Design of Shotcrete for Dynamic Rock Support by Static Testing

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Preface
This thesis is the final part of my studies for a Master’s degree in Civil Engineering with specialization in Soil and Rock Engineering. The studies have been conducted at the department of Civil, Environmental and Natural Resources Engineering at Luleå University of Technology. The originator and client of the work described in this thesis was Lars Malmgren, LKAB. The practical work was supervised by Jimmy Töyrä, LKAB, while the academic work was supervised by Andreas Eitzenberger, Luleå University of Technology.

Several persons at LKAB and the university have been a valuable support during the work with this thesis. Everyone is not mentioned here due to the risk of forgetting someone. However, Bengt-Göran Mikko and Benjamin Krutrök at LKAB Berg & Betong deserve special thanks for their help with both the preparation and the performing of the tests.

Since this is the final work of my studies, I want to thank my family and all my friends for their support during my studies. The past years would have felt much longer without all of you.

Malmberget, October 2013

Fredrik Thyni
Abstract
The Swedish mining company Luossavaara-Kiirunavaara AB (LKAB) operates two underground iron ore mines in the northernmost part of Sweden. Both mines use the mining method sublevel caving, with production levels at a depth of around 900 meters. At these depths, high in-situ stresses are present in the rock mass, which in combination with a large-scale mining method can induce seismic events in the rock mass. For this reason, LKAB installs a dynamic rock support system capable of withstanding seismic events. The dynamic rock support consists of steel fiber reinforced shotcrete with an external steel mesh and dynamic rock bolts. The installation of the dynamic rock support requires significant amounts of support material annually. In order to obtain a faster and cheaper installation of the rock support system, LKAB want to reduce the amount of support material used. However, the strength of the support system must be preserved even if the quantity of material is reduced.

This thesis presents results from static tests performed on circular panels of shotcrete with and without underlying steel mesh as well as panels with embedded steel mesh. All tests were based on the test standard ASTM C1550. The purpose of the static tests was to investigate the effect of the shotcrete layer thickness on the strength of the dynamic rock support used by LKAB. The installation of dynamic rock support would be faster and more material-efficient if the thickness of the shotcrete layer could be reduced. Also, a literature review was conducted to study the design of dynamic rock support in other mines in the world.

The analysis of the test results indicate that the shotcrete layer has a critical role in the load bearing and energy absorbing capacity of the dynamic rock support system. Thus, a reduction in thickness of the shotcrete layer would result in lower strength of the support system. For example, a halving of the currently prescribed thickness of 100 mm to 50 mm when external steel mesh is installed would result in a strength corresponding to approximately 20% of the current strength.

The results also indicate that embedded mesh give a higher peak load capacity and higher residual strength to the shotcrete, compared to the external mesh. However, a weakness with embedded mesh is the fact that wire failure occur at smaller deformations than what is observed for external mesh. Due to the small scale of the tests, there are uncertainties regarding the relationship between these panel tests and the real support installed in the mine. Hence, further tests on a larger scale are required to obtain a better understanding of the interaction between shotcrete and steel mesh.
**Sammanfattning**


Analysen från testerna indikerar att sprutbetonglagret har en avgörande roll på bärformågan samt den energiabsorberade förmågan hos dynamisk bergförstärkning. Således skulle en minskning av tjockleken på sprutbetongskiktet medföra en sänkning av hållfastheten för förstärkningssystemet. Som exempel skulle en minskning från nuvarande föreskriven tjocklek (100 mm) till 50 mm ge en hållfasthet motsvarande ca 20 % av den nuvarande hållfastheten då utanpåliggande nät används. Testresultaten indikerar även att ingjutna stålnät ger högre maximal- och residual hållfasthet jämfört med utanpåliggande nät. En svaghets hos ingjutna nät är dock att stora deformationer kan orsaka brott hos stålvajrarna i nätet. På grund av de relativt småskaliga testerna finns osäkerheter kring sambandet mellan dessa plattester och bergförstärkningen som finns installerad i gruvorna, så ytterligare tester i större skala erfordras för säkrare slutsatser.
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1 Introduction

In the northernmost part of Sweden lies a great iron ore field where the state-owned mining company Luossavaara-Kiirunavaara Ltd. (LKAB) operates two underground iron ore mines. The largest mine regarding production is the Kiirunavaara mine and is located in Kiruna. Here, a large continuous iron ore body is being mined since more than 110 years ago. The second mine, located in Malmberget, extracts iron ore from several ore bodies where mining activities started already in the 18th century.

The production in these mines was 26.3 Mt refined iron ore products in 2012. The mining method used to extract the ore is large-scale sublevel caving, where nearly vertical slices with a height between 20-30 meters are mined from each sublevel.

The long operating time of the mines and the large production volumes has resulted in a continuous lowering of the production and main haulage levels. Present mining is performed at a depth of about 900 meters in both mines. In order to meet future production plans, new main haulage levels are being constructed. These main levels are located on the 1250 and 1365 meter levels in the Malmberget and the Kiirunavaara mines respectively.

Along with the increasing depths follows several challenges in the mining process. Longer transportation distances, increased demands for ventilation capacity and increased volumes of permeating water are some of the problems which need to be solved in order to maintain an effective mining operation. Another consequence from the increased depth is the higher rock stresses around the drifts and excavations.

Redistribution and concentration of high stresses in the rock mass can induce seismic events close to the mines. The energy released in these events can cause serious damage to the excavations underground as rockbursts, a phenomenon that has become more prevalent at LKAB since the year 2007. Determination of the location and magnitude for possible seismic events and the related rockbursts is a difficult task. The safety for miners and equipment is therefore maintained by the use of well-designed rock support and rock reinforcement systems. Since the difficulties related to high rock stresses will increase as the mining continues deeper, it will be more important to understand the behavior of the rock support installed underground.
1.1 Objectives
At certain areas in the mines, LKAB installs a dynamic rock support system. This support system consists of steel fiber reinforced shotcrete covered with welded steel mesh and rock bolts capable of withstanding dynamic loads.

The aim with this thesis is to determine the effect of the shotcrete layer thickness on the strength of the dynamic rock support. This is important in order to design a cost-effective support system that can still maintain safe and accessible production areas. A comparison is also performed between the capacity of shotcrete reinforced with embedded steel mesh and external steel mesh.

Based on the importance of the shotcrete thickness, the main purpose of the thesis is to give a recommended thickness of the shotcrete layer used in the dynamic rock support. The best suited thickness is considered to be the one which gives most advantages regarding safety for underground personnel and an effective usage of shotcrete.

1.2 Delineation
During the work only plain shotcrete, fiber-reinforced shotcrete and shotcrete combined with welded steel mesh was considered in the laboratory tests. Thus, no consideration was taken to the interaction between rock, shotcrete, rock bolts and other elements in the support systems of the mines. All tests involving steel meshes used the same mesh quality and dimensions. The shotcrete used in the samples was also of the same quality, with the single exception that some samples did not have steel fibers in the shotcrete.

