.

![Stress Graph](image)

*Fig. 5.17: Maximum principal stress in shotcrete lining.*
The stresses in the lining wall are similar to the results when the thickness is uniform. The regular model will have much higher lining wall stresses, which can be seen in Fig. 5.19. This can be compared to Fig. 5.10, which shows the tensile stress in the lining wall for different shotcrete thicknesses and have a similar pattern to Fig. 5.19. The rock bolting pattern with the rock bolts at the peaks is favourable compared to the rock bolting pattern with the rock bolts at the depressions. As previously stated, only the mean value of the shotcrete thickness seems to be of importance.

Fig. 5.20 shows the deflection in the lining ceiling. There seems not to be any difference between the irregular model with rock bolting pattern at the peaks and the other with rock bolting pattern at the depressions. Only the mean value of the thickness variation seems to matter. The regular model has much larger deflection.

Fig. 5.21 shows the deflection in the lining. It seems like the regular lining can be more affected by the thickness variation compared to the irregular linings. There is a possibility that the variation in thickness can be make the linings somewhat stiffer, and that this will be increased by the irregularity of the lining.
Fig. 5.19: Stress in lining wall.

5.20: Deflections in lining ceiling.
5.21: Deflection in lining wall.

5.5 Further discussion

Malmgren and Nordlund (2007) also state that it is favourable to place the rock bolts on the peaks of the shotcrete shell and not on the depressions, since the number of failures in the lining will be reduced (Fig. 2.14). This is confirmed in the model used in this report and in the model used in Nilsson.

But there is also a question of how the placement of the rock bolts affects the number of failures in the interface. According to Malmgren and Nordlund (2007), it will influence the number of failures in the interface (Fig 2.14), but it is unclear how big the effect will be. In the load distribution where the horizontal pressure was increased it was favourable to have the rock bolts at the peaks of the shotcrete shell. In the other load case where the horizontal pressure was decreased, it was favourable to the rock bolts at the depressions of the shotcrete shell. The failures in the interface did only account for a minor part of the total number of failures, which maybe indicates and there was no comparison to the case with a smooth shotcrete lining.

The model used by Malmgren and Nordlund (2007) also gives the result that the regular model will give increased compressive stresses compared to the smooth model (Figs. 2.3. and 2.7). Since there is no interaction with the surrounding rock in the model used in this report, it is difficult to reach any conclusions about this matter, but the result seems to indicate that the model with the rock bolts at the depressions has lower compressive stresses than either the smooth model or the model with the rock bolts at the peaks. The two latter models have roughly the same maximum compressive stresses. The visualizations seem to indicate that the irregular structure has varying compressive stresses in the tunnel ceiling (Fig. 5.22).
There is also a question of how the irregularity increases the stiffness in the Borio and Peila (2009) model and for the Son and Cording (2006) model. In Borio and Peila, when the thickness of the shotcrete lining was increased, the stresses at the edges of the model were increased. This could be related to the stiffness of the model. This will probably be true also for the case studied by Son and Cording (2006) as well (see Fig. 2.19), where the increased irregularity height-to-liner ratio is increased which will give higher stresses. The increased irregularity could increase the stiffness in both the model, but the authors have not measured this parameter.

5.6 **Connection between stiffness and stresses**

In rock mechanics, there is an intimate connection between the stresses at the rock surface and the deformations of the rock mass. The reason for this is that the rock surface stresses are reduced if some deformations are allowed. This is shown in Fig. 5.23 where the upper curve is the ground response curve, where the ordinate shows the stresses in the rock wall and the abscissa the deformations of the rock wall, or the movement of the rock wall. The other curve shows the response of the shotcrete when the deformations increase. Similar rock support response curves can be made for different types of rock support that behave differently. The shotcrete has a brittle failure while the rock bolts have a ductile failure, and the slopes of the rock support curves depend of the elasto-plastic behaviour of the material used in the rock support.

The main conclusion from the ground reaction curve is that a flexible rock support, e.g. a thin shotcrete lining with rock bolts, can be far more effective than a heavy but stiff rock support, e.g.
metal frames or concrete arches. The reason for this is that some deformations are allowed and the rock wall stresses will decrease, which means it will be easier to support the rock mass. This is one of the conclusions that resulted in the New Austrian tunnelling method, which involves light and flexible rock support instead of stiffer types of rock support, according to Hoek and Brown (1980), instead of heavier rock support with higher stiffness.

The results from this report and previous research demonstrate that the stiffness of the shotcrete lining is changed when the irregularity increases. This is shown both directly in Nilsson (2003) and indirectly in Malmgren and Nordlund (2007). The results from Nilsson (2003) and the results from this report (Fig. 5.4) show that the irregular shotcrete shell has much smaller deformations than the regular one. This is true for all thicknesses used in the models but the effect seems to be bigger for the thin shotcrete lining (Fig. 5.4) since the effect is more noticeable, especially in Nilsson (2003), where a slab is used to investigate the effect.

