CHALMERS

Wire cutting as a complement to drill and blast in vibration sensitive environments

Master of Science Thesis in the Master's Programme Geo & Water Engineering

NILS GUSTAFSSON

Department of Civil and Environmental Engineering
Division of GeoEngineering
Engineering Geology
CHALMERS UNIVERSITY OF TECHNOLOGY
Göteborg, Sweden 2010

Master's Thesis 2011:111

MASTER'S THESIS 2011:111

Wire cutting as a complement to drill and blast in vibration sensitive environments

Master of Science Thesis in the Master's Programme Geo & Water Engineering

NILS GUSTAFSSON

Department of Civil and Environmental Engineering
Division of GeoEngineering
Engineering Geology
CHALMERS UNIVERSITY OF TECHNOLOGY

Göteborg, Sweden 2010

Wire cutting as a complement to drill and blast in vibration sensitive environments
Master of Science Thesis in the Master's Programme Geo & Water Engineering
NILS GUSTAFSSON

Examensarbete/Institutionen för bygg- och miljöteknik,
Chalmers tekniska högskola 2011:111

Department of Civil and Environmental Engineering
Division of GeoEngineering
Engineering Geology
Chalmers University of Technology
SE-412 96 Göteborg
Sweden
Telephone: + 46 (0)31-772 1000

Cover:
A wire cut tunnel profile at Henriksdals wastewater treatment plant. Photo: author

Name of the printers / Department of Civil and Environmental Engineering
Göteborg, Sweden 2010

Wire cutting as a complement to drill and blast in vibration sensitive environments
Master of Science Thesis in the Master's Programme Geo & Water Engineering
NILS GUSTAFSSON
Department of Civil and Environmental Engineering
GeoEngineering
Engineering Geology
Chalmers University of Technology

ABSTRACT

In vibration sensitive environments traditional excavation methods such as the Austrian method (drill and blast) or tunnelling with a TBM (tunnel boring machine) may not be suitable. The aim of this thesis is to investigate what differences might be if excavation is done with wire cutting instead. The differences will be viewed from following perspectives: geometry and stress situation, economy and time, and effects on the surroundings with focus on vibrations. The different tunnel profiles and the resulting stress properties have been examined in computer simulations to visualize the differences. Because of the limitation in tunnel profiles a wire cut profile will have a more rectangular shape which may have unfavourable stress properties. It will result in stress concentrations and tensile stresses which implies for more reinforcements to secure the loose rock mass. To see the economic differences of the methods two blasted tunnels with different vibration regulations have been examined and compared with wire cutting. Data from two blasted tunnels with different vibration regulations have been used to calculate a price for one excavated cubic meter of rock. The comparison shows that wire cutting is roughly twice as expensive as drill and blast for the tunnel with high vibration regulations. The difference is greater for the tunnel with less vibration regulations. The practical aspects of the different techniques are hard to estimate in monetary values. Because of its higher price wire cutting has only been used for special applications or when vibration regulations make blasting impossible. Wire cutting can be performed close to very vibration sensitive constructions or in the vicinity of people but to a higher cost than conventional drill and blast method.

Key words: drill and blast, vibrations, wire cut, tunnel geometry, stress orientation, excavation cost

Linsågning som komplement till sprängning i vibrationskänsliga miljöer
Examensarbete inom Geo & Water Engineering
NILS GUSTAFSSON
Institutionen för bygg- och miljöteknik
Avdelningen för geologi och geoteknik
Teknisk geologi
Chalmers tekniska högskola

SAMMANFATTNING

I vibrationskänsliga miljöer lämpar sig ibland inte traditionella tunneldrivningsmetoder som sprängning eller en tunnelborrmaskin. Syftet med detta examensarbete är att utreda skillnaderna om berguttaget sker med linsågning istället. Skillnaderna kommer att betraktas ur följande perspektiv: geometri- och spänningssituation, ekonomi och tid och omgivningspåverkan, främst vibrationer. De olika tunnelprofilerna och de resulterande spänningssituationerna har undersökts i ett simuleringsprogram för att visualisera skillnaderna. I och med begränsningarna med utförandet kan en linsågad profil ha en något mer rektangulär form som kan vara utsatt för en mer negativ spänningssituation. Det kommer att resultera i spänningskoncentrationer och dragspänningar vilket innebär mer förstärkningsåtgärder för att stabilisera lösa stenblock. För att titta på de ekonomiska skillnaderna mellan metoderna har två sprängda tunnlar med olika vibrationskrav betraktats för att sedan jämföras med linsågning. Data från två sprängda tunnlar med olika vibrationskrav har använts för att räkna ut ett pris på en kubikmeter uttaget berg. Jämförelsen visar att linsågningen kostar ungefär dubbelt så mycket som sprängning för tunneln med stränga vibrationskrav. Skillnaden är ännu större för tunneln med generösare vibrationskrav. De praktiska aspekterna är svåra att uppskatta i kronor. På grund av sitt höga pris har linsågning bara använts i speciella situationer eller när höga vibrationskrav omöjliggör sprängning. Linsågning kan utföras nära väldigt känsliga anläggningar och i närheten av människor men till en kostnad som är högre än konventionell sprängning.

Nyckelord: sprängning, vibrationer, linsågning, tunnelprofil, spänningssituation, kostnad för berguttag

Contents

ABSTRACT I

SAMMANFATTNING II

CONTENTS III

PREFACE VI

1 INTRODUCTION 1

1.1 Background 1

1.2 Aim and limitation 1

1.3 Scope of work 2

2 EXCAVATION METHODS 3

2.1 Blasting 3
2.1.1 The blasting plan 3
2.1.2 Components 4
2.1.3 Ignition 4
2.1.4 The wedge 5

2.2 Tunnel boring machine 5
2.2.1 Procedure 6
2.2.2 Properties of a TBM-excavated tunnel 6

2.3 Vibrations 7

2.4 Careful blasting 8

3 WIRE CUTTING 10

3.1 Procedure 10
3.1.1 Shafts 12
3.1.2 Cuts 14
3.1.3 Tunnel profiles 16

4 ROCK MECHANICAL ASPECTS 18

4.1 Background 18

4.2 Wedge failure 18

4.3 Arching 19

4.4 Rock overburden 20

4.5 Rock classification system 20
4.5.1 RMR 21
4.5.2 Q-system 21

5 NUMERICAL MODELLING OF ROCK STRESSES REGARDING DIFFERENT TUNNEL PROFILES 22

5.1 Profile simulation 22
5.1.1 Loose zone 23

5.2 Results 23
5.2.1 Rectangular shape 23
5.2.2 Rounded roof shape 24
5.2.3 Circular shape 25

6 ROCK REINFORCEMENT IN A WIRE CUTTING PERSPECTIVE 27

6.1 Rock bolts 27
6.1.1 Normal bolt pattern 28
6.1.2 Selective bolting 29

6.2 Shotcrete 29
6.2.1 Adhesitivity 30

6.3 Steel arches 30

6.4 Concrete arches 30
6.4.1 Shotcrete arches 31
6.4.2 Site casted concrete arches 31

7 ECONOMICAL COMPARISON FOR ROCK EXCAVATION OF TWO TUNNELS WITH DIFFERENT CONDITIONS 32

7.1 Normal blasting 32

7.2 Careful blasting 33

7.3 Result 33

8 DISCUSSION 35

9 CONCLUSIONS 36

10 REFERENCES 37

11 APPENDICES 40

Preface

This master thesis is the final part of my Master of Science education at Chalmers University of Technology. The thesis is written at the Department of Civil and Environmental Engineering and division of GeoEngineering in cooperation with NCC Teknik in Stockholm during the spring of 2010. I would like to thank the following persons for their contribution and help with my thesis:

Lars O Ericsson, my examiner at Chalmers, for guidance and help.

Carl Johan Gårdinger, my supervisor and mentor at NCC Teknik, for guidance, knowledge, advice, motivation and everything in between.

The staff at NCC Teknik for making me feel welcome and for all the help.

Bengt Ludvig – Petro Team Engineering
Carl Lind – Nitro Consult
Cecilia Montelius - NCC
Gunnar Ehrenfeldt - Trafikverket
Lars-Olof Dahlström - NCC
Lars-Åke Öhlén - NCC
Monika Sammelin - NCC
Nils Outters - NCC
Rolf Christiansson - SKB
Sid Pathel - NCC
Thomas Dalmalm - Trafikverket
Thomas Krekula - NCC
Urban Nyberg - DWTeknikk

Stockholm June 2010-06-11

Nils Gustafsson

1 Introduction

1.1 Background

In today's society there is a growing need for infrastructure. Due to urbanization more and more land is being used for structural development and infrastructure is located underground instead. In many cities the underground environment consists of subway tunnels, highway tunnels, railroad tunnels, water tunnels, access tunnels, and other types of infrastructure tunnels. When building new tunnels it is probable that construction will take place close to already existing underground constructions. This composes a problem when traditional drill and blast methods might cause damage to the existing tunnel.

The most commonly used method for excavating rock mass is drilling and blasting. It is a method that is both time- and cost effective. However, the method induces vibrations that are spread in the ground and can damage structures in its surroundings. When blasting is executed underground it is often performed with a smaller amount of explosives to decrease the vibrations but to the cost of reduced efficiency.

Wire cutting is a method that traditionally has been used in quarries but not so much in infrastructure projects. The technique can be used for shafts, advancements of tunnels and for cuts. The wire wears through the rock and is therefore a vibration free and quiet method. The surface after cutting is smooth which can be beneficial for some applications. The method has so far only been used in special applications or when the surroundings made blasting impossible.

Compared to drill and blast, the wire cutting method can result in a different geometry and therefore a different stress situation. This may lead to other reinforcement methods compared to the drill and blast technique.

1.2 Aim and limitation

The general aim of this thesis is to evaluate wire cutting as a rock extraction method from several perspectives. The thesis will also pay attention to some practical aspects that may vary from project to project.

The aim is to see what the consequences will be when using wire cutting instead of more traditional methods of excavating rock mass in vibration sensitive environments.

The consequences of the sometimes limited geometry when using the wire cutting technique will be examined to see how the stress situation change and what this means for the reinforcement needs.

Blasting journals from two tunnels with different vibration regulations are examined and compared with wire cutting to give an economic perspective over the two techniques. The results will be evaluated to see when a technique is economically preferable

The procedures of the methods will be examined and also the practical aspects that arises from the execution.

Focus will be on wire cutting when used for shafts, cuts and tunnels and compared with drilling and blasting, used in environments with vibration regulations.

1.3 Scope of work

The thesis describes different underground extraction methods, to put in comparison with wire cutting. The basics of rock mechanics and rock reinforcements are presented to show differences between wire cutting and other techniques.

The theory and procedure of wire cutting is described with information derived from interviews with property developers, consultants and entrepreneurs.

Computer simulations are used to show how different geometries affect the stress situation for excavations and what this means for the reinforcement needs and bearing capacity.

Blasting journals from two tunnels are used to make an economical comparison between blasting and wire cutting.

2 Excavation methods

In today's infrastructure projects a common way to excavate the rock mass is by drilling and blasting but for some tunnels a tunnel boring machine is used. With drill and blast it is possible to create geometries that are impossible with a TBM. The two methods differ greatly in rock excavation technique.

2.1 Blasting

In the field of civil engineering, blasting has gone from a manual occupation where personal skill and great experience was a key factor to a technical science with many applications and sophisticated methods. Today's blasting is performed in environments that were not suitable for blasting not too long ago. The result is however still dependent on the blasters competence and experience.

Blasting is the desirable technique for underground excavation because of its high efficiency and fairly cheap execution. (Langefors 1963).

2.1.1 The blasting plan

The procedure for a blast is:

- Drilling
- Charging
- Blasting
- Ventilation
- Scaling
- Loading and transport

2.1.1.1 Drilling

Drilling is carried out after a predetermined plan which is adjusted for criteria such as vibration limits, geology and joints.

Tunnels being built today near existing infrastructure have often vibration restrictions for avoiding potential damages. Under such circumstances careful blasting is carried out during the construction of the tunnel.