1.3 Method
The work in this project consisted of an initial literature study covering the function of the rock support elements and common practices for the design of dynamic rock support systems, presented in sections 3 and 4. The second part of the work consisted of practical laboratory tests on cast round determinate concrete panels according to ASTM C1550, with and without steel mesh presented in sections 5 and 6. Finally, the results from the ASTM C1550 tests were compared in order to evaluate the influence of the shotcrete thickness on the load and energy absorbing capacity for dynamic rock support, see section 7.
2 Rock mass properties and seismicity in LKAB’s mines

2.1 Mining method and general information

LKAB uses a mining method called large-scale sublevel caving in both Kiirunavaara and Malmberget. The orebody is divided in horizontal sublevels with thicknesses between 20-30 m (Figure 1). Cross-cuts, i.e. production drifts, are driven horizontally through the ore on each sublevel with a spacing of 25 m along the strike of the orebody. Blast holes are drilled from the cross-cuts upwards into the ore in rounds with a fan-shaped pattern. After drilling, the rounds are blasted one at a time. A single round can produce about 10 000 tons of ore (Quinteiro et al., 2001).

In the Kiirunavaara mine, a single continuous orebody is being mined. The orebody contains magnetite and is about 4.5 km long with an average width of 80 m. Due to the large size of the ore, the mining is divided into ten different blocks, numbered according to their y-coordinates along the orebody from north to south. After extraction, the ore is loaded into orepasses which transport the ore down to the main haulage level. Remotely controlled trains then transport the ore to a central crushing facility. After crushing, the ore is hoisted to the concentrating and pelletizing plants at the surface (Taaveniku & Wälitalo, 2008). A new main haulage level on the 1365 meter level was taken into operation during 2013 and will take over the production from the old 1045 meter level.

The Malmberget mine differs from the Kiirunavaara mine since it contains about 20 different ore bodies of varying size, of which 13 are currently being mined. The ore bodies are being mined at different depths. Therefore, three different main haulage levels are currently in operation at the 600, 1000 and 1250 meter levels. The mining method is similar to the one in Kiruna with the difference that trucks, rather than trains, are used to transport the ore to the crushers on the main haulage levels.

Figure 1. Schematic view of the large-scale sublevel caving mining method (from Hamrin, 1986).
2.2 Geology and rock mechanical properties

The Kiirunavaara orebody strikes from south to north and dips 50° to 70° to the east and is surrounded by a footwall constituting of syenite porphyry and a hanging wall constituting of quartz porphyry. Uniaxial compressive strength (UCS) of the ore is between 115 and 190 MPa while the UCS of the side rock ranges between 90 to 430 MPa, but is mainly between 200-300 MPa. The main causes for the large variations are the different degrees of weathering and the distribution of minerals included in the rock mass (Quinteiro et al., 2001).

The dip of the ore bodies in Malmberget varies between 45° to 70° and the width from 20 to 100 m. The ore is mainly magnetite, with exception of a few ore bodies consisting of hematite. Some of the ore bodies are situated very close to each other and the footwall of one orebody can thus be the hanging wall of another. The rock surrounding the ore is made up of different metamorphosed volcanic rocks such as gneisses and fine-grained feldspar-quartz rocks. These rocks are called leptites and are often divided into red or grey leptite, but combinations of these do occur. Some parts of the ore bodies also contain veins of granite. Uniaxial compressive strength of the Malmberget ore is in the interval 85 to 140 MPa, the UCS of the red leptite is around 170 to 220 MPa while the UCS of the grey leptite is around 70 to 160 MPa (Quinteiro et al., 2001).

2.3 Seismicity

The large-scale ore extraction causes significant stress redistributions in the surrounding rock mass. As a consequence, the magnitude of in-situ stresses can either increase or decrease in different areas of the mines. The effect of such changes becomes more evident as the mines become deeper, since the magnitude of the virgin stresses increases with depth (Kaiser & Cai, 2012).

As the stress levels change, the risk for failure in the rock increases. According to Larsson (2004) there are three factors governing the consequence and extent of a failure, namely stress state, mining activities and geomechanical conditions. The last factor involves the properties of the rock mass with its geological discontinuities such as faults, joints and water bearing zones. Failure in the rock occurs due to the influence of all three factors. Different failure modes occur in rock with different properties of these factors, e.g. brittle and strong rock subjected to high stresses around a drift in a mine experience a higher risk for violent failures compared to a drift in a soft rock with low stresses.

A violent failure in the rock is denoted as a seismic event, and can be defined as a failure that rapidly releases the stored energy in the rock mass. A general designation for the presence of seismic events and the corresponding deformations in a rock mass is seismicity (Larsson, 2004).

The released energy from a seismic event is spread out in the rock mass as seismic waves. For seismic events at great depths in rock there are mainly two types of waves emitted, called compressive and shearing waves, respectively. Nordlund et al. (2012) explains that the propagation of these waves in the rock mass occur by movement of particles, as a result of the disturbance generating the seismic event. When a particle moves away from its initial state of
equilibrium, it affects the particle beside it which in turn is disturbed and thereby transmits the energy released by the disturbance. Thus, the propagation of the seismic waves only transmits the energy through the rock mass but no material.

Since the amount of energy released is finite and because the waves propagate in the rock mass with a spherical front, the energy affecting the particles will decrease as the distance from the source increases. This attenuation, dependent only on distance and geometry, is referred to as geometrical damping and results in a low influence on an excavation subjected to seismic energy when the distance to the source is large. Another form of attenuation that affects the seismic waves is the material damping from the rock mass, in which the waves are travelling. This damping occurs when energy in the wave is lost due to friction as the rock deforms at cavities and joints, or local failures are induced in the rock mass (Nordlund et al., 2012).

Kaiser et al. (1996) describes how mining-induced seismic activity changes during the operational life of an underground mine. Beginning with small, local seismic events such as failures in highly stressed areas close to drifts and excavations, the magnitudes of the events increase as larger volumes of rock is subjected to stress redistributions. These redistributions occur when the mine grows in size as more rock is excavated. In time, the extraction of ore can create large volumes of broken rock or open stopes with considerable influence on the virgin stress state at a regional scale. At this stage the induced seismic events might be so large they affect the entire mine. Eventually, the energy released from large seismic events might lead to the initiation of additional, smaller, seismic events at other locations throughout the mine.

2.4 Seismic monitoring at LKAB’s mines

The underground mines in Kiruna and Malmberget are both equipped with their own seismic monitoring systems that detects and registers seismic events. The detection and registration is done by detectors called geophones which are cast inside boreholes in the rock mass. New geophones are continuously added to the monitoring systems, which currently consist of 174 geophones in Malmberget and 219 geophones in Kiruna.

The released energy of a seismic event travels through the rock mass as waves and are registered by the nearby geophones. The location of the seismic event is then determined based on the registration time of each geophone, since their positions are known. The accuracy of the localization is better in areas with many geophones (LKAB, 2013a).