![Fig. 5.23: The ground response curve and rock support curve. From Oreste (2003).](image-url)

The results all show that the irregular shotcrete shell was stiffer than the regular one. Nilsson (2003) compared the irregular slab and the regular slab, and found that the irregular slab had a higher load bearing capacity. One problem with this result is that the model does not incorporate the deformation of the rock mass in the result. The model calculated the response of the different shotcrete shell for the same loads, but this might be unrealistic. The ground reaction curve
implies that the rock support with the biggest deformations might get a reduced load since the deformations will reduce the load that the lining is subjected to. This might be a problem in the model that could be adjusted if the rock mass is incorporated in future numerical models.

But both the stiffness of the lining and the surrounding rock are important. Son and Cording (2006) use the flexibility ratio to measure how stiff the interaction between these two parameters are. One the conclusions in Son and Cording (2006) is that is the shotcrete lining becomes flexibly compared to the ground; the irregularity will not a very big effect on the lining stresses and that the flexibility ratio is very dependent on the rock mass rating, or overall quality of the rock, and that stresses will increase when the rock quality decreases.

One conclusion from the results in this report could be that the irregularity of the rock surface makes the resulting shotcrete stiffer, which could increase the stresses in the lining and induce damages. This is just a hypothesis, but it the restrained deformations a big enough, the stresses could probably be big enough to make quite a difference for the stress distribution of the shotcrete lining. As has been noted above, one of the problems with the calculations have been that the force applied onto the shotcrete lining has not been related to the deformations, and that this could maybe be done to show a more realistic picture of the stresses in the shotcrete lining. It would probably also be possible to estimate the effects of the properties of the surrounding rock, since this also influences the flexibility ratio and the stress distribution of the lining and the stresses in the rock surface. Malmgren and Nordlund (2007) uses the rock-shotcrete connection, and their results shows among other things that the stresses at the rock-shotcrete interface might be important, since it corresponds to the failures in the lining as well.

The results also showed that the regular lining had a significantly increased flexibility in redistributing the stresses along the lining. Figs. 5.14 and 5.15 show that the irregular lining has a much slower response, and Figs. 5.16 and 5.17 show that this is also true for the deformations of the lining. This could probably have a significant impact on the lining since the loads could probably be redistributed and reduced if deformations happen at certain parts of the lining. The irregular lining has a much higher stiffness and could probably not utilize the influence of this effect significantly compared to the regular lining.

5.7 Usefulness for the design practice

The numerical models used in this report and the scientific papers reviewed in chapter 2 have shown that the irregularity clearly has an impact on the performance of the shotcrete lining. But the rock surface is never as systematic in its irregularity as the models. A real rock surface is probably much more irregular, with many different kinds of irregularities. Considering both that the rock surface is probably quite irregular and the organizational problems that exists, even if the rock bolts were placed at the best possible places, it would probably only increase the load bearing capacity a couple of percent. It is probably more important to give guidelines to ascertain
that the rock bolts are not placed at the wrong places, i.e. the depressions of the irregular rock surface, where the results from this report and Nilsson (2003) have shown that hey the reduce the load bearing capacity.

The best way to do this would probably be to give detailed instructions not to place the rock bolts at the depressions of the shotcrete shell, while trying as much as possible to place the rock bolts at the peaks. Since the structure is so irregular, it will probably be difficult to create the kind of pressure distributions around the rock bolts that are placed at the peaks of the shotcrete structure that Nilsson (2003) discusses, as which has been discussed in this report. But if the rock bolts are not placed at the depressions, the load bearing capacity would probably be better. Figs. 5.24 and 5.25 show how the rock structure can look in-situ and the photographs also indicate the practical difficulties in finding the best positions on the shotcrete shell to place the rock bolts.

Fig. 5.24: Peaks of the rock surface that shows the difficulties in placing the rock bolts at the structurally most favourable positions of the structure. Photograph by Johan Spross.
Presently, there are often no design calculations done in regular design, according to Frank Ouchterlony (2009), and empirical models are usually used to decide the shotcrete thickness and bolt distance. Often several empirical models are used to complement each other and minimize risk, according to Frank Ouchterlony (2009). Since the experience from earlier projects is wide, the result will probably always be on the safe side. The leads to two conclusions: there is room for improvements to make a new design process that is more realistic, and might also save costs if the rock support can be more properly designed to resist the loads.

But it probably also indicates the difficulties in establishing a useful design procedure based on calculations of the load effects and rock support. It is very difficult and expensive to measure the parameters that the design will be based on, according to Frank Ouchterlony (2009). Both the adhesion strength and the quality of the shotcrete layers are parameters that are difficult to control and may show strong local variation, according to Nord and Stille (1990). The adhesion strength depends on the quality of the rock surface, which depends on how clean and dry the surface is. As we have seen, practical problems can influence the properties of these parameters, according to Frank Ouchterlony (2009). The quality of the shotcrete surface is also dependant on the skill of the workman who handles the machine, and this can change from person to person or from day to day. The working conditions can also be difficult, especially when several machines need access.