The vibration level is a function of the cooperative charge, the distance to the receiver and site specific constants, sees equation 2.1. Thus, in order to reduce vibrations the cooperative charge has to be reduced. This can be achieved by reducing the amount of holes with the same interval number, drilling finer holes or drilling shorter holes or a combination of these. (Ambraseys & Hendron, 1968).

$$v = \alpha \times \left(\frac{\sqrt{Q}}{r}\right)^{\beta} \quad (2.1)$$

Where;

v= vibration level [mm/sec]

Q= cooperative charge [kg]

r= distance to the vibration receiver [m]

α & β= site specific constants

When the drilling plan is decided the holes are loaded with the chosen explosives. The holes are also fitted with primer and ignition cap. (Olofsson 1999).

2.1.1.2 Detonation

At the detonation point the chemical energy in the explosives is released and affects the rock in three stages.

During the compression stage a shock-wave is spread in rock with a speed of 3000-6000m/s, depending on the geology. The shock-wave crushes the adjacent rock and creates micro fractures. After the compression stage the shock-wave is reflected against free surfaces and creates tensile- and shear stresses. In this stage most of the breakage to the rock is made. In the last stage the blasting fume penetrates the newly created micro fractures and the already existing fractures. Under high pressure the gas expand the fractures and the rock cracks up and hurls against an open surface. The blasting procedure is very rapid and is considered complete when the borehole volume has expanded ten times, which takes about 5ms. (Stål & Wedel 1984).

2.1.1.3 Ventilation

After the detonation the gas from the explosion needs to be ventilated. The gas is toxic and consists of CO_2, nitrogen, water vapour, CO, NO_X and nitro-glycerine gases. The composition of the gas is dependent on the type of explosives. Emulsion explosives usually consist of less harmful gases. (Olofsson 1999).

2.1.1.4 Scaling

After the blast there can be loose rocks in the walls and in the roof that need to be removed to ensure the safety of the workers. The loose rock is removed by an iron bar, first by a machine then by hand. The volume of the scaled rocks depends on the EDZ, Excavation Damage Zone, which is related to the cooperative charge, a more powerful detonation requires more scaling.

After scaling is done the stones are loaded, commonly with a wheel-loader to a truck for transport to a crusher.

After the removal of the rock drilling begins for the next cycle. (Olofsson 1999).

2.1.2 Components

The basic principle of a successful explosion is an explosive material and something that can detonate it. The detonation should occur under a controlled situation.

The explosives most commonly used for tunnel excavation are cartridge explosives and emulsion (SSE, site sensitized emulsion). Cartridge explosives are used for careful blasting and are nitro-glycerine based. Since the explosive is packed as a cartridge the weight is more exact than bulk based emulsion. The most common cartridge explosive is dynamite.

When the vibration limits are less strict emulsion is used. It is a mixture and is sensitized when it’s being pumped into the holes. It is hard to dose in small quantities and is therefore not used in sensitive areas. (Olofsson 1999).

2.1.3 Ignition

The ignition system ignites an ignition cap that will start the detonation. The idea with an ignition system is to ignite the caps with different time intervals so the cooperative charge will not be too great.

There are three different commonly used ignition systems; non electric, electric and electronic.

The non electric system is best suited in environments exposed to electric disturbances such as power lines or thunder. The ignition system has a time delay element for individual ignitions. The time delays ranges from 25 to 6000 ms in 32 intervals to ensure advanced ignition plans. (BergUtbildarna AB 2009).

The most used electric ignition system for tunnel blasting is the 0,5sec delay ignition cap. The short interval blasting system was a huge development for blasting when it was first introduced. Due to more electrified activities in tunnel blasting the electric system has been more and more replaced by the non electric, NONEL, system.

In the electronic ignition cap the pyrotechnic delay element is replaced with a programmed electronic time control. This means that the system is very flexible with delay times ranging from 1 to 16000ms. With the electronic ignition caps it is easier to create an ignition plan where each ignition cap gets an individual delay time. In this way the chance of multiple ignition caps to detonate at the same time are small. (Olofsson 1999).

With electronic ignition caps there is a smaller risk of exceeding the vibration levels but the caps costs four times more than the regular ignition caps. (Stiftelsen bergteknisk forskning 2010 (a)).

Some explosives cannot be ignited with just an ignition cap and so they have to be fitted with a primer. A primer is an explosive that can be ignited with an ignition cap. The primer is usually a bit of dynamite or pentyl and comes in a cartridge packing. Primer is always used when emulsion explosives are used. (Olofsson 1999).

2.1.4 The wedge

The greatest difference between bench blasting and tunnel blasting is the number of open surfaces. A tunnel front has only one free surface. It is therefore crucial to create an opening for the rock to breach against. The constricted tunnel bedrock needs more blasting force to break than what is the case with bench blasting.

The first step when blasting a tunnel is creating a wedge. The wedge consists of a number of drilled holes in a coarse dimension in a special pattern, often situated in the lower part of the tunnel front. The wedge is blasted against the drilled holes, and then the stope is blasted against the wedge. The last part to be blasted is the contour of the tunnel. (Olofsson 1999).

2.2 Tunnel boring machine

A tunnel boring machine, TBM, is used to excavate tunnels with a constant circular cross section. The breakthrough for the technique came in the 1950's when the rotating head was invented. TBM's are often used for long sewer tunnels with great flows or for hydropower constructions. The most common diameter is 6-7m but there are models ranging from 2 up to 10m, see Figure 1.

Figure 1. A tunnel boring machine and the cutter head with the circular cutters.

TBM-tunnels are suitable for long tunnels with a constant cross section and with moderate curve radius.

2.2.1 Procedure

The TBM crushes the rock mass with mounted disc cutters on a rotating head. The rotating head has the same diameter as the final tunnel profile. The machine presses a gripper system against the sides of the tunnel to hold the machine in place while the rotating head is pushed against the rock. After 1-2m the machine releases the gripper system and retracts the rotating head. The machines moves forward and the gripper system are pushed to the side of tunnel and the rotating head is forced against the rock again.

The excavated rock is transported on conveyor belts for removal.

A TBM can reinforce the rock mass in two ways; it can be fitted with equipment for grouting and installing rock bolts by drilling holes in the rock mass, or by installing concrete- or steel lining in the tunnel. Traditional reinforcement with bolts and shotcrete is time consuming.

The technique of grinding the rock creates less damage to the rock mass which can mean less reinforcement.

2.2.2 Properties of a TBM-excavated tunnel

The technique of using TBM is originally developed for homogenous and softer rock types with low compression strength and small groundwater intrusion.

The technique is not optimal for typical Swedish conditions with its hard rock mass. The hard rock mass means low advancement speed and the cutters get worn down quickly. The grouting and reinforcements are also time consuming.

However, the technique has been used in Sweden in few projects. The most famous one is the Hallandsås tunnel. Two tunnels in Stockholm used for wastewater has been drilled with a TBM. The length and the use of the tunnels made the big investment of the rig profitable. (Sandström 2010).

The rock surface after drilling is smooth and requires low supplementary work before usage. The smooth sides also contribute to low turbulence which is beneficial from a flow perspective. The lack of detonations gives small vibrations and the technique is therefore beneficial in vibration sensitive areas. However, the technique gives rise to structure-born sounds. (Hapgood 2004).

Because of it circular geometry a TBM tunnel is beneficial from stress situation point of view. The lack of sharp corners means that there are no stress concentrations. However, a zone with tensile stress will occur with a height of approximately half the tunnel radius, according to Dahlström[1].

It is hard to estimate costs for a bored tunnel because for every tunnel a new TBM is built or custom rebuilt. Two bored tunnels with the same cross section area and at same depth can vary ten times in cost because of different ground conditions in mechanical and hydrological respect. It is therefore impossible to estimate the rate of advance and cost per meter for a TBM tunnel without extensive investigations. (Baumann 1993). This extreme difference in price is also applicable for other excavation methods.

2.3 Vibrations

Ground vibrations are seismic movements which can be induced by blasting. Often when excavating underground there are vibration regulations to ensure the safety of surrounding people and constructions.

A vibration is an energy transfer which is spread in the surrounding rock. The vibrations from blasting depend on the cooperative charge, how the rock is constricted, the rock characteristics, the distance from the blasting site and the soil characteristics.

By choosing the right blasting method and the right blasting plan the vibration from the blasting can be controlled. (Olofsson 1999).

When the vibration, or more correctly the energy transfer, reaches a building or a construction it can cause damages if the energy reaches a critical level. How sensitive a building is for vibrations depends on the house status, its foundation and the magnitude and duration of the vibrations.

In many cases the vibration limits are determined by electronic installations which are very sensitive to vibrations.

The maximum allowed vibrations levels are determined by how sensitive surrounding buildings are, electronic installations and/or if there are people in the vicinity. The vibration limits thereby determine how powerful the blast can be, according to Lind[2].

The vibration levels for constructions are measured in velocity, mm/sec and for installations in acceleration, mm/sec^2.

Before blasting can begin an inspection of surrounding constructions and installation are made to estimate the vibration level they can withstand without damage.

Because the vibration is distance dependant, a low vibration level in combination with a small distance to the source of the vibration, the cooperative charge of the detonation must be very small. At 0,5mm/sec most people experience the vibration as clearly detectable and the Stockholm subway system allows vibrations up to 10mm/sec without any specific control for blasting. (Olofsson 1999).

---

[1] Lars-Olof Dahlström, professor, NCC Teknik. Interview 2010-04-09.

[2] Carl Lind, civil engineer, Nitro Consult. Interview 2010-03-12.

2.4 Careful blasting

When blasting is restricted in some way, usually with smaller charges and/or shorter blast length it is called careful blasting. Almost all blasting in populated areas is restricted.

With the development of blasting technique, explosives and refined calculation methods it is now possible to perform blasting where it previously had been impossible due to vibration sensitive surroundings.

Vibrations are partly dependent on ground properties which make each site specific. A test blast should be carried out to determine local conditions and to recommend vibration levels. A lowered vibration level increases the cost for drilling and blasting significantly.

The increased costs is due to increased drilling, higher charging costs, increased number of blasts, cost for vibration measurements, higher costs for planning, inspection and governmental relations and insurance costs.

A maximum vibration level of 18mm/sec can tenfold the cost compared to a non-regulated blast. (Fomenko & Rudegran 2008).

In situations with vibration limits the cooperative charge can be as small as 165g. For this small charge the length of the bore holes, the c/c-distance for the bore holes and the diameter of the boreholes are very fine. The diameter of the hole is as slim that is possible for the machine. Smaller charges will not break loose the rock.

With these small volumes the system with emulsion explosives cannot be used. The specific volume needs to be exact to avoid a too powerful detonation and the SSE system is to inaccurate due to pumps and flows when starting and stopping. SSE is only suitable for volumes of 2kg/hole or more. Under these extreme situations with small charges cartridge explosions are used, usually dynamite.

With no vibration regulation about 1,5-2 drillmeter/m^3 is required to break the rock. For careful blasting it takes about 10 drillmeter/m^3 rock. But these ten drilled meters are of finer diameter and in a tighter pattern. Because of more drill-holes per cubic meter rock the amount of explosives can be divided up to more detonations with less cooperative charge, leading to decreased vibrations, according to Öhlen[3].

To ensure that the detonation does not exceed the vibration level it is vital that only one charge detonate at the time. If the levels have been exceeded a probable cause can be that two charges went off at the same time. By examining the vibration data from a geophone it is possible to determine which two charges detonated. When the charges have been identified the detonation plan can be changed to avoid simultaneous detonation, according to Lind[4].

The most common way to reduce vibration is changing the blasting plan. The easiest thing is to change the detonation plan to ensure that no charges detonate simultaneously.

If there is still a risk of exceeding vibration levels the length of the bore hole can be reduced. If this doesn't help the dimensions and the c/c-distance can be changed.

[3] Lars-Åke Öhlen, site manager NCC Construction. Norrströmstunneln, Stockholm City Line. Interview 2010-03-15.

[4] Carl Lind, civil engineer, Nitro Consult. Interview 2010-03-12.

A way of excavating a tunnel is the pilot and stope method. As seen in figure 2 the tunnel profile is divided into sections. First the pilot is blasted. It admits an inspection of the rock to see potential loose wedges. A pilot can also make the rock less constricted which make the stope easier to excavate and requires fewer explosives. The pilot and stope method is used for larger tunnel areas, according to Gårdinger[5].