The size, or magnitudes, of seismic events in LKAB’s mines are described according to a local scale. This local scale is similar to the better known Richter scale, but is adapted to seismic events with a higher frequency and smaller magnitude than events causing earthquakes. All registered seismic events in the mines are stored, which makes it possible to use the information to get a better understanding of the rock mass behavior. For instance, increased seismic activity in a certain area of the rock mass can indicate a progressive caving in the rock mass.
The information from the monitoring system can also be used as support for decision making to e.g. temporarily close areas of the mines that are exposed to increased seismic activity due to safety reason for the personnel. The largest seismic event registered by the monitoring system in Kiruna had an magnitude of 3.0 and was located at Block 19. The largest event in Malmberget also had a magnitude of 3.0 and was localized to the hanging wall of one of the ore bodies (LKAB, 2013a).

2.5 Rockbursts
When waves from a seismic event hit an underground excavation, e.g. a drift in a mine, rapid failures might be induced in the excavation boundary due to an increment of stresses caused by the radiated waves. The waves can also cause movement of pre-existing blocks in the rock close to the boundary of the excavation. Both these events may lead to fallouts of rock that damages equipment or injure workers. This type of seismically induced failures at an underground mine or excavation is commonly called rockbursts (Nordlund et al., 2012).

In some cases, the seismic event triggering a rockburst and the rockburst itself might be the very same event. Kaiser et al. (1996) distinguishes between the different phenomenon as remotely triggered rockbursts and self-initiated rockbursts, respectively, and defines the term rockburst as damage to an excavation that occur in a sudden or violent manner in association with a seismic event. Other similar definitions of rockbursts can be found in for example Blake & Hedley (2003). However, as mentioned by e.g. Larsson (2004), these definitions of rockbursts do not explicitly state the extent of damage and the sequence of the failure. Hence, the term rockburst can have various meanings for different mines experiencing varying degrees of failure.

2.5.1 Rockburst source mechanisms
In order to understand the initiation and occurrence of rockbursts, knowledge about the mechanisms in the source seismic event is required. Ortlepp & Stacey (1994) presents the following five common source mechanisms, in ascending order of energy magnitude:

1. Strain bursting
2. Buckling
3. Face crushing
4. Virgin shear in the rock mass
5. Reactivated shear on existing faults and/or shear rupture on existing discontinuities

These source mechanisms can further be divided into two groups, where the first three mentioned can be classed as self-initiated with the seismic event and the rockburst coinciding in location, i.e. are the same event. In these cases the local stress situation and rock properties around an excavation are the main factors initiating failure. Strain burst and face crushing usually takes place in a massive rock surface with high strength as the rock fail in small, sharp pieces as shown in Figure 2 a). Buckling mainly occur in rock with a pre-existing laminated structure and breaks loose plates of rock along the lamination, see Figure 2 b) (Ortlepp & Stacey, 1994).
Mechanisms 4 and 5 can be classed as sources leading to remotely triggered rockbursts. The common characteristics for these mechanisms are the fact that the shearing occur along a plane in the rock mass, which might extend several hundred meters and hence cause large impacts on the underground mining drifts and facilities at large distances. Conditions that make these mechanisms possible are probably only present at large scale mining operations, according to Ortlepp & Stacey (1994).

2.5.2 Rockburst damage mechanisms
The source mechanisms discussed above are the events that release energy triggering a rockburst. However, each source mechanism can induce different types of damage to an excavation depending on e.g. the geometry of the excavation and distance to the triggering event. According to Kaiser et al. (1996), the damage mechanism triggered by the source event is the main cause of damage on the excavation, regardless of the source mechanism present.

Kaiser et al. (1996) identified the following main damage mechanisms for rockbursts in Canadian mines, see Figure 3:

1. Rock bulking due to fracturing
2. Rock ejection due to seismic energy transfer
3. Rockfalls induced by seismic shaking

A similar grouping and description of damage mechanisms is presented in e.g. Ortlepp & Stacey (1994).

The bulking mechanism occurs due to the fact that the total volume of rock increases when the rock fails. This occur either due to a self-initiated failure from stress concentrations, e.g. strain burst and buckling, or because of a sudden stress increase from a remote event. In general it is possible to further divide bulking into two sub-classes. The first sub-class is bulking without ejection of rock material, where the failure takes place in a relatively stable process and falls down due to gravity. The second sub-class is bulking with ejection of rock material which occurs if excess energy remains after the rock has failed. Bulking with ejection causes the loosened rock pieces to move into the empty space of the excavation. Depending
on the amount of energy present, the kinetic energy of the loose rock pieces can be very high and pose a hazard to personnel and equipment. Unfortunately it is very difficult to determine if a rock ejection will occur or not. Hence, underground observations are required to identify the local characteristics of the failures and design the necessary support based on these observations (Kaiser et al., 1996).

![Diagram of rockburst mechanisms](image)

**Figure 3. The three main damage mechanisms of rockbursts (from Kaiser et al., 1996).**

Rock ejection may occur when energy waves from a seismic event reach the excavation boundary, where rock blocks are capable of moving freely into the excavation. The risk for ejection is high for rock masses with pre-existing blocks formed by intersecting joints or fracturing (Kaiser et al., 1996). Since compressional waves are reflected as tensile waves at free surfaces, there is also a risk that blocks are ejected from initially massive rock due to tensile failures (Nordlund et al., 2012). The severity of damages caused by rock ejection depends on the amount of kinetic energy in the released block. The amount of kinetic energy is in turn governed by the mass of the rock block and its velocity at ejection (Kaiser et al., 1996).

Fallout of rock due to vibrations is induced when waves from a seismic event causes the rock to accelerate from the previously stable state. The acceleration can cause failures, e.g. tensile failures in the rock, which creates loose blocks of rock that might fall down due to gravity (Kaiser et al., 1996).

Larsson (2004) predicts a future increase in the occurrence of seismic events and violent failures in the Swedish mines as a result of deeper production levels combined with high stresses and a rock mass of high quality. As the induced failures become more severe, the volume of fallouts from each failure will be larger. Hence, the increased failures create potentially large production losses, equipment damage, hazards for underground personnel and possibly total collapses of mining drifts and areas if adequate rock support is not installed.
2.6 Dynamic loads on rock support systems at LKAB

LKAB categorizes the occurring failure mechanisms of the rock into two groups, based on Kaiser et al. (1996). These groups are:

1. Rock ejection
2. Rock bulking due to fracturing

For category 1 the energy absorbing capacity of the rock support is the governing factor of safety, while the deformation capacity of the rock support is of interest in category 2 (Woldemedhin, 2013). Since the installed support system must be able to sustain varying loading conditions and large deformations, the term dynamic rock support is used.

For the case with rock ejections, the rock support must be designed according to both the magnitude of a seismic event and the distance from the event to the rock support (Kaiser et al., 1996). The radiated energy from a seismic event will attenuate with distance. Therefore, the amount of energy that hit a drift or other excavations will differ from the energy amount at the source. When a rock support system subjected to dynamic loads is designed, it is common to use the ground motion velocity on the excavation boundary represented as peak particle velocity (PPV) in the calculation of the dynamic loads. It has been shown that the ejection velocity of rock pieces at rockbursts seldom exceed the PPV (Kaiser et al., 1996). It should be mentioned however; that an amplification of the PPV might occur at the surface of excavations, but the mechanisms behind this amplification is still not entirely understood (Potvin et al., 2010). In addition, the assumption that the PPV is equal to the ejection velocity is based on observations where the dominant wavelengths of energy waves from a seismic event are longer than the drift dimensions. Thus, it is possible to assume that reflections of the energy waves can be ignored (Kaiser et al., 1996). Following this assumption however, no consideration is taken to the impact of shock waves with high frequencies.