Fig. 5.25: Shotcrete surface after it has been sprayed with shotcrete. Note the difficulty in finding the most favourable positions for the rock bolts on the surface, photograph by Johan Spross.
The number of failures in the lining and in the interface seems to be connected, according to Malmgren and Nordlund (2007). The ratio of lining failures and interface failures seems to be related to the irregularity. The shotcrete thickness has a large impact as well, where an increase in the shotcrete thickness can reduce the number of failures in the lining, but can increase the number of interface failures. It may be possible to optimize the number of failures if the right thickness and bolt distance is selected for a specific irregularity. Such an optimization could probably be very useful.

5.8 Suggestions for further research

If the model used by Nilsson (2003) and this report is going to be used again, it would be very good if the interface strength could be included. As Malmgren and Nordlund (2007) shows, this is an important parameter and the number of failures in the lining is probably dependant on the interface strength. The connection between the interface failures and the lining failures would also be interest to investigate.

The model could also be improved if the effect of the stress distribution in the ground could be included. It would be preferable if the variation of the pressure coefficient, $K$, could be included. This has been modelled both by Borio and Peila (2009) and Son and Cording (2007). The results from this report shows that the regular lining has a higher ability to redistribute the stresses that arise in the lining compared to an irregular lining, and this might impact the behaviour of the shotcrete lining significantly when the stresses increase. Since the Swedish rock mass is dominated by horizontal pressures, according to Nord and Stille (1990), who state that this typically are bigger than the vertical pressure, it would be interesting to simulate the Swedish conditions.

If information about the real geometry before and after the rock opening has been sprayed with shotcrete was available, a more realistic model could be used. This could be achieved by using data from scanning in tunnels, according to Dan Palmgren (2009). Such information also describes the variation in the shotcrete thickness, which could also be of interest for an analysis of the impact of the variation. The results from this report shows that only the mean value of the shotcrete thickness seems to be of interest since the stresses can be redistributed to other parts of the lining. This could be further investigated and could be of practical importance, since if this result could be confirmed and compared to practical results, it could indicate that the time allotted for the shotcrete spraying phase could be reduced, would mean that the speed of tunnel construction would increase, while cost would probably be reduced somewhat.

It would be interesting to see if the empirical models that are used today could be combined with calculations of the influence of irregularity of the shotcrete shell, or at least by some more detailed measurement to assess the how the empirical models work in reality. This would
increase the knowledge of the possible improvement that could be made with a more accurate design process.

It could also be useful to model different ground conditions. Malmgren and Nordlund (2007) use very hard rock while the models by Son and Cording (2006) and Borio and Peila (2009) are based on soft ground conditions. This will probably influence the stress distribution in the shotcrete shell significantly. The flexibility ratio, which Son and Cording (2006) used as a parameter, is also influenced by the ground conditions. Soft ground would probably decrease the flexibility ratio, according to Eq. (2.4), because the Young’s modulus decreases when the rock mass is soft, according to Malmgren and Nordlund (2007).

As previously noted, it would be interesting to use the deformation of the rock surface in the model, since this probably is a very important parameter that is related to the stress of the surrounding ground. This requires that the interaction between the rock mass and the shotcrete is included in the model.

It would also be valuable to use sharper edges in a model, to study the influence of this parameter using more realistic information. Both Nilsson (2003) and this report uses smooth edges modelled by sinus-curves, but both Malmgren and Nordlund (2007), Borio and Peila (2009), and Son and Cording (2006) uses rough edges, that probable have an impact in how the stress will be distributed along the lining. The results from Borio and Peila (2009) suggest that the highest tensile stresses occur at the peaks of the irregularities of the rock wall. Fig. 1.1 also suggests that the real irregularities are quite sharp.

The stiffness of the lining seems to be of interests for the structural behaviour of the shotcrete shell. This is shown both in Borio and Peila (2009) and Son and Cording (2006), where the stiffness of the lining is used as a parameter both directly and indirectly. The stiffness of the lining could also be of interest if the flexibility ratio that is defined in Son and Cording (2006), Eq. (2.4). This could also be of practical use, since Eq. (2.4), with certain changes, could be used to measure the influence of the stiffness of both the ground and of the shotcrete lining, and the interaction between these. If typical values of these parameters are knows, it could give a basic knowledge of how the lining will be affected by the load. This information could probably be useful in construction. An approximation of the influence of this parameter is probably presently used in some sort of empirical system, but a formalization of the information could probably also be useful. It could be of interest to measure the deformations all over the model and especially to measure the influence these deformations have.
6. Sources

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