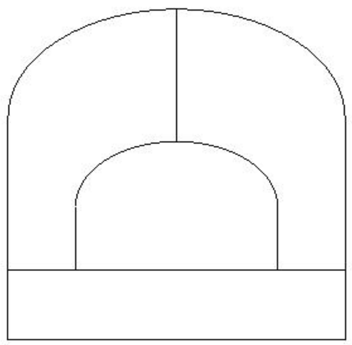

Figure 2. A tunnel profile divided into pilot, two stopes and the bench.

It is possible to blast very close to sensitive constructions but to achieve very low vibration levels the cooperative charge needs to be very low. It means that the diameter of the hole must be very small and the pattern very tight. Drilling that kind of narrow hole is difficult and usually done by hand. It takes a long time and so it becomes expensive. Because of the small cooperative charge only a small volume of rock is released at the detonation. It means that the drill and blast cycle; drilling, charging, blasting, ventilation, scaling and transport take place often which is time consuming.

[5] Carl Johan Gårdinger, civil engineer, NCC Teknik. Interview 2010-04-12.

3 Wire cutting

Wire cutting has been used for a long time to cut or saw through the rock. The greatest use has been in quarries for extracting marble stone. The technique has been used because of its careful removal and the ability to obtain desired shape, see figure 3.

It has become more popular in infrastructure project because of the development of the wire. The industrial diamonds that are mounted on the wire are now sufficiently strong to cut through the hard granite rock.

The greatest benefit of wire cutting is the vibration free extraction of rock. Because of its vibration free technique the method has mostly been used in special cases with strong vibration regulations, such as extraction near sensitive installations or other situations when blasting is not possible. The excavation method leaves a very smooth surface which can be useful in some applications and because of the vibration free technique the EDZ, excavation damaged zone, is minimal.

Figure 3. The clean and smooth surface of the wire cut profile compared with the rougher surface of the drill and blasted one.

The technique can also be used when blasting is carried out close to a sensitive object. The rock mass that is to be excavated is disengaged by wire cutting to prevent the vibrations from the blasting to spread to the surroundings.

The merciful technique is based on abrasion, meaning that the rock is worn down smoothly and continuous, according to Nyberg[6].

3.1 Procedure

The idea of wire cutting is to wear down the rock and create a cut. With one or several cuts a part of the rock is exposed and can be removed as a homogenous piece.

There are three types of wire cuts; shafts, cuts and tunnel profiles and each type have different execution methods.

[6] Urban Nyberg, general manager, DWTeknikk. Interview 2010-04-28.

Depending on the accessibility there are two main principles of cutting the rock mass. The simple method is pulling the wire round the rock mass threw drilled holes. If this is not possible the wire can, by using a rig with bars and pulleys, be pushed against the rock mass.

When cutting the rock mass with the first method drilled holes are made so the wire can be pulled around the wedge that is supposed to be removed. Then the ends of the wire are connected together and mounted to a motor. The motor pulls the wire round which wears the rock down. To avoid a slack on the wire the motor is mounted on rails and is pulled back to keep the wire tight. The normal tensile stress in the wire is 200 kg. Here lies the limitation in the efficiency, if the wire was capable of higher tensile stress the efficiency would increase, according to Nyberg[7].

After four to six hours of cutting the cases surrounding the wire are worn down and need replacement. If the cases are worn down unevenly cutting will become irregular and ineffective. The cases on the wire are shown in figure 4. The bearings in the pulleys are also changed regularly to avoid failure.

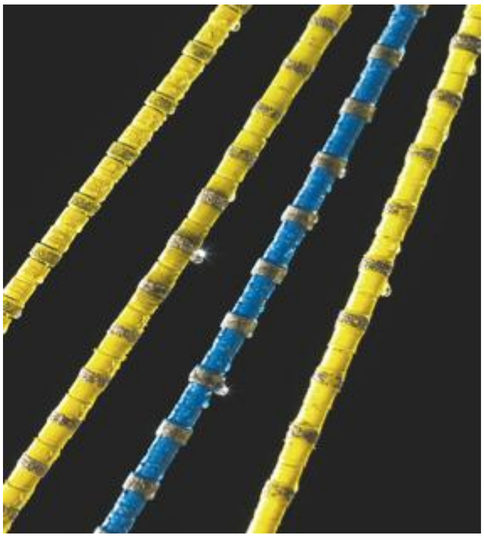

Figure 4. The wire with diamond covered casings.

Since the installation of the rigs is time consuming it is desirable to cut big surfaces instead of small. Beside the maintenance the actual cutting is moderately personnel demanding and cutting can go on during the whole day because of the low disturbance to the surroundings, according to Nyberg[8].

When the holes are drilled and the wire has been run through them and back to the motor the cutting can begin. The wire is cooled with water that is also used to flush away the cuttings. When the wire has cut through the rock it is placed through new holes and a new cut begins. The water used to flush away the cuttings will also reduce the dust that otherwise would be produced by the cutting. The working environment is therefore dust free and moderately quiet. Because of the wire's high velocity, 27 m/sec, there is a risk of a lash if the wire break and the safety distance is therefore the same length as the wire. The safety distance is behind the motor i.e. in the same direction as the wire. Nyberg[9].

[7] Urban Nyberg, general manager, DWTeknikk. Interview 2010-04-28.

[8] Urban Nyberg, general manager, DWTeknikk. Interview 2010-04-28

[9] Urban Nyberg, general manager, DWTeknikk. Interview 2010-04-28

3.1.1 Shafts

A shaft is a vertical excavation which is accessed from two levels. There are several positive aspects for constructing a shaft with wire cutting and is therefore probably the most favourable type of cut.

Four holes are drilled from the surface level down to the tunnel or underground cavity in a square or rectangular pattern. The wire is then fed through one of the holes down to the underground level and then fed back up to the surface through another hole. The two ends of the wire are then connected and mounted to the motor that is installed on rails at the surface. The motor can also be placed in the tunnel to reduce the noise level, this admits cutting during noise restricted hours. The wire is then pulled round and starts to wear through the rock, as illustrated in figure 5. The velocity that the wire cuts through the rock depends on the rocks quality and the stress situation. When one side is cut the wire is moved and starts to cut on the opposite side, according to Krekula[10].

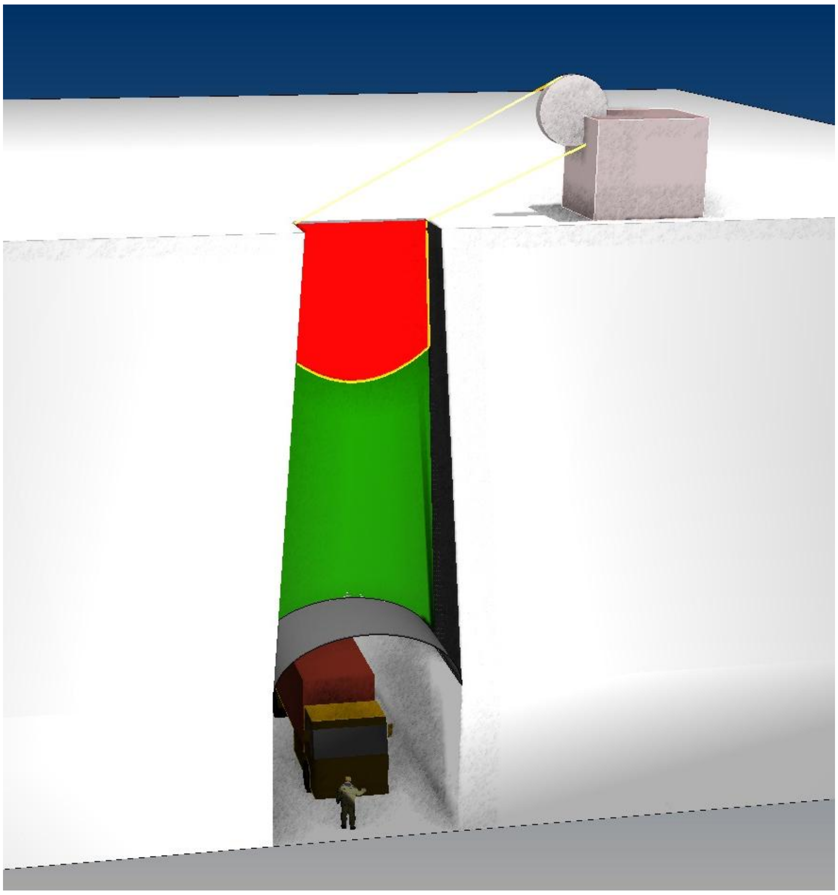

Figure 5. Principle of a wire cut shaft. The green area is already cut and the red area is the remaining area to be cut.

Under what is going to be the shaft a concrete pillar is casted to bear the weight of the rock mass when it's disengaged. When the four sides are cut the rock mass now stands on the concrete pillar and is no longer connected to the rock. The pillar is then blasted and the cut volume falls down in the tunnel where it can be blasted to convenient pieces for removal. Since the wedge is detached from the rock it can now be blasted without creating too high vibration levels. (DWTeknikk 2010). Another

[10] Thomas Krekula, project manager, NCC Construction Sverige AB. Interview 2010-02-23.

way to remove the rock mass is to mould bars in the excavated piece and lower it threw the shaft down to the tunnel with the help of a hydraulic jack. The removal of the rock mass can in some cases be facilitated if the rock mass is in one piece, see figure 6.

Figure 6. The excavated rock mass as a homogenous unit.

Wire cutting can be used for both small and big shafts. The limitation in size is the handling of the detached rock mass, according to Nyberg[11].

The holes are angled out from each other so the shaft gets a cone shape with the larger side downwards. This is to ensure that the rock volume doesn't get stuck when all four sides are cut. (DWTeknikk 2010).

When the rock is removed a shaft is created with smooth sides, see figure 7. The sides are scaled by hand and reinforced if needed. At shallow depths the need of reinforcements in shafts are small due to low horizontal stresses, according to Outters[12].

[11] Urban Nyberg, general manager, DWTeknikk. Interview 2010-04-28

[12] Nils Outters, Licentiate of Engineering, NCC Teknik. Interview 2010-04-20.

Figure 7. The smooth surfaces of a wire cut shaft.

If the shaft is blasted more rigorous scaling has to be done because of the more extended disturbed zone and the surface is usually reinforced with shotcrete. If the shaft is used for ventilation the smooth surface from the wire cut method is to prefer because of less turbulence, according to Nyberg[13].

3.1.2 Cuts

Wire cutting is used to make two dimensional cuts in rock, often close to sensitive objects or other situations which exclude blasting.

If the geometry does not allow the wire to be drawn around the volume of rock another technique can be applied.

The idea is to cut the rock from the surface and down to the desired depth. 250mm core drilled holes are made vertically with a c/c-distance that is dependent on the rock's quality. The possibility of a curved cut increases with the c/c-distance but is also dependent on the rocks joint system and fractures.

In the bottom of the vertical core-drilled hole a bilge pump is placed. The pump is placed under the pulley to drain the cooling water and drill cuttings. It is important that the pump has sufficient capacity to keep the hole dry. If part of the cut is under the groundwater table and the pumps capacity is less than the inflow, the pump cannot drain the hole. In such case there is a risk that cutting will be made with decreased efficiency. Water will stuck on the cases on the wire and overwhelm it. There is also a risk that the cases will aquaplane on the rock surface and slide instead of cut the rock.

In the core drilled holes a rod is placed with a pulley at the end, see figure 8. The wire is fed down the hole and back with the help of the pulley, up to the surface level and then down in the next hole. From the second hole the wire goes back to the motor. With this system the cut is made from the surface level down to the desired depth. See figure 9.

---

[13] Urban Nyberg, general manager, DWTeknikk. Interview 2010-04-28

Figure 8. A blind cut with the rod sticking up with the pulley on top.

After the cut is made the rock mass is still connected at the bottom. So if vibration levels are restricted the semi-detached rock mass can be blasted with small charges or knocked to pieces with a hydraulic jackhammer.

This technique allows cutting in volumes that are not accessed from the sides, just from the surface. When using the technique with rods and pulleys the wire is pushed against the rock instead of pulled. This reduces the efficiency of the cutting with approximately 35-40% compared with pulling the wire.