In order to obtain relevant data of loads acting on the installed rock support system, LKAB assumes that the rock support system must be able to withstand a seismic event of a certain magnitude located at a certain distance from the excavation, as described by for example Potvin et al. (2010).

The rock mass properties and rock stress situations differ between different areas of the mines. For example, some areas in the mines experiences larger problems with rockbursts than others. Thus, the rock support system will be subjected to different conditions depending on the location due to the varying rock mass properties in the mines. In order to optimize the rock support system with regard to these differences, LKAB use different magnitudes for the estimated seismic events at different areas of the mines. The differences in magnitude are considered when the dynamic loads on the support system are estimated (Malmgren, 2009).
3 Rock reinforcement and rock support systems in LKAB’s mines

When rock support systems are described in the literature, a distinction is often made between supporting and reinforcing elements. Supporting elements are installed to support the surface of the rock surrounding an excavation, while the principle of reinforcing elements is to create a combined material of rock and reinforcement that mobilizes the inherent strength of the rock mass (Nordlund et al., 2012). Reinforcing elements commonly consists of rock bolts, with stiff or yieldable properties depending on the rock mass and the purpose of the excavation.

Rock support elements can be further described as retaining or holding, see Figure 4. Kaiser et al. (1996) explains that the principle with retaining elements is to prevent pieces of broken rock from falling down on equipment or personnel, as well as to prohibit progressive failures in the rock mass which may lead to a total collapse of an excavation. Some examples of common retaining elements are stiff elements such as shotcrete or cast concrete liners or more ductile elements such as different kinds of steel wire meshes.

The purpose with holding elements is to connect the retaining elements to firm rock and in turn prevent loose blocks from falling down due to gravity. This is accomplished by the use of e.g. cable bolts or conventional rock bolts, where loads from the retaining elements are transferred to the bolts (Kaiser et al., 1996).

![Figure 4. Principal sketch showing the function of reinforcing, retaining and holding rock support elements. (from Kaiser et al., 1996).](image)

The combination of several support elements forms a rock support system, where the different elements are connected to or complement each other in different ways. Throughout the remainder of this thesis, rock support and rock support systems will be used as a general designation for different combinations of both supporting and reinforcing elements if nothing else is explicitly stated.

3.1 Bolts

A variety of rock bolts are available on the market today. The rock bolts vary from simple rebars intended for cement or resin grouting to more sophisticated bolts with built-in indicators of rock movement and ability to sustain large deformations (Rustan, 2010). A
careful decision of appropriate bolts can improve the stability of underground excavations subjected to rockbursts or large deformations in the rock mass. However, the impact from rock bolts on the effectiveness of the support system lies outside the scope of this thesis. Therefore, only a short presentation is given of the bolts that currently are used by LKAB.

LKAB mainly uses three types of bolts as reinforcement elements in the drifts, crosscuts and other excavations underground. The first type is the Kiruna bolt, made of a 3.05 m long, 20 mm diameter rebar with a wedge at the end of the bolt acting as an anchor to achieve an instant support, Figure 5. During installation of the bolt, the percussion equipment of the rig activates the anchor. The whole bolt is grouted with cement in order to improve the adherence between bolt and rock. Full capacity of the bolt is achieved when the grout has cured. The outer end of the bolt is equipped with a face plate which is tightened to the rock surface by a steel nut. If the bolts must be capable to deform during loading, the Kiruna bolt can be modified by mounting a metal or plastic sleeve around the middle part of the bolt as shown in Figure 6. This creates a section of the rebar that is not in contact with the grout and instead has possibility to yield freely, increasing the deformation capacity of the bolt.

![Figure 5. The end anchor of a Kiruna bolt a) before installation and b) after installation with the anchor expanded (from Malmfalten, 2013).](image)

The second bolt type is the Swellex bolt, which consists of a hollow plastic covered steel bolt with an initial diameter of 37 mm and a length of 3 m. The bolt is inserted into a drilled borehole after which it is inflated by highly pressurized water until it fills the entire borehole. A face plate on the end of the bolt provides support to the rock surface, Figure 7. The Swellex bolt transfers the load to the rock by friction between the bolt and the rock. Although this bolt type is fast and easy to install, it lacks the corrosive protection of the concrete grout and is hence more vulnerable to corrosion compared to the Kiruna bolt.

![Figure 6. Illustration of a modified Kiruna bolt. 1: Wedge. 2: Rebar. 3: Plastic tube. 4: Shrink tube with glue. 5: Washer. 6: Thread. (from Malmfalten, 2013).](image)
The third bolt used in LKAB’s underground mines is the D-bolt. The D-bolt is fully grouted inside the borehole, and has three points where the bolt is flattened as shown in Figure 8. These flattened points act as anchors, allowing the metal between the anchors to elongate during loading. The surface of the bolt is smooth in contrast to the rebar of the Kiruna bolt because of the anchor points. An advantage of the D-bolt is that the energy absorbing capacity does not depend on a sleeve around the bolt. This eliminates potential problems during bolt installation where the metal or plastic sleeve can be damaged, leading to a loss of its function. Like the Kiruna bolt, the D-bolt is equipped with a wedge at the end of the bolt acting as an initial anchor until the cement grout has cured. LKAB uses D-bolts of diameter 20 mm and a length of 3 m.

In mining areas with large roof spans, large rock blocks or wedges can be formed by intersecting joints. Standard bolts are ineffective in these areas due to the large size of the blocks or wedges and the limited length of the bolts. In these areas LKAB installs cable bolts from 7 m length which are fully grouted and have a high load capacity. The total diameter of the wire is 15.2 mm and if required, they can be installed double folded to further increase the load bearing capacity. After the grout has cured, a face plate is installed at the end of the wire together with a wire lock.

### 3.2 Shotcrete

Shotcrete is a common surface support, used to create a stabilizing surface that fixes loose pieces in the rock surface and enhances the ability of the rock to support itself (i.e. creating confinement). The use of shotcrete in LKAB’s underground mines began more than 30 years ago (Malmgren, 2001).

The components of shotcrete are cement, sand and fine-grained rock material which is applied to and compacted on the rock surface by the use of pneumatics. Chemicals are added in order to improve certain properties, for example the addition of accelerators which makes the shotcrete cure rapidly and hence adhere to the rock surface during application (Hoek et al., 1995).
Holmgren (1992) describes some of the properties for plain shotcrete rock supports. The strength of a plain shotcrete support is heavily dependent on the adhesion between the shotcrete and the rock surface. When shotcrete is applied to a rock surface, it penetrates open joints in the rock and thereby seals the joints. This prevents clay minerals in the joints to be flushed out by water or dried out by air, which could lower the strength of the rock mass.