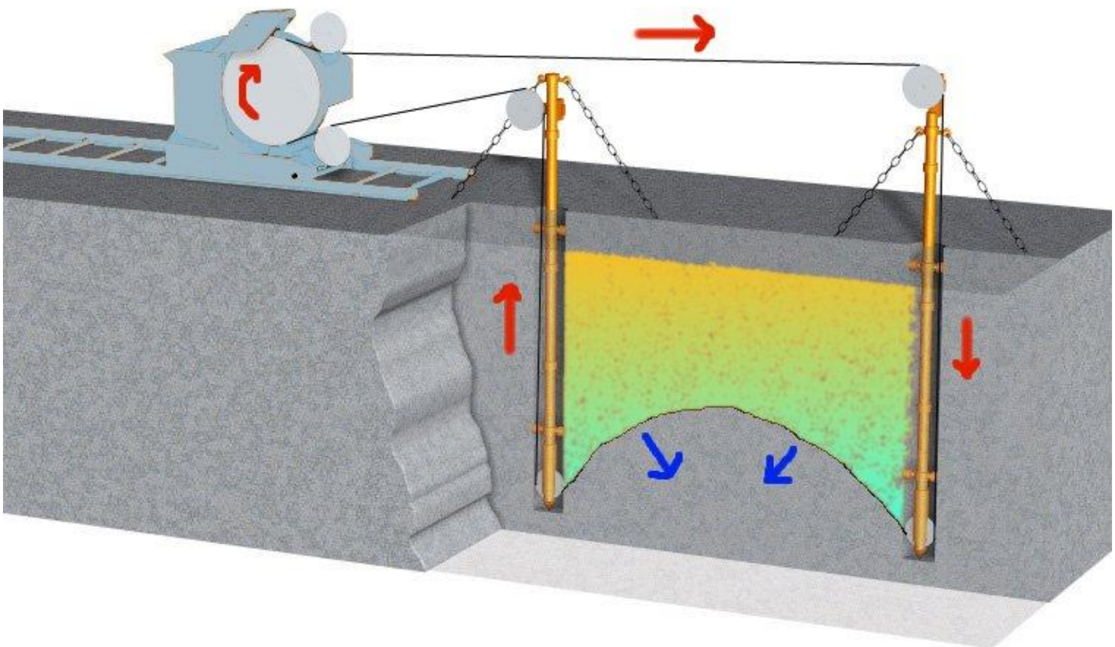

Figure 9. The red arrows shows the direction of the wire and the blue arrows shows the progress of the cutting.

The 250-mm blind cuts mean that the pulley has a small radius which leads to great stress on the wire. If the blind cuts are made 350-mm instead the tension on the wire can be greater and the cutting more effective. However, there are few entrepreneurs that can drill 350-mm holes and the costs for bigger holes are therefore significantly more expensive, according to Nyberg[14]. Figure 10 shows two blind cuts and the smooth cut surfaces.

Figure 10. Two blind cuts and the cut surface.

3.1.3 Tunnel profiles

It is possible to excavate tunnels with wire cutting. The principle is the same as for cuts using rods and pulleys. Depending on the desired shape of the tunnel the number of cuts varies.

This is a time consuming method compared to conventional blasting and is therefore only used in special situations. It is often used when a tunnel crosses near an already existing tunnels and blasting cannot be performed because of blasting regulations, according to Krekula[15].

After the sides are cut the block is still attached to the rock in the back side. To cut the last side the rods needs to be moved and the wire re-fixed. This manoeuvre is complicated and time consuming, according to Nyberg[16].

If the tunnel front is accessed from both sides the cutting procedure is less complicated. Holes are drilled threw the rock mass and wire fed threw one hole and the back again threw another hole. The wire is then connected to the motor. When cutting this way the block can be divided into smaller units more easily because of the cheaper and simpler drilling procedure. This facilitates the removal of the blocks.

This way of excavating a tunnel is often used when passing crucial zones. Wire cutting also means that no cover for fly rocks needs to be installed. Krekula[17].

---

[14] Urban Nyberg, general manager, DWTeknikk. Interview 2010-04-28

[15] Thomas Krekula, project manager, NCC Construction Sverige AB. Interview 2010-02-23.

[16] Urban Nyberg, general manager, DWTeknikk. Interview 2010-04-28

[17] Thomas Krekula, project manager, NCC Construction Sverige AB. Interview 2010-02-23.

Figure 11. A wire cut tunnel front passing under an existing tunnel.

4 Rock mechanical aspects

There is a decisive difference between mechanics and rock mechanics and between strength of materials and strength of rock. In mechanics the loads are often known as well as the strength properties. Because of the rock mass often inhomogeneous structure it is hard to predict the rock mass load and the strength of the rock mass.

What so special when building in rock is that the rock mass itself is both the load and the construction material. This in combination with the sometimes unknown characteristics of the rock mass requires sufficient reinforcement methods (Hansàgi 1965).

The stability of an underground excavation is dependent on the rock mass character, the excavation geometry and the stress situation around the cavity.

The reason for stability problems in rock may vary but it is always a matter of interaction between stress and the rock's strength. There are three different kinds of stability problems:

- Fracture along joints and plane of weakness

- Fracture in rock mass because of low strength in the material, the tensions is not extreme,

- Fracture in rock mass because of high stresses, good quality in material.

In Sweden the first scenario is the most common reason for stability problems. Wedges fall or slide out because of their dead weight. During normal stress conditions fractures occur in rock that has low or no strength, normally in joints. The rock surrounding the weak zone does not fracture and the weak zone is small and dependent on the excavation method. (Nelson 2000).

If an excavation is made with wire cutting instead of traditional drill and blast the geometry of the excavation may have another shape. It is more likely that the geometry will have a more rectangular shape which implies a greater zone of tensile stress and also stress concentrations in the corners, see Chapter 5, Section 5.2.1.

4.1 Background

Mankind has been building in the underground for centuries and the principle of rock mechanics has been understood for a long time. A breakthrough in the science was when rock was recognised as both elastic and discontinuous.

When building an underground construction it is extremely important that one understands the rock. The geological model and the data used in the model are important for understanding the behaviour of the rock. But even with large data samples the results from the model can be inaccurate if the model itself is wrong or misinterpret. (Hoek 2007).

4.2 Wedge failure

The most common type of failure in a tunnel in jointed rock is when a wedge falls from the tunnel roof or slides out of the sidewalls. (Hoek 2007).

A wedge is a part of homogenous rock surrounded by joints or open surfaces so that a separated volume is formed. During excavation a free face is created which removes the restraint from the surrounding rock. Then a wedge can slide or fall from the surface into the tunnel.

Without rock support the stability of the tunnel may deteriorate quickly when loose wedges fall or slide into the tunnel. When a wedge falls it causes a reduction in the restraint and the interlocking of the rock mass, which can lead to slides of other wedges. The sliding will continue until natural arching action is achieved or the tunnel is filled with fallen wedges. (Hoek 2007).

Before blasting starts there are in-situ stresses that are normally distributed in a horizontal and vertical plane. A wedge is affected by theses stresses and is fixed. When the blasting and excavation start the stresses will change and a wedge in the tunnel roof will lose its vertical stress and support from underneath. At the same time the horizontal stresses will change direction and follow the tunnel contour, see figure 12.

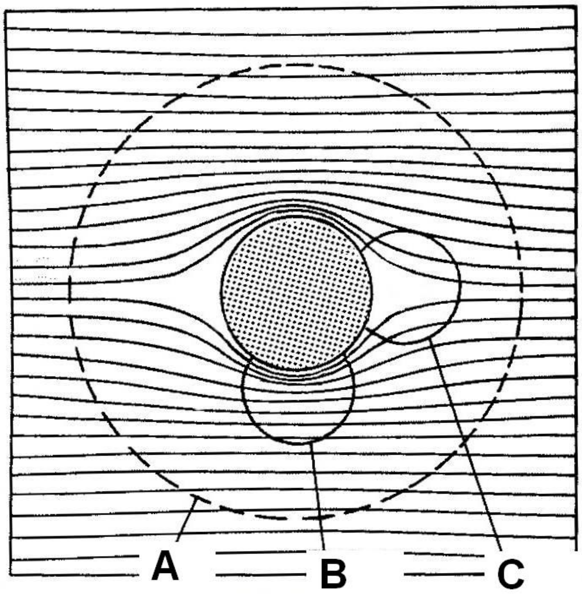

Figure 12. Changes in stress orientation because of the cavity. A: the affected area, B: zone with compressive stress, C: zone with tensile stress.

The change of direction of the stresses is the main reason why the wedge doesn't fall and this phenomenon is called the arching action. The stability of a wedge and the characteristics of the arch are correlated.

When the rock mass has elastic properties the size and direction of the forces that effect upon the wedge can be estimated if the in-situ forces are known. (Stille & Nord 1990).

4.3 Arching

When excavating in hard jointed rock the arching action is important for the tunnel stability. The arch is created by loose rocks that carry the vertical load and around the opening. If there is a risk that one of the rocks can fall it needs to be supported, otherwise the whole arch may collapse. The arch can collapse in three different ways:

- by sliding in a joint
- by crushing the joint or the rock
- rotation of rocks

The most common reason for an arch failure is sliding in a joint. It occurs when the shear stress exceeds the joints shear strength.

The geometrical factors that affect the arching action are the:

- bearing distance
- the dip of the joints
- the distance between the joints

The arch is also affected by the rock mass properties, the horizontal pressure stress and the friction and coarseness of the joint planes.

The horizontal pressure stress that can establish is influenced by the in situ stress, additional elastic stress from the excavation and additional stress from rotation of rocks. In typical Swedish rock it is unusual that the arch becomes unstable due to rotation of rock. (Stille & Nord 1990).

The arching action is dependent on the geometry of the excavation. The arching action will be situated closer to the cavity if the cavity has a more rounded profile while if the cavity has a more rectangular profile the overburden between the cavity and the arch is greater. A greater overburden means a bigger volume of rock with potential loose wedges that needs to be secured.

4.4 Rock overburden

Excavation with shallow rock overburden is often more complicated than deeper excavation.

The common definition for poor rock overburden is that the overburden is less than half of the tunnel span. Building tunnels with poor rock overburden has become more common with more infrastructures in urban areas. Overburden may be poor because the tunnel lies shallow or because of a crossing with another underground construction.

Building with poor overburden is complicated in several ways and there are different difficulties depending on what makes the overburden poor.

The stand up time for an excavation with poor rock overburden is usually much shorter because the rock does not form a supporting arch on its own. The rocks structural strength is not sufficient to form a load-carrying structure and a pre-reinforcement has to be installed to avoid collapse. This has to be done both during tunnel excavation and during the lifespan of the tunnel.

Building tunnels with poor rock overburden is somewhat similar to building in weak rock from a stability perspective. The horizontal stresses are often small due to weathered rock and so are the vertical stresses because of the small rock overburden. (Gerken & Renner 2009).

4.5 Rock classification system

When designing an underground construction there are three different design approaches; analytical, observational and empirical. The most commonly used method is the empirical. Using a rock mass classification system provides a systematic working procedure in an otherwise sometimes trial and error procedure. (Wikipedia (a) 2010).

By using one or two classification systems, estimation of the composition and characteristics of the rock mass can be provided which can give initial estimates of rock support requirements. Deformation and strength properties of the rock can also be achieved.

A rock mass classification does not replace more elaborate design studies and one should be aware of its limitations.

Today in tunnel construction the RMR, Rock Mass Rating, and the Rock tunnelling quality index, the Q-system, are the most frequently used. The two systems are often used together in the same project, according to Outters[18]. The two methods incorporate geological, geometric and engineering parameters and result in numerical values which suggest tunnel support measures. The difference between the two classification systems is the lack of a stress parameter in the RMR model.

4.5.1 RMR

The rock mass rating (RMR) system estimates the strength of the rock masses by using six parameters:

- uniaxial compressive strength of rock material
- rock quality designation, a rock mass in situ parameter
- spacing of discontinuities
- condition of discontinuities
- groundwater conditions
- orientation of discontinuities

The parameters are assigned a value, derived from field surveys, corresponding to the characteristics of the rock. The sum of the six parameters is the RMR value. The RMR value results in guidelines for tunnel support. (Hoek 2007).

4.5.2 Q-system

The other common classification system is the Q-system which was developed in Norway by Barton et al. It was developed after analysis of 212 underground excavation cases. The index determines the rock mass characteristics and tunnel support requirements. The index is defined by:

- rock quality designation
- joint set number
- joint set roughness number
- joint alteration number
- joint water reduction number
- stress reduction factor

The index can also be described by the three parameters: block size, inter-block shear strength and active stress. (Hoek 2007).