Plain shotcrete is only able to support rock subjected to small displacements and static loads. Due to its brittleness, cracks develop in plain shotcrete at small deformations. The cracks eliminate the post crack strength of the shotcrete and its ability to yield during loading (Holmgren, 1992). The shotcrete can therefore be reinforced in order to increase its ductility. The use of reinforcement also distributes the location and reduces the size of cracks in the shotcrete (Malmgren, 2001).

When rock bolts are installed after the shotcrete layer, the shotcrete and rock bolt can interact. If the shotcrete is reinforced, loads on the shotcrete layer can be transferred to the face plate of the rock bolt by bending in the shotcrete layer. Because of the bolts and reinforcement, the need for adhesive strength for the shotcrete is reduced (Holmgren, 1992).

Reinforcement in shotcrete originally consisted of wire mesh that was installed manually on the rock surface prior to the application of shotcrete. Mesh used for reinforcement must have large openings in the grid in order to allow the shotcrete to penetrate the mesh and cover it properly. Because of this, welded wire mesh with large openings is often used instead of chain-link meshes. Mesh reinforced shotcrete is relatively stiff and is today mainly used in areas with poor rock quality where a stiff rock support is required (Hoek et al., 1995).

Another reinforcement material for shotcrete is fibers that are added to the shotcrete during the mixing phase. Fibers can be manufactured from a number of different materials, including steel, polymer materials, glass or carbon fibers. Compared to reinforcement with wire mesh, fibers are randomly distributed in the shotcrete layer and therefore give a better distribution of cracks in the shotcrete (Melbye et al., 2001). Compared to steel mesh, steel fibers give a faster application of the rock support since they are added to the rock surface simultaneously as the shotcrete. Reinforcing with steel mesh requires two separate steps where the first is the installation of the mesh and the second is the application of shotcrete. Hence, the support application is more effective when fibers are used and underground personnel are not exposed to unsupported rock since no mesh must be installed prior to the shotcrete (Nilsson, 2000).

LKAB installs mesh reinforced shotcrete in mining areas with poor rock. The steel meshes are welded wire meshes that can be installed either manually or by machine handling. All new-developed drifts in LKAB’s underground mines are supported with a steel fiber reinforced shotcrete. LKAB exclusively uses steel fibers, with a dosage of 40 kg steel fibers per m³ shotcrete (LKAB, 2010).

Depending on the rock conditions, LKAB applies a 70-150 mm thick layer of shotcrete, where thicker layers are applied in areas with poor rock or where high loads are expected. When the shotcrete is reinforced by embedded steel meshes the thickness of the shotcrete is increased to a minimum of 100 mm (LKAB, 2013b).
3.2.1 Load bearing capacity and failure mechanisms

The energy absorption of shotcrete is a useful parameter when the capacity of shotcrete is evaluated, especially when designing a dynamic rock support. As mentioned by Malmgren (2005), the advantage with using energy absorption as a design parameter is that it includes both load capacity and displacement before and after cracking of the shotcrete. The energy absorption capacity is often referred to as toughness, which is a measure of the total amount of energy absorbed before and after cracking of the shotcrete (Cengiz & Turanli, 2004).

Several studies, e.g. Hoek et al. (1995), Kirsten & Labrum (1990), Ortlepp & Stacey (1998) and Melbye et al. (2001) present static and dynamic test results on shotcrete. The tests show that shotcrete reinforced with steel fibers has a similar load-carrying capacity as shotcrete reinforced with steel mesh. However, differences between the fiber type and quantity used, as well as differences in steel mesh type and dimensions makes a direct comparison difficult. Hence, it is difficult to determine which one of the two reinforcement types that gives the highest strength or energy absorption capacity to the shotcrete support without practical tests.

Cengiz & Turanli (2004) performed tests on shotcrete panels reinforced with only steel fibers or steel mesh, respectively. The panels were prepared and tested according to the specifications of a panel test specified by the European Federation of National Associations Representing producers and applicators of specialist building products for Concrete (EFNARC). The EFNARC panel test uses a panel with dimensions of 600x600x100 mm (Figure 9). The panels were supported along its four edges by a metal frame and loaded with a point load through a contact surface of 100x100 mm. A detailed description of the EFNARC panel test can be found in EFNARC (1996).

Figure 9. Dimensions of the tested EFNARC panels (from Cengiz & Turanli, 2004).

The steel mesh used by Cengiz & Turanli (2004) had a wire diameter of 8 mm and a wire spacing of 150x150 mm, while the fiber content in the steel fiber reinforced panels were 35 or 50 kg/m$^3$. The average cross-sectional area per meter for the steel mesh used in the tests was thus 335 mm$^2$/m. These values can be compared to the corresponding values for LKAB’s mesh, where the average cross-sectional area for the mesh was 317 mm$^2$/m and the fiber dosage was 40 kg/m$^3$. All panels were tested until a deflection of 25 mm after 28 days of curing with three panels for each fiber dosage and three panels with steel mesh. The results from the tests are shown in Figure 10 and Figure 11. “SMRS” stands for Steel Mesh
Reinforced Shotcrete and “SFRS” stands for Steel Fiber Reinforced Shotcrete (Cengiz & Turanli, 2004). As seen in the figures, the panels reinforced by the steel mesh had the highest residual strength and the highest toughness in the test. These results are in contrast to the studies mentioned above, and show the influence from steel mesh dimensions and fiber dosage.

When fiber reinforced shotcrete fails in flexure, the steel fibers are either pulled out of the shotcrete matrix or ripped apart. Based on this, the design of the shotcrete mix should aim at creating failure through pull-out of fibers, since this gives a more yielding behavior during cracking (Malmgren, 2005).

Malmgren et al. (2004) performed an extensive failure mapping in the Kiirunavaara mine, where failures in plain shotcrete were mapped in drifts. Their results showed that a shotcrete
thickness thinner than 20 mm did not provide any considerable support to the rock. The mapping also showed that failures in the shotcrete often occurred close to apexes on the rock surface, since these blocked the sprayed shotcrete and led to thin shotcrete layers around the apexes (Malmgren et al., 2004). Apexes on the rock surface are important to consider when shotcrete is applied to the rock.

Results from profile scanning by e.g. Hirscher (2012) and thickness measurements in the Malmberget mine by e.g. Savilahti (2011) and Thyni (2011) indicate large differences in the thickness of the shotcrete layer. The problem with apexes arises when the shotcrete spraying rig cannot be positioned under unreinforced rock due to safety risks. In these cases, the spraying crew might have difficulties to see behind apexes and obstacles in the wall or roof of a drift, causing some parts of the rock surface to be poorly covered by the shotcrete.

3.2.2 Calculation of peak load

Laboratory tests are often used as a quality control of shotcrete manufacturing and to estimate the load bearing capacity. One such test method is specified by the American Society for Testing and Materials (ASTM) International. The test is designated ASTM C1550 and uses round panels of shotcrete supported on three pivots. The panel is subjected to a central point load. The load carrying capacity is measured in order to obtain the flexural toughness of the panel.

The preparation and execution of laboratory tests on shotcrete panels are time consuming and expensive, especially if the tested volumes are large. Therefore, the design of shotcrete rock supports can be simplified by the use of calculation models that can predict the properties of a shotcrete support with different characteristics. There are existing models to calculate the point load carrying capacity of shotcrete panels at cracking and post-crack loads. For example, Bernard (2004) describes a model which uses yield line theory to estimate the load carrying capacity of the shotcrete panels tested in the ASTM C1550 standard test method, using the equation

\[ P = 3\sqrt{3}m' \frac{R}{r} \]  

(3.1)

Where \( P \) is the central point load capacity of the C-1550 panel, \( R \) is the radius of the panel, \( r \) is the radius of the supports (Figure 12) and \( m' \) is the moment capacity.
Figure 12. Schematic sketch of the point loaded panels with load, supports and the radii for the panel and the supports.

The moment capacity of a rectangular section of unit width is obtained from

\[ m' = \frac{\sigma d^2}{6} \]  \hspace{1cm} (3.2)

where \( \sigma \) is the tensile stress in the outer fiber and \( d \) is the height of the section (Bernard, 2004). It is difficult to analytically determine the value of the tensile strength of shotcrete which will be equal to \( \sigma \) in equation (3.2) at cracking of the shotcrete, since many parameters influence the value. The contribution from the fibers depends on e.g. the added volume of fibers in the shotcrete mix, the bond strength between fiber and the shotcrete matrix (Malmgren, 2001). In the analysis of the performed tests in this thesis, the value of \( \sigma \) was therefore determined from the obtained test results.

Equation (3.2) is useful for homogeneous materials subjected to loading, such as fiber reinforced shotcrete. However, equation (3.2) is not suitable for calculating the moment capacity of the mesh reinforced panels where the steel mesh is located at a specific position inside the panels. The moment capacity of the mesh reinforced panels can instead be estimated by formulas used for reinforced concrete beams, such as the equations

\[ M_u = A_s f_{st} (d - 0.4x) \]  \hspace{1cm} (3.3)

\[ A_s f_{st} = F_s \]  \hspace{1cm} (3.4)

\[ F_s = F_c \]  \hspace{1cm} (3.5)

\[ z = d - 0.4x \]  \hspace{1cm} (3.6)

\[ F_s \cdot z = M_u \]  \hspace{1cm} (3.7)

where \( M_u \) denotes the moment capacity of the reinforced concrete beam, \( A_s \) is the total cross sectional area of the steel reinforcement and \( f_{st} \) is the tensile strength of the reinforcement steel. Further, \( d \) is the distance between the upper edge of the compressed part of the beam and the center of the reinforcement layer, \( x \) represents the thickness of the compressed part of the beam during loading, \( F_s \) is the tensile stress in the steel reinforcement and \( F_c \) is the compressional stress in the concrete, see Figure 13 (Isaksson et al., 2005).
Figure 13. Sketch over stresses and moment in a reinforced concrete beam.

The thickness of the compressed part of the beam, $x$, is determined by

$$x = \frac{A_{sf}}{f_{cc}^{0.8}}$$

(3.8)

where $f_{cc}$ is the compressional strength of the concrete. A drawback when equations (3.3) - (3.8) are used for modeling of the panels reinforced with both steel fibers and mesh is that the contribution from the steel fibers as reinforcement is not considered.

### 3.2.3 Calculation of energy absorbing capacity for shotcrete

The energy absorbing capacity, or toughness, of a shotcrete support is a useful parameter in the design of the support. The energy absorbed by an ASTM C1550 panel is determined by integration of the area under the load-deflection curve (ASTM, 2012). However, Bernard (2004) states that the energy absorbed by an ASTM C1550 panel also can be determined for a specific deflection interval by multiplying the deflection with the average load capacity of that interval. The relationship used for this calculation is

$$W_\gamma = P\gamma$$

(3.9)

where $W_\gamma$ is the post-crack energy absorption of a deflection $\gamma$ with corresponding average load capacity $P$, as shown in Figure 14.

Figure 14. Schematic load-displacement diagram of an ASTM C1550 panel test. The energy absorption $W_\gamma$ can be determined by the deflection $\gamma$ and the average load capacity $P$ (from Bernard, 2004).
Equation (3.9) requires knowledge of the average residual load capacity of the panel for which the toughness is determined. Since no relations for calculating the residual load of fiber-reinforced panels were found, the relation between panel thickness and residual load capacity was determined empirically.

3.3 Steel mesh

Steel wire mesh is often used in underground mines as retaining support on the rock surface instead of shotcrete. There are three main types of steel meshes. The most commonly used is the welded wire mesh, where steel wires are welded together forming a square pattern of usually 100x100 mm with a wire diameter ranging between 4-6 mm, see Figure 15a. The dimension of the wire in a mesh is often designated as X-gauge or #X, where the X stands for a specific wire gauge specified in Table 1. Other standards also exist for wire dimensions with varying diameters for each gauge.

Table 1. Wire gauges according to the (British) Imperial Standard Wire Gauge (SWG) and the American Wire Gauge (AWG) systems. The corresponding wire diameters are given in millimeters. From AMTEX (2013).

<table>
<thead>
<tr>
<th>Wire Gauge</th>
<th>SWG Diameter (mm)</th>
<th>AWG Diameter (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>8.23</td>
<td>8.25</td>
</tr>
<tr>
<td>1</td>
<td>7.62</td>
<td>7.35</td>
</tr>
<tr>
<td>2</td>
<td>7.01</td>
<td>6.54</td>
</tr>
<tr>
<td>3</td>
<td>6.40</td>
<td>5.83</td>
</tr>
<tr>
<td>4</td>
<td>5.89</td>
<td>5.19</td>
</tr>
<tr>
<td>5</td>
<td>5.38</td>
<td>4.62</td>
</tr>
<tr>
<td>6</td>
<td>4.88</td>
<td>4.12</td>
</tr>
<tr>
<td>7</td>
<td>4.47</td>
<td>3.66</td>
</tr>
<tr>
<td>8</td>
<td>4.06</td>
<td>3.26</td>
</tr>
<tr>
<td>9</td>
<td>3.66</td>
<td>2.91</td>
</tr>
<tr>
<td>10</td>
<td>3.25</td>
<td>2.59</td>
</tr>
</tbody>
</table>

The second type of wire mesh is the chain link mesh, in which the wire is woven to form a diamond shaped pattern, see Figure 15b (Player et al., 2008). A third mesh type is the expanded metal mesh, but its use in mines seems to be limited.

![Figure 15. An example of a) welded wire mesh and b) chain link mesh (from Player et al., 2008).](image)
Welded wire mesh is generally stiffer during initial loading than chain link mesh. A chain link mesh will only be able to prevent loosening of rock if a sufficient amount of pre-loading is generated in the mesh at the installation (Kaiser et al., 1996).

### 3.3.1 Load bearing capacity and failure mechanisms

Player et al. (2008) performed static and dynamic tests on welded wire mesh where the diameter was 5.6 mm and the wire spacing was 100x100 mm as well as on chain link mesh with wire diameter 4 mm. In the tests, rupture was defined as failure of a wire or a weld in the mesh. The average static rupture loads were 44 kN for the welded wire mesh and 145 kN for the chain link mesh. The corresponding average displacements were 186 mm for the welded wire mesh and 307 mm for the chain link mesh. Force-displacement curves for the two mesh types are shown in Figure 16. The results from the dynamic tests are not presented since this thesis focus on the static properties of the rock support.