[18] Nils Outters, Licentiate of Engineering, NCC Teknik. Interview 2010-04-20.

5 Numerical modelling of rock stresses regarding different tunnel profiles

With modern excavation methods it is possible to excavate underground facilities in desired shape and size. But usually the shape is determined by the function of the underground construction. Even shapes with great spans are possible in rock with poor overburden due to refined reinforcement methods.

With the drill and blast method it is possible to create the desired shapes but in some environments it not suitable due to vibration restrictions. In some of these cases the problem is solved with wire cutting instead.

When an excavation is made in the rock the stresses follow the contour of the cavity. A more streamlined cavity decreases the stress concentration that usually occurs in the corners and that is why a more rounded shape is more desirable. A rounded roofline, an arch shape, also contributes to carry the vertical loads more effectively.

The best suited geometry from a stress situation point of view is a circular or a horse shoe formed geometry. But in many cases another shape or profile is more desirable. When using the drill and blast technique the desired shape is relatively easy to achieve, the drill holes are placed where the rock is to be removed.

Because of the excavation technique of the wire cutting the geometry is usually somewhat simplified. The often desired arch shape of a tunnel is harder to create with wire cutting so the design often becomes more rectangular. It is possible to create a more rounded shape but a more rounded shape requires extra core drilled holes. (Nyberg DWT)

A different shape of the underground construction will influence the bearing capacity and the stress situation. How these consequences will result in different reinforcements is a site specific issue and differs from case to case.

The shape of the excavation has a decisive role of the stress situation together with the in-situ stress. (Stille & Nord 1990).

5.1 Profile simulation

To see how the different tunnel profiles influence the stress situations a simulation with different tunnel profiles was made. The different profiles were chosen to visualize how stress has a more negative effect on a more rectangular profile. A profile that is more exposed to stress due to its more rectangular shape needs more reinforcement to avoid collapse.

Depending on the stress situation and the shape of the excavation there is tensile stress or compressive stress in the tunnel roof. The size of the loose core of rock between the tunnel roof and the arch depends on the stress situation. A bigger loose core requires more reinforcement.

The data used in the models are taken from the commuter-train tunnel in central Stockholm, Stockholm City Line. The conditions for Stockholm City Line are considered valid for environments in the Stockholm area.

The profiles in the simulation stretch from a square shape to a shape with an arched shape roof. A circular shape is also modelled to represent a bored tunnel.

In the simulation model some assumptions were made. The rock mass is considered to be homogenous and isotropic or transversely isotropic. The rock mass is also expected to be linearly elastic. The modelled excavation is of infinite length normal to the plane section.

The equation that describes the stress situation around a hole in an infinite plane in one direction was developed by Ernst Gustav Kirsch.

The simulations are made in the software programme Examine 2D from RocScience. In appendix 3 there is a screenshot from the software programme.

5.1.1 Loose zone

The loose zone is a volume of rock exposed to tensile stress. Beyond the zone the rock mass is exposed to regular compressive stress. Because of the rock mass's poor tensile strength the loose zone can be regarded as a load on the tunnel roof. If a loose zone is occurring in the tunnel side it is possible that rocks will fall out.

If the volume of loose rock mass is known the amount of bolts to secure the rock mass can be estimated. Due to the difficult estimation where loose rock mass becomes "hard" a widely used estimation is that to secure the rock mass the bolts should have a length that is twice as long as the loose rock thickness. As the simulation shows, the volume of loose rock increases both in width and in height. Thus a greater volume of loose rock needs to be secured not only with more bolts, it also requires longer bolts.

The equation 5.1. shows the amount of bolts required to secure the loose rock. (Bjurström & Heimersson 1975)

$$N = \left(\frac{V * \rho * n}{B} \right) \tag{5.1}$$

N= the amount of bolts [pcs]

V= the volume of the loose zone [m^3],

ρ= rock mass density [kg/m^3]

B= rock bolts load-bearing capacity [kN]

n= factor of safety

5.2 Results

The Stockholm City Line, as most of Swedish infrastructural projects, lies moderately shallow which results in a low vertical stress, close to zero. The largest horizontal stress is perpendicular to the tunnel and is 4.7MPa. The stress parallel to tunnel is 2.3MPa.

It is the horizontal stresses that affect the cavity due to low vertical stress.

5.2.1 Rectangular shape

The rectangular shaped cavity creates less of an arch above the tunnel roof. This means that there is greater risk that potential loose wedges will fall into the cavity because the lack of upholding forces of the arch.

The sharper corners create stress concentrations where also the maximum stress occurs, 7.7MPa. These corners needs to be extra reinforced to avoid failure, especially the upper corners.

The relatively strong horizontal stresses are distributed over and under the cavity which results in tensile stresses at the sides and compressive stresses at the top and bottom. The greatest tensile stress occurs at middle of the sides with a value of -1.4MPa. In figure 13 the blue areas represent areas with tensile stress. A tensile stress instead of a compressive stress means greater risk that loose wedges will slide into the cavity, a compressive force would better lock the wedges in place.

The lack of an arching action is compensated with more reinforcement to secure the cavity.

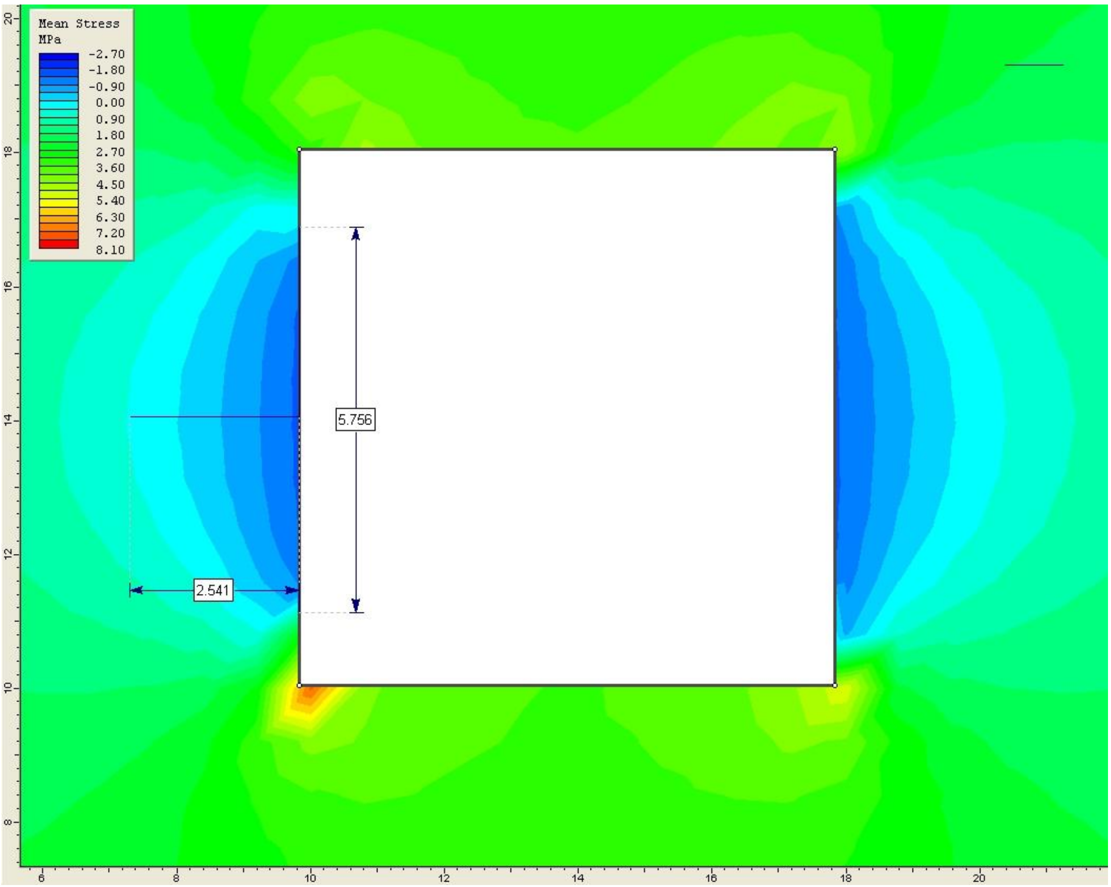

Figure 13. A rectangular tunnel profile with zones of tensile stress and lack of arch.

5.2.2 Rounded roof shape

The rounded roof shape displays similarities with the rectangular shape. It is also exposed to tensile stresses at the sides and strong compressive forces at the lower corners. The great difference is the arching action developed over the cavity.

With the rounded roof line the roof functions as an arch instead of a straight beam which is more suitable from a load bearing perspective.

The rounded roof and its raising are pressed down by the horizontal forces. This effect causes pressure on the arch and creates the arching action even if there are low or no vertical load.

The tunnel profile with a rounded tunnel roof has a smaller zone with tensile stress at the side of the tunnel. This zone which is exposed to tensile stress requires more reinforcements than if it had been exposed to compressive stress. There is a bigger probability that wedges will fall out from a rock mass exposed to tensile stress. When the rock is exposed to compressive stress the joints coarseness and the friction will better hold the wedges in place.

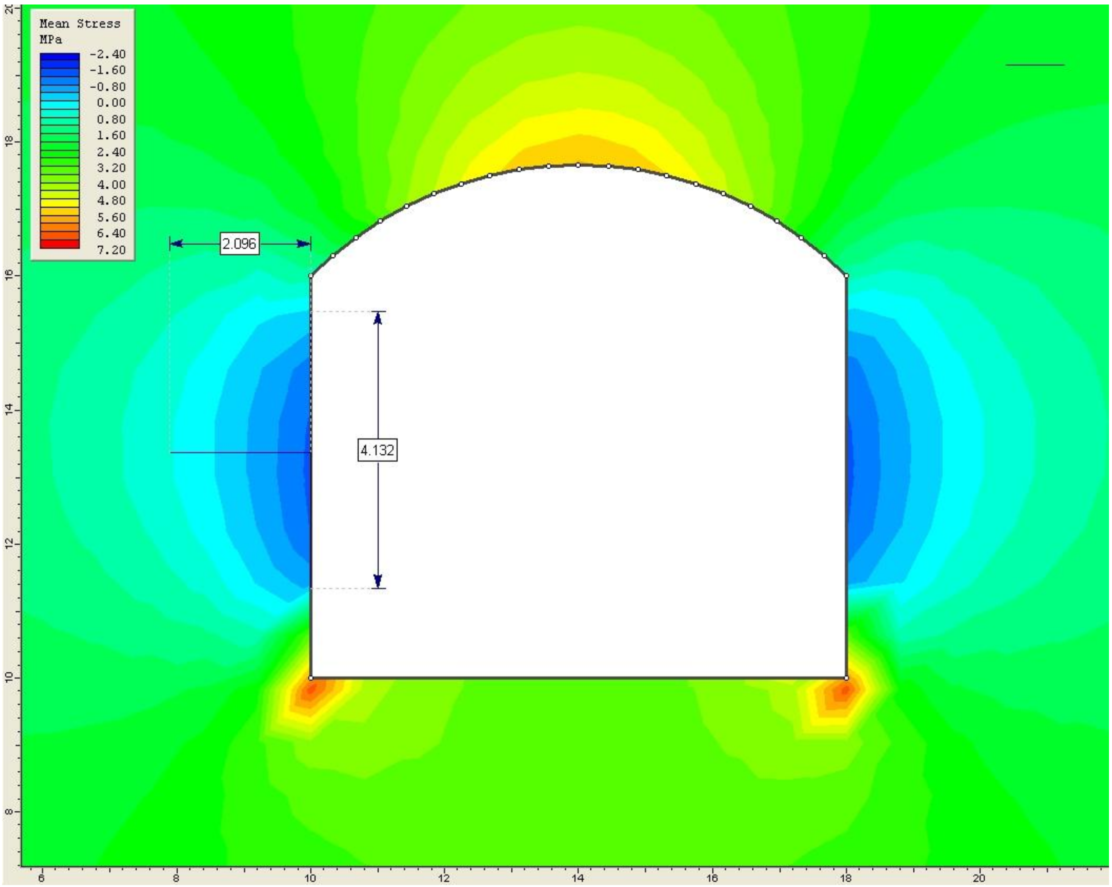

Figure. 14. A tunnel profile with an arch shaped roof. Zones with tensile stress and a zone with compressive stress above the tunnel roof which functions as a load bearing arch.