![Figure 16. A typical force-displacement response during static tests of welded wire mesh and chain link mesh (from Player et al. 2008).](image)

Kaiser et al. (1996) presents results from static pull tests performed on welded wire meshes of three different dimensions, where typical load-displacement curves are shown in Figure 17. The wire spacing was 100x100 mm, and the meshes were held by four bolts in a 1.2 m diamond pattern during the tests. Unfortunately, the wire gauge system used in the tests was not specified. A majority of the tested meshes experienced a drop of load carrying capacity after the initial peak load, which was caused by failure of a mesh wire. Through redistribution of the load to the remaining mesh, the meshes were able to regain their loading capacity and were in some cases even able to carry higher loads than the initial peak load (Kaiser et al., 1996).
Kaiser et al. (1996) also report results from more than 200 pull tests of different mesh types. These results showed that the peak displacement of welded wire meshes in general were in the interval 100 to 200 mm. Meshes with heavier gauge were able to sustain the larger displacements. The results also showed that chain link mesh had typical peak displacements in the interval 400 to 450 mm.

In addition, Kaiser et al. (1996) studied the energy absorbing capacity and corresponding displacements during pull tests for welded wire mesh, chain link mesh and expanded metal mesh. The results showed that welded wire mesh absorbed energy already at small displacements; while the other two mesh types underwent large deformations before any significant energy absorption occurred (Figure 18). Again, the wire gauge system used was not specified. For large displacements, chain link and expanded metal mesh showed the highest energy absorbing capacity.
An important factor to consider when results from pull tests on welded wire mesh are examined is the position of holding points on the mesh in relation to the wire direction in the mesh. As stated by Kaiser et al. (1996), welded wire mesh is stiffest when the wires are oriented at 90° angle between the holding points and softest when the wire is oriented at an angle of 45° relative to the holding points, see Figure 19. The different loading directions can be obtained by varying the point of loading on the mesh in relation to the holding points, i.e. bolts (Kaiser et al., 1996).

![Figure 19. Schematic sketches of welded wire meshes with the wire orientation a) 90° in relation to the four holding points and b) 45° in relation to the four holding points.](image)

The results presented by Kaiser et al. (1996) show that the initial stiffness is similar for all tested dimensions of the welded wire mesh (Figure 17). With larger dimensions, i.e. smaller gauge number, the peak loads and corresponding displacements increases. When the different mesh types were compared in Figure 18, it was clear that the welded steel mesh had a better initial energy absorbing capacity than the chain-link and the expanded metal mesh. Figure 16 shows that the peak load was much smaller for the welded steel mesh, but on the contrary the chain-link mesh allowed larger displacements. Large displacements in the rock support can be unfavorable since the function of certain underground excavations, for example transportation drifts, can be disturbed by excessive deformations of the rock mass.

### 3.4 Fiber-reinforced shotcrete with external welded steel mesh

Some areas in LKAB’s underground mines are subjected to large deformations in the surrounding rock while other areas are subjected to a higher risk of rockbursts. In these areas it has been determined that to only use fiber reinforced shotcrete is inadequate. Shotcrete reinforced with embedded steel mesh is also not useful since it eventually fails due to large deformations.

In order to obtain an initially stiff support that is able to undergo large deformations, LKAB has designed a dynamic rock support system consisting of a 100 mm thick layer of fiber reinforced shotcrete, covered by welded steel mesh which is bolted in a 1x1 m pattern, see Figure 20. The welded steel mesh consists of stress-relieved, annealed steel with a wire diameter of 5.5 mm and a wire spacing of 75x75 mm (LKAB, 2010).
3.4.1 Load bearing capacity and failure mechanisms

When the mesh used by LKAB is compared with the mesh tested by Player et al. (2008) (see section 3.3.1), it is obvious that LKAB’s mesh contain more steel and hence should be stronger due to the larger total steel area in a cross section of the mesh, see Table 2.

Results from static tests on mesh with specifications commonly used in Canadian mines showed that the failure load was in the interval 24-28 kN, with a corresponding deformation of around 125-175 mm (Malmgren, 2009). Since LKAB’s mesh contain significantly more steel than the Canadian mesh, it is likely that LKAB’s mesh has a higher breaking load. However, no full scale tests have so far been conducted on LKAB’s mesh and hence there is no available data for the breaking load.

<table>
<thead>
<tr>
<th>Mesh Type</th>
<th>Wire Diameter</th>
<th>Wire Spacing</th>
<th>Wire Area / Meter Mesh</th>
</tr>
</thead>
<tbody>
<tr>
<td>LKAB’s mesh</td>
<td>5.5 mm</td>
<td>75x75 mm</td>
<td>317 mm²/m</td>
</tr>
<tr>
<td>Canadian mesh</td>
<td>4.9 mm</td>
<td>100x100 mm</td>
<td>189 mm²/m</td>
</tr>
</tbody>
</table>

The rock support system with external mesh has proved to be useful in areas of the mines experiencing large deformations. In these areas, loose rocks between bolts are carried by the steel mesh and smaller pieces of rock are held together by the shotcrete. However, the dynamic properties of this support system are influenced by the installed rock bolts. LKAB believe that also the bolts should be able to sustain large deformations in order to make the whole support system more durable to dynamic loads. This reasoning is also mentioned by Ortlepp & Stacey (1998), who states that the use of yieldable retaining support elements, i.e. shotcrete and mesh, provides limited effect if the holding elements, i.e. bolts, lack the same ability to yield. Also, the connections between the elements, like plates and nuts, must be properly matched to the capacity of the total support system (Ortlepp & Stacey, 1998).
3.4.2 Calculation of peak load
Modeling of shotcrete equipped with external steel meshes is a complicated task. The difficulty comes from the complex interaction between the shotcrete and the steel mesh, e.g., friction forces between the mesh and the shotcrete, clamping of the mesh at rock bolts, etc. A simplification for calculations can be made by considering the support as composite beams of steel and concrete (Figure 21).

\[
M_{pc} = R_s \left[ \frac{D}{2} D_s - \frac{R_s}{R_c} \left( \frac{D_s - D_P}{2} \right) \right]
\]

where \( R_s \) is the tensile resistance of the steel beam, \( D \) is the height of the steel beam, \( D_s \) is the height of the concrete slab, \( D_P \) is taking into account the shape of the concrete slab and is equal to zero for solid slabs and \( R_c \) is the compressive resistance of the concrete slab, determined for an effective width of the concrete slab supported by the steel beam. In equation (3.10) it is assumed that the plastic neutral axis lies in the concrete slab, i.e. that \( R_c > R_s \). This is correct for the tested shotcrete panels where the small dimensions of the steel wires gives a small resistance.

Calculations of composite steel and concrete beams takes into account the shearing forces created between the concrete slab and steel beam. Equation (3.10) is based on the assumption that the top flange of the steel beam is completely restrained by the concrete slab. However, due to the way in which the steel meshes are attached to the shotcrete, it is probably not valid to do this assumption. When steel mesh is bolted over shotcrete, a gap between the mesh and the shotcrete is normally created with a size in the order of 10 mm. Because of this gap, the mesh is able to move independently during the initial loading until the shotcrete deflects enough to come in contact with the mesh.