5.2.3 Circular shape

The circular shape, that represents a bored tunnel, displays zones of tensile stress in the horizontal plane and compressive stress in the vertical plane.

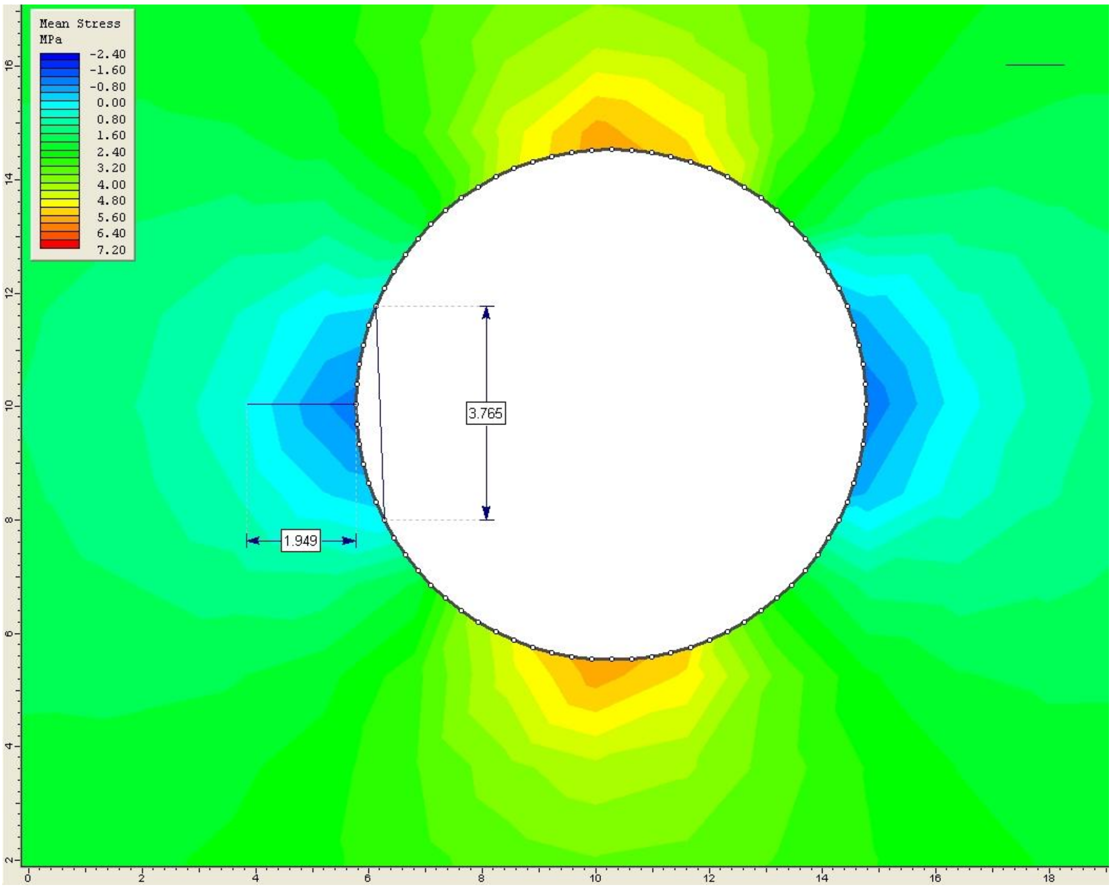

Figure. 15. A circular tunnel profile with compressive stress above and under the cavity and tensile stresses in the horizontal plane.

6 Rock reinforcement in a wire cutting perspective

The objective of rock reinforcement is to use the inherent strength of the rock mass so it becomes self-supporting. Rock support is the term used when structural elements carry the weight of rock blocks or rock masses isolated by joints or zones of loosened rock.

Rock reinforcements are the measures taken to create a safe and stable cavity in the rock. The most common rock reinforcement materials are steel and concrete. The dimensions of the reinforcements are different depending on the lifetime and usage of the construction. Constructions where people reside have a higher factor of safety than e.g. a mine. There is also a time difference with rock reinforcement. The great difference is between short time reinforcement and permanent reinforcement.

The short time reinforcement can be considered as a temporary reinforcement and is used to reach a safety level during the excavation phase. The reinforcement actions are applied at the tunnel front. The permanent measures are installed a couple of hundred meters behind the front or when it's practicable. The temporary reinforcements are often part of the permanent measures.

The reinforcement measures mentioned in the text are the most common used strategies in the Western Europe, and some other techniques may be used in other parts of the world. (Stille & Nord, 1990).

6.1 Rock bolts

Using rock bolts as rock reinforcement is one of the most used methods, mainly because the technique is easy, fairly cheap, and robust and offers great mechanical opportunities. The idea is to anchor loose wedges into the underlying steady rock. But the technique can also join loose wedges into a bearing construction (see figure 16), decrease the scattering in rock mass, and facilitate the stabilization of the rock mass. Rock bolts are often used in combination with other reinforcement methods.

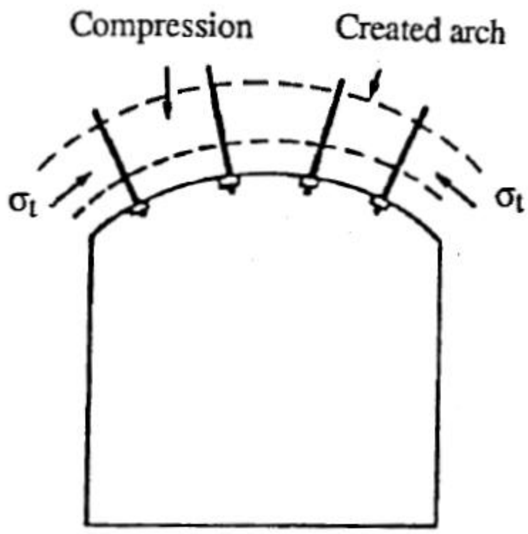

Figure 16. Rock bolts securing loose wedges into a load bearing arch

There are different kinds of bolts with different manner of action. There are tensioned non- or partly grouted bolts, grouted non tensioned bolts, grouted and tensioned bolts and friction bolts. (Stille & Nord 1990).

The most frequently used bolt in Swedish tunnel building is the grouted non tensioned bolt. The bolt is usually a reinforcing bar rod with standard diameter and of standard quality, according to Gårdinger[19].

The procedure of installing a rock bolt is drilling a hole and filling it with cement. The duration of hardening can vary from a couple of hours to 4-6 days depending on if an accelerating supplement has been added. After the borehole is filled with cement the bolt is installed in the hole.

Because of the high strength of the cement a variation of the borehole is not that relevant. Under favourable conditions the anchorage strength of a bolt can reach 10kN/cm of installed bolt, but one should not count for more than 2kN/cm. If plastic is used instead of cement the anchorage strength can reach 5 to 6kN/cm.

There is no non-destructive testing method of rock bolts today why the installation phase is very important to reach a satisfying result.

After installation the bolt head is often covered with shotcrete to protect it from corrosion.

The rock bolt dimension should correlate with the loose rock masses size, direction and kind so there is a balance between the reinforcement requirement and the reinforcement of the bolt.

When dimensioning for rock bolts these parameters needs to be determined:

- type of bolt
- length of bolt
- load bearing capacity
- number of bolts
- placing and direction
- installation and control procedure

(Stille & Nord 1990).

6.1.1 Normal bolt pattern

The normal procedure of reinforcing the rock mass is by bolting. The amount and pattern of the bolts are determined from the rock classification. (Bjurström & Heimersson 1975).

The bolt reinforcement is assumed to contribute to the rock mass stabilization by joining elements that are separated by joints. In this way the loose blocks will work together in a constructive load-bearing unit.

With systematic bolting loose blocks can be bolted to form an arch. (Stille & Nord 1990).

[19] Carl Johan Gårdinger, civil engineer, NCC Teknik. Interview 2010-02-25.

6.1.2 Selective bolting

When using selective bolting the rock mass has already been stabilized. But there can be loose wedges that need to be anchored in the underlying stable rock. The size of the loose wedge is appreciated by the orientation, character and frequency of the joints. When the size is known the wedge can be anchored with sufficient amount of bolts. During the excavation a geologist maps the newly excavated area to see if the joints correspond to the rock classification. (Stille & Nord 1999).

Where wire cutting is used mapping can be somewhat different. The frequency of the joints is easy to determine but the orientation can be harder to estimate because of the smooth surface, see figure 17. The orientation can be determined if there is a three-dimensional cut but is harder if there is only one side that is cut, according to Montelius[20]. There is often a blasted surface in the vicinity that can be used for geological mapping. On a blasted surface the orientation of the joints is easier to map than on a wire cut surface.

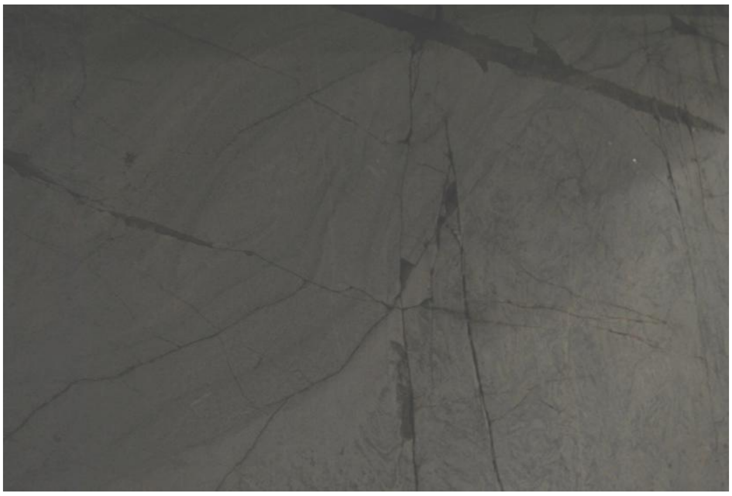

Figure. 17. Joints on a wire cut surface.

6.2 Shotcrete

Shotcrete is a common measure for reinforcing underground excavation. It mainly has two functions; keep smaller rocks from falling down and give an overall support.

Shotcrete with steel fibre reinforcement has in many projects replaced the traditional wire mesh reinforced plain concrete. The idea with steel fibre is to increase the ductility of the shotcrete so it can carry the load of a deformed rock. (Hoek 2007).

Shotcrete can be applied with two different methods; wet or dry method, where the wet method is most common in civil projects.

When applying the shotcrete the concrete paste pierce into the joints and seal the surface which stops small blocks from falling out. The bearing capacity of the shotcrete depends on the adhesitivity between the rock surface and the shotcrete. (Stille & Nord, 1990).

It is hard to estimate the bearing capacity since it is an imprecise process where experience is a great factor. However it almost always performs better than expected. (Hoek 2007). Despite the lack of rules for dimensioning, shotcrete is considered as a

[20] Cecilia Montelius, geologist, NCC Teknik. Interview 2010-04-20.

reinforcement method with considerable capacity even in complicated conditions. (Stille & Nord, 1990).

The shotcrete is sprayed on pneumatically on the rock surface in layers not thicker than 20cm at the time. About 15-20% of the sprayed shotcrete rebound from the surface. The wet method requires a rather high vct-number and therefore an accelerator and other adulterant are added. The final structural strength is about 25-30MPa. Regardless of which method the shotcrete is sprayed on with but the rock surface has to be cleaned to ensure full adhesitivity. (Stille & Nord 1990).

6.2.1 Adhesitivity

The adhesitivity between the rock's surface and the shotcrete is the most important property of the shotcrete. It prevents the shotcrete bending fractures. Steel-fibre reinforced shotcrete decrease the risk of such failure but it is still the adhesitivity that determines if the shotcrete will break or not.

The adhesitivity is dependent on the rock's mineral composition and also on the surface's character. The adhesitivity will also be better if the surface is washed before the shotcrete is sprayed on. (Vedin 2006)

A wire cut surface is smoother and has two to three times' greater adhesitivity than a blasted surface. This could mean that the shotcrete layer could be thinner when applied on a wire cut surface but with the same reinforcement strength, according to Öhlen[21].

6.3 Steel arches

The steel arches as reinforcement method descent from the mining industry in the 1920's and 30's. The method is more of a supporting construction than of reinforcement like the bolts and shotcrete. Installing a steel arch is somewhat time-consuming and is therefore only used in very poor rock where bolt and shotcrete are not sufficient.

When the loads are known for what the steel arch will be exposed to the arch can be dimensioned. The arch is made of steel HEB- or I-beams that are bolted together to form the desired shape.

Steel arches can be an economical solution when other methods are hard to install or when the reinforcement must have effect immediately. This could be the case when the rock has disintegrated close to soil or is full of joints which lead to short close-up time. Steel arches can also be a reasonable solution when there is a strong water inflow which makes it difficult for the shotcrete to attach. These scenarios are most common at the beginning of the tunnel. (Stille & Nord 1990).

6.4 Concrete arches

Concrete arches are used when the rock quality is poor or when there is a stretch with poor rock overburden. It is often used for a limited stretch or in a special area where the conventional reinforcement methods are not sufficient.

There are two ways of constructing the arches; sprayed on with shotcrete with reinforcement net or casted in a form with reinforcement bars. As in the case with

[21] Lars-Åke Öhlen, site manager NCC Construction. Norrströmstunneln, Stockholm City Line. Interview 2010.03-15

steel arches a wire cut surface facilitate the installation of arches due to the smother surface, according to Gårdinger[22].

6.4.1 Shotcrete arches

When making shotcrete arches reinforcement net is first bolted to the rock. The shotcrete is then sprayed on in layers to the required thickness. The thickness is measured from the theoretical tunnel contour. The theoretical tunnel contour is the determined tunnel front area and for achieving this specific area more rock is usually excavated than necessary. When blasting irregularities in the surface occur and the tunnel contour is measured from the tops. This means that the pits need to be filled with shotcrete. A wire cut surface is smooth and the cut surface can more easily be made closer to the theoretical tunnel contour, as illustrated in figure 18. This means that the sprayed on concrete can be thinner but still reach the required thickness. More concrete is consequently being used on a blasted surface, according to Gårdinger[23].

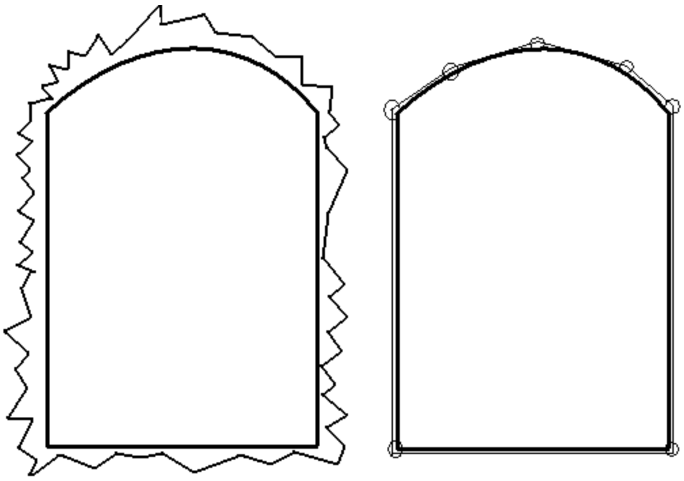

Figure 18. The theoretical tunnel contour and the amount of overbreak depending on the excavation method: blasting to the left and wire cutting to the right.

6.4.2 Site casted concrete arches

The site casted arch is made with a mould and reinforcement bars. As in the case with the shotcrete arches a blasted surface requires more concrete than casting the arch against a wire cut surface. It is also easier to create the mould and fitting it when there are sharp edges and smooth surfaces. The mould is also easier to seal against a wire cut surface, according to Gårdinger[24].

[22] Carl Johan Gårdinger, civil engineer, NCC Teknik. Interview 2010-02-25.

[23] Carl Johan Gårdinger, civil engineer, NCC Teknik. Interview 2010-02-25.

[24] Carl Johan Gårdinger, civil engineer, NCC Teknik. Interview 2010-02-25.

7 Economical comparison for rock excavation of two tunnels with different conditions

Wire cutting and rock blasting are two different methods for extracting rock mass in an underground excavation. An economical comparison was made to distinguish which type of method is suitable for different excavation types.

In the comparison values from two different tunnels have been examined. The data comes from the blasting journals with the following parameters; number of holes, length of detonation, area, specific drilling, specific charge, maximum cooperative charge and type and amount of explosives. These data have been calculated to obtain how much explosives where needed to excavate one cubic meter of rock.

The two tunnels have been blasted in different ways; carefully and normally. The two sites are part of the project Stockholm City Line. Different explosives are used in the tunnels. In the tunnel with more restricted vibration regulations only cartridge explosives have been used. In the tunnel with longer drill holes and more coarse drill diameter an emulsion explosive was used. The more vibration sensitive tunnel was blasted with pilot and stope to decrease the cooperative charge. Empirical values were used for drilling and charging. There are different values for the carefully blasted tunnel and for the normally blasted tunnel because of more drilling and charging for the carefully blasted tunnel. For the carefully blasted tunnel the amount of drilled meter and the amount of drilled holes are greater than for the normally blasted tunnel.

Focus in the comparison is on the economy and differences in tunnel profile and reinforcement needs are not considered. The costs used in the comparison origins from NCC's contract offer department and can be found in appendix 2.

The aim with the comparison is to give an idea over the costs for excavating tunnels with different methods and different conditions.

The data for the wire cutting is an empirical value which include cutting, drilling of blind cuts, wear, spare parts and manpower.

The extraction of rock masses is assumed to be equal for both methods.

The economical comparison has been made according to a generalization of underground excavation. No regards have been taken to the design process or the question of regulations. The comparison is made only with the purpose to analyze the economical consequences from different excavation methods.

7.1 Normal blasting

The data is from a working tunnel with a tunnel area of $62m^2$. The studied part of the tunnel is 40 metres long and was excavated with seven detonation rounds, each round was 5,8 metres which is the normal length for a tunnel when excavating without vibration regulations. Each round used an average of 884kg of explosives and the specific charge, the amount of explosive for one m^3, was 2,5kg. The average cooperative charge was 8,3kg.

The tunnel is situated 100 meters from the sensitive equipment of the subway system. The vibration limit was 30mm/sec but if the entrepreneur could not guarantee the limits the close by part of the subway was closed down. If the subway is closed down and evacuated of people the vibration limit is raised to 92mm/sec instead.

The simulated wire cutting is performed with four blind cuts and by using rods and pulleys. Because of the tunnel length and the handling of the excavated masses the cutting has to be divided into four segments. This creates three more cross-section areas. The total area to cut is 1552m^2. The explosive being used was SSE with primer.

7.2 Careful blasting

For the comparison with a carefully blasted tunnel the data from an access tunnel within the Stockholm City Line project has been used. The tunnel is situated only ten meters away from the subway system and the vibration limit was originally 10 mm/sec but was later changed to 30mm/sec. If the subway was closed down, for example during the night, the limit was raised to 70mm/sec. The blasting was carried out during the night to utilize the more generous vibration limit. Some of the blasts had higher vibration levels and some lower which shows in the data files from the length of the rounds.

The studied part of the tunnel is 10m and with a cross section area of 63m^2. It took twenty rounds to excavate 10 metres of tunnel. The excavation was carried out by blasting a pilot and then the stope. The average length of the rounds was 1,85m and the specific charge 1,3kg. The cooperative charge was 0,4kg.

Excavating the rock mass with wire cutting is performed in the same way as in the case with normal blasting. Since the tunnel is ten meters long the cutting can be made in one segment.

7.3 Result

For the tunnel with low vibration regulations an emulsion explosive was used. It is cheaper than cartridge explosives but can only be used for greater volumes. The comparison cost for excavating one m^3 of rock mass is 171 SEK. The biggest cost is drilling, 99 SEK and charging 32 SEK. The cost for excavating one m^3 with wire cutting is 963 SEK.

For the tunnel with vibration regulations the cost for one m^3 is 436 SEK. The biggest cost is drilling for 190 SEK and charging for 118 SEK. Almost one fourth of the costs are the ignition caps. If electronic ignition caps were used the cost would increase with roughly 150 SEK.

If the excavation was to be done with wire cutting instead the cost for one m^3 would have been 955 SEK.

The tunnel with low vibration regulations is the cheapest to excavate. Less drilling and cheaper explosives are the main reason of the lower costs. In this comparison it is included as a reference. The tunnel with higher vibration regulation is much more expensive to excavate than the other blasted tunnel but cheaper than wire cutting. According to Öhlén[25] it may have been possible to excavate the tunnel during daytime with the lower level of 30mm/sec but with very small charges, cooperative chargers down to 100g. But even these very small chargers would create vibration levels up to 30mm/sec. Exceeding the vibration limits may result in fines. It is also uncertain if the chargers could break the rock mass. Under such circumstances wire cutting may be the only excavation method available, even if it costs more.

---

[25] Lars-Åke Öhlen, site manager NCC Construction. Norrströmstunneln, Stockholm City Line. Interview 2010.03-15

Because of the more generous vibration limits during the traffic stop the blasting took place during the night. Even with short rounds there is only time for one detonation a day. It means that the ten metre long tunnel takes twenty days to excavate. A normal excavation speed for wire cutting is 5 m^2/hour and so the tunnel will be cut out in 77 hours if the cutting is performed continuously.

8 Discussion

The method of using drill and blast for excavating rock mass has been the most common method for a long time and will probably continue to be so. With more advanced blasting plans and more sophisticated detonators it has become possible to detonate also in special and sensitive environments. However, there is a level when cooperative charge is too small to break the rock mass, and a detonation will always create vibrations. In such conditions wire cutting is a good alternative.

Wire cutting has been used in vibration sensitive environments where blasting is too expensive due to short round lengths or other circumstances where practical aspects weight in favour of wire cutting.

Wire cutting is performed without interruptions and can thus inter alias, be performed in environments close to traffic. Blasting in such situations could cause traffic stops during the detonation. In situations where traffic is heavy, every stop will have negative effects on traffic flow. Excavation without traffic stops is an advantage for society, not only economically, -decreased dissatisfaction due to less traffic stops creates goodwill.

In this thesis, wire cutting has been compared to blasting with a focus on infrastructural projects. In this kind of projects, excavation is always reinforced with rock bolts or shotcrete, regardless of what method is being used.

If the method of excavation will result in a different tunnel profile, the stress situation and the stress orientation will be different for the two excavation types. The difference may result in a bigger zone with loose rock. However, since the tunnel is being reinforced, the difference of the loose zone won't significantly affect the total economy of the project. The cost for the extra rock bolts would probably be small compared to the total project budget. If very big areas would be wire cut and the zones with loose rocks would be much bigger, perhaps a different tunnel profile would have more impact on the project economy than the choice of the excavation method.

Because of the higher cost, wire cutting will only be used when blasting is impossible due to vibrations or when specific circumstances make wire cutting favourable. Those specific circumstances can vary from the need to extract the rock mass in one piece or to make it possible to execute the extraction of the rock mass without disturbing the nearby environment. In addition, in some projects a smooth surface may be desired, e.g. when concreting is to be used. In such situations wire cutting is useful.

The smooth surface created by using wire cutting could also be used as a road surface. When excavating rock mass from detonation, the bottom of the tunnel is covered with macadam to facilitate the accessibility for vehicles. The rock mass from the detonation is transported to a crusher and then transported back to the tunnel. As an alternative, if the tunnel would have been wire cut the smooth surface could be used as the road surface.

9 Conclusions

The objective of this thesis is to compare wire cutting to blasting in environments where blasting has to be performed with decreased efficiency due to strict vibration regulations. Wire cutting and blasting are two completely different methods of excavating rock mass. The different methods will affect the rock mass and consequently have consequences on the construction.

Blasting is the cheapest way of extracting rock mass, but with greater vibration restrictions, the method becomes less economically efficient.

An economical comparison shows that careful blasting costs half as much as excavation with wire cutting. Wire cutting could preferably be used when regulations are too strict for blasting or when the practical advantages are valued higher than the extra cost for wire cutting.

A wire cut tunnel profile often means a more rectangular shape than a blasted tunnel profile. Numerical modelling of rock stresses according to different tunnel profiles has showed different stress orientations and concentrations, especially in the corners. A rectangular shape from the wire cutting implies greater zones with loose rock since the shape decreases the load bearing arch. Most probably, if the span is not too extensive, the differences in the bearing capacity could be treated by extra reinforcement in the form of rock bolts and shotcrete. It is possible to create rounded profiles with wire cutting but a circular shape requires more blind cuts and the blind cuts are expensive and the rig is time consuming to install.

The smooth surface result after wire cutting has up to three times higher adhesitivity than a blasted surface. The better adhesitivity could be used to decrease the thickness of the shotcrete layer but seldom is.

Geological mapping can be affected of the smooth surfaces, especially if there is a lack of blasted surfaces in the vicinity to use as a reference but this is probably only a problem if the whole cavity is excavated with wire cutting technique. Because of the greater costs, wire cutting is, and will probably still be, a technique for special applications where blasting is more or less impossible. Wire cutting needs to be cheaper to compete with drill and blast. A way of reducing costs could be the development of wires to make it possible to cut the rock mass more efficiently and quicker.

10 References

Ambraseys, N. R. & Hendron, A. J. (1968). *Dynamic behaviour of rock masses*.J. Wileys & Sons.

Baumann, Theodor (1993). *Comparison between the projects. In Technology of bored tunnels under deep waterways*. International symposium Copenhagen, 3-5 november 1993. Copenhagen, Denmark.

Bergmekanikdag 2010 (a), Föredrag. (2010). Unpublished manuscript. Stockholm: Stiftelsen bergteknisk forskning

Bergbultning, dimensionering, praxis och tillämpningar. Report no 9. (1979) (b). Stiftelsen Bergteknisk forskning - BeFo. Stockholm

Bjurström, S. & Heimersson, M. (1975) *Bergbultning - Dimensionering, praxis och tillämpningar*. Stockholm. BeFo Rapport Nr. 8.

Brady, B.H.G. & Brown, E.T. (1999). *Rock mechanics – For underground mining*. Oxford: Kluwer Academic Publishers.

DWTeknikk (2010). *Film about Sødra Lenken and wire sawing*. Information video. Available: http://dwteknikk.com/ENG/web/projects.htm. Accessed: 2010-03-02.

Fomenko, Olga & Rudegran Alsterdal, Emil (2008). *Prognostisering av sprängningsinducerade vibrationer i urban miljö*. Luleå: Luleå University of Technology, Institutionen för samhällsbyggnad. Bachelors thesis.

Gerken, Christian & Renner, Linda (2000). *Bergförstärkning vid liten bergtäckning*. Luleå: Tekniska Universitetet, Institutionen för Väg- och vattenbyggnad, avdelningen för Bergmekanik. Master thesis.

Grundkurs i sprängteknik (2009). Unpublished manuscript. Vaxholm: BergUtbildarna AB

Hansági, Imre (1965). *Praktisk bergmekanik och bergförstärkning*. Uppsala: Almqvist & Wiksell/ Gebers Förlag AB

Hapgood, Fred (2004). *The underground cutting edge: the innovators who made diggin tunnels high-tech.* Electronic. Available: http://www.mindfully.org/Technology/2004/Underground-Boring-Machines1sep04.htm. Accessed: 2010-12-01.

Hoek, Evert (2007). *Practical Rock Engineering*. Electronic. Available: http://www.rocscience.com/hoek/PracticalRockEngineering.asp. Accessed: 2010-02-22.

Hoek, E. & Brown, E.T. (1980*). Underground excavations in rock*. 2:nd edition. London: Chapman & Hall

Langefors, Ulf & Kihlström, B. (1963). *Rock Blasting*. Uppsala: Almqvist & Wiksells/ Gebers Förlag AB

Nelson, Magnus (2000). *Brottsannolikhet vid dimensionering av berganläggningar – en studie med fokusering på blockstabilitet*. Stockholm: Kungliga Tekniska Högskolan, Institutionen för Anläggning och miljö, avdelning för Jord- och bergmekanik. Rapport 3071.

Olofsson, Stig (2007). *Modern bergsprängningsteknik.* Ärla: APPLEX

Sandström, Henry (2010). *60 års utveckling av sprängteknik.* (Electronic). Available: http://www.matpool.se/spr-utveckling/. Accessed: 2010-12-02.

Stille, Håkan & Nord, Gunnar (1990). *Bergmekanik del 2*. Stockholm: Kungliga Tekniska Högskolan, Institutionen för Jord- och bergmekanik. Kurslitteratur

Stål, Torbjörn & Wedel, Per (1984). *Handboken Bygg Geoteknik*. Stockholm: LiberFörlag

Vedin, Per (2006). *Sprutbetongförstärkning – Förslag till förbättringar i produktionskedjan*. Luleå: Luleå University of Technology, Institutionen för samhällsbyggnad. Master thesis.

Wikipedia (updated 2010). (a). *Rock mass classification*. (Electronic). Available: http://en.wikipedia.org/wiki/Rock_mass_classification. Accessed: 2010-06-03.

Oral references:

Carl-Johan Gårdinger - NCC Teknik

Cecilia Montelius- NCC Teknik

Lars-Olof Dahlström – NCC Teknik

Urban Nyberg –DWTeknikk

Carl Lind – Nitro Consult

Rolf Christiansson – SKB

Nils Outters – NCC Teknik

Register of figures:

Figure 1	Source: http://en.wikipedia.org/wiki/Tunnel_boring_machine
Figure 2	Source: Carl Johan Gårdinger
Figure 3	Source: http://www.wirekapning.se/bilderalla.html
Figure 4	Source: Carl Johan Gårdinger
Figure 5	Source: Thomas Krekula, NCC
Figure 6	Source: author
Figure 7	Source: Rolf Christiansson, SKB
Figure 8	Source: Uppländska Berg AB
Figure 9	Source: author
Figure 10	Source: author
Figure 11	Source: Hoek, E. & Brown, E.T. (1980*). Underground excavations in rock*. 2:nd edition. London: Chapman & Hall
Figure 12:	Source: Examiner 2D
Figure 13:	Source: Examiner 2D
Figure 14:	Source: Examiner 2D
Figure 15:	Source: Stille, Håkan & Nord, Gunnar (1990). *Bergmekanik del 2*. Stockholm: Kungliga Tekniska Högskolan, Institutionen för Jord- och bergmekanik. Kurslitteratur
Figure 16:	Source: author
Figure 17:	Source: author

11 Appendices

Appendix 1 Economical comparison

Appendix 2 Blasting journals

Appendix 3 Screenshot from computer simulation

11.1 Appendix 1

Försiktig

Material	Pris/kg	Pris/st	Åtgång totalt	Åtgång/m3	Pris/m3
SSE	9		0	0	0
Sprängkapsel		19	1641	2,600633914	49,41204437
Nonel		1,1	10000	15,84786054	17,43264659
Block		22	45,58333333	0,072239831	1,589276281
Gurit	37,24		75,14	0,119080824	4,434569889
Rocksplit	37,24		57,14	0,090554675	3,372256101
Dynorex	37,24		290,9	0,461014263	17,16817116
NobelPrime	221		0	0	0
Kemix	93		238,845	0,378518225	35,20219493
Borrning					190
Laddning					118
Summa					**436,6111593**

Salvarea	
Längd	10
Volym	631

Grov

Material	Pris/kg	Pris/st	Åtgång totalt	Åtgång/m3	Pris/m3
SSE	9		5968	2,354612168	21,19150951
Sprängkapsel		19	917	0,361792788	6,874062969
Nonel		1,1	2100	0,828533102	0,911386412
Block		22	25,47222222	0,0100498	0,221095593
Gurit	37,24		0	0	0
Rocksplit	37,24		0	0	0
Dynorex	37,24		112,8	0,044504064	1,657331334
NobelPrime	221		110,7	0,043675531	9,652292275
Kemix	93		0	0	0
Borrning					99
Laddning					32
Summa					**171,5076781**

Salvarea	
Längd	40,6
Volym	2534,6

Wiresågning		Kostnad	Kostnad/m3
Försiktig		1570	
Omkrets	32		
Längd	10		
Mantelarea	320		
Tvärsnittsarea	64		
Total area	384	602880	**955,4358162**
Grov		1570	
Omkrets	32		
Längd	40,6		
Mantelarea	1299,2		
Tvärsnittsarea	64		
Total area	1555,2	2441664	**963,3330703**

Appendix 1

Försiktig Salva nr	Specifik borrm [m/m3]	Salvlängd [m]	Specifik laddn [kg/m3]
1	3,6	2	0,9
2	5	1,5	1,2
3	2,3	2	0,5
4	1,4	2,4	0,3
5	9	1	3
6	5	1,5	1,2
7	3,6	1,5	0,9
8	11,7	2,8	1,2
9	5,5	1,5	1,4
10	5,6	2,3	0,4
11	3,4	2,3	0,2
12	9,5	1,5	2,7
13	3,6	2,8	0,4
14	4	1,8	0,9
15	3	1,8	1
16	10,2	1,8	2,9
17	6	1,5	1,7
18	6	1,5	1,7
19	3	1	2,6
20	5,9	2,4	1,6
	Totalt		1,335

Stor salva
40,6 meter lång, tunnelarea 62,4m

	Specifik borrm [m/m3]	Salvlängd [m]	Specifik laddn	Total laddn [kg]
1	2,2	5,8	2,7	1032,4
2	2,2	5,8	2,4	826,8
3	2	5,8	2,3	857,9
4	2	5,8	2,3	857,9
5	2	5,8	2,3	857,9
6	2,2	5,8	2,6	879,3
7	2,2	5,8	2,6	879,3
	Totalt			

Appendix 1

Total laddn [kg]	Max samv laddn [kg]	Salvarea [r	Salvvolym	Kemix 17 [l	DynoRex 2	Rocksplit b	SSE [kg]	NobelPrim	Gurit 17mn	Antal hål [st]
17,32	0,44	10	20	10,12	7,2					36
37	0,37	20	30	22	15					100
9,2	0,42	20	40	10,12	9,2					46
9,585	0,56	20	48	2,565	5,4	7,02				27
29,7	0,31	10	10	9,9	19,8					90
37	0,37	20	30	22	15					100
26,27	0,37	20	30		10,65				15,62	71
48,4	0,69	15	42	16,72	8,8	22,88				176
16,5	0,37	20	30	24,2	16,5					110
13,44	0,17	20	46		5,6	13,44				112
8,04	0,17	20	46		3,35	8,04				67
19	0,42	10	15	20,9	19					95
17,28	0,3	20	56		3,6	5,76			11,52	72
33,6	0,42	20	36		33,6					80
37,2	0,62	20	36		37,2					60
30,6	0,52	10	18	22,44	30,6					102
12	0,42	10	15	13,2	12					60
12	0,42	10	15	13,2	12					60
48	0,85	20	20		3				48	60
51,48	0,64	20	48	51,48	23,4					117
513,615			631	238,845	290,9	57,14	0	0	75,14	1641

Max samv laddn	Salvarea [m2]	Salvvolym	Kemix 17 [l	DynoRex 2	Rocksplit b	SSE [kg]	NobelPrim	Gurit 17mn	Antal hål [st]
8	65	377				933,1	99,275		143
8	59	342,2				826,8			129
8,3	65	377				854,7	3,225		129
8,3	65	377				854,7	3,225		129
8,3	65	377				854,7	3,225		129
8,6	59	342,2		56,4		822	0,875		129
8,6	59	342,2		56,4		822	0,875		129
		2534,6	0	112,8	0	5968	110,7	0	917

11.2 Appendix 2

SPRÄNGJOURNAL
Citybanan - Norrströmstunneln - B4-9523

NCC

Salva nr:	*Arbetsområde/tunneldel:*	*Datum/tid:*	*Sektion:*
T71	**09.01 Arbetstunnel Vattugaraget**	**2010-01-13 01:30**	**00/004**

Ansv. sprängarbas:	*Ansv. arbetsledare:*	*X:*	*Y:*	*Z:*
Lars-Åke Harrysson	Hans Eriksson	100 360	78 944	-24

Ant. hål:	*Salvlängd:*	*Salvarea:*	*Salvvolym:*	*Ant. borrm.:*	*Specifik borrn.:*
110 st	1,5 m	20 m²		165	5,5 m/m³

Max samv. laddn.:	*Specifik laddn.:*
0,37 kg	1,4 kg/m³

Laddmängder:

Håltyp	*Antal hål*	*Sprängämne*	*Laddmängd/hål (kg)*	*Total laddning (kg)*	*Noteringar*
	110	Kemix 17	0,22	24,2	
			Summa:	**24,2**	
Stross					
Stross	110	DynoRex 22mm	0,15	16,5	
			Summa:	**16,5**	

11.3 Appendix 3