Another problem with using the simplification of a composite beam is to determine the contribution of load capacity from the different steel wires when a point load is applied to the simplified beam. The interaction between shotcrete and mesh is difficult to estimate by the fact that the wires in the mesh are oriented randomly in relation to the supports and the point
load. In addition, the external mesh is not in direct contact with the shotcrete at all locations since there are only four bolts securing the mesh to the shotcrete. Thus, there are initially no clamping forces on the mesh between the bolts, which makes it possible for the mesh to move freely in relation to the shotcrete between the bolts.

No existing model to calculate the peak load was found appropriate for shotcrete with external steel mesh. The possible simplification of a composite beam was not considered to be justified due to the many assumptions and uncertainties discussed above. The peak loads after cracking in relation to shotcrete thickness will therefore be forecasted based on the test results obtained in the work of this thesis.
4 Case studies of rock support in seismically active mines

A literature review was performed in order to learn how rock support systems are designed in other mines that are experiencing seismic events. Special focus was laid on mines with a mining method similar to LKAB’s mines, where large volumes of rock are affected in the rock mass. Two mines in Canada were found to be relevant for this study and are therefore included in this section. Both mines use steel mesh as a retaining support on the excavation boundary.

4.1 The LaRonde mine, Canada

At the LaRonde mine located in north-east Canada, Agnico Eagle is conducting mining in massive lenses of sulphide ore containing gold, silver, copper and zinc. In 2007, the mining was performed at a depth between 980 to 2360 meters below surface (Mercier-Langevin & Turcotte, 2007). These large mining depths result in high stresses. Together with a steeply dipping foliation, complicated failure mechanisms are generated at some areas of the mine.

The haulage drifts in the LaRonde mine are sensitive to buckling in the walls since these drifts run parallel to the foliation of the rock. Measurements have shown that the wall convergence in some areas has exceeded one meter, with visible fractures in the rock mass being up to 6 meters deep (Mercier-Langevin & Turcotte, 2007).

The rock support system in the LaRonde mine consists of a primary support with different types of bolts (resin-grouted, split-set, cement grouted, etc.) and a secondary support with different types of cable bolts or cone bolts for areas with large spans or areas subjected to dynamic loading.

The surface support generally seems to consist of welded 6-gauge wire mesh. Unfortunately, the used wire mesh system is not stated. Mercier-Langevin & Turcotte (2007) determined that welded 8- and 9-gauge wire mesh was unable to resist the displacements induced in the haulage drifts, while chain-link mesh was determined as too flexible since it was unable to prevent increased loosening of rock once a fracture was initiated. This created large volumes of fractured rock between the rock bolts. Shotcrete seems to be used as a support element in the mine, but detailed information about the thickness of the shotcrete was not presented.

In addition to the welded 6-gauge wire mesh, straps with 0-gauge mesh are installed to provide a better load distribution between the bolts with good results according to Mercier-Langevin & Turcotte (2007).

4.2 The Brunswick Mine, Canada

The Brunswick mine located in eastern Canada was formerly operated by Xstrata, but was closed in 2013 due to exhausted ore deposits (CBC News, 2013). Before closing, the mine was using long hole open stoping with delayed backfilling to extract the sulphide ore with high zinc content down to a depth around 1200 meters below surface (Joughin et al., 2002). The rock mass was in general competent and brittle, and stress induced failure in the rock was a common phenomenon.
During 1999, the Brunswick mine experienced an increase in the occurrence of seismic events and corresponding violent ejections of rock. These events were caused by a combination of increasing stress levels in remaining ore reserves, a generally highly stressed rock mass and unfavorable geological properties.

In order to counteract the violent ejections of rock due to seismic events, the mine developed a yielding rock support system. This support system consisted of shotcrete covered by chain-link mesh with wire diameter 4.9 mm and 50 mm apertures; attached to the rock with 2.3 meter long modified cone bolts in a 1x1 m pattern. In addition, 300 mm wide straps of mesh with diameter 7.7 mm and wire spacing 100x100 mm was bolted on top of the mesh, running along the axis of a drift (Joughin et al., 2002).

On the 13th of October 2000, a seismic event occurred in the Brunswick mine. The mine’s seismic monitoring system registered the magnitude to 1.6 on the local scale, and a second event with magnitude 2.7 followed on the 17th of October together with several smaller events between these two main events. Observed damages in two nearby crosscuts indicated that the installed dynamic rock support system had performed well, although bending of bolt plates had occurred, large open cracks had formed in the shotcrete behind the chain-link mesh and several bolts had undergone a displacement of several centimeters (Joughin et al., 2002).

The installation of the dynamic rock support was not completed in this area at the time of the seismic activity, however. Hence, a large portion of the area had only the mine’s conventional drift support system. This system consisted of 7 m twin cable bolts in the back, combined with 2.3 m long resin grouted rebars in a 1.5x1.5 m pattern. The surface support consisted of welded wire mesh with 3.7 mm diameter wire with 100x100 mm wire spacing, covered with regular shotcrete. The walls of the drifts had only fiber reinforced shotcrete, without mesh. The conventional drift support system was considered to be too stiff for the expected dynamic loads. This was confirmed during the seismic events since one of the damaged crosscuts, which was initially secured by conventional support, was being secured with dynamic support at the time of the seismic events. Since the additional support was installed only in some parts of the crosscut, it was possible to distinguish the effect of the two support systems, as shown in Figure 22. The part of the crosscut which was secured by the conventional support suffered from extensive damage while the dynamic support maintained its stability (Joughin et al., 2002).
4.3 Analysis of case studies

The case studies indicate that steel mesh is a common element in rock support systems with dynamic loads. Both sources in the case study describe the dimensions of the used steel mesh and bolts, but do not present any thickness or properties of the shotcrete. This might indicate that the supporting effect of shotcrete is neglected in the design of the support systems. The main purpose of the shotcrete seems to be the agglomeration of small pieces of loosened rock into larger panels which are easier supported by the mesh. Loads acting on the support system are therefore probably designed to be carried by the rock bolts alone, with the steel mesh acting as a retaining element (Mercier-Langevin & Turcotte, 2007).
5 Performed Tests
The performance of shotcrete equipped with external steel mesh and shotcrete reinforced with embedded steel mesh were studied in laboratory tests. The tests used centrally loaded round determinate panels (RDP) made of shotcrete. All tests were prepared and executed at the concrete laboratory of LKAB Berg och Betong in Kiruna (subsidiary of LKAB, contractor of shotcrete supply for the mines). To distinguish the contribution the steel mesh has on the strength, tests were first performed on panels made of only fiber reinforced shotcrete.

5.1 Testing procedure
The tests on the shotcrete panels were performed using the testing equipment specified in the standard ASTM C1550. The standard describes a test method for measuring the flexural toughness of fiber reinforced shotcrete, defined as the energy absorbed by the shotcrete panel during loading. The absorbed energy was determined from the area under the load-deflection curve between the onset of loading and a certain deflection interval of the panel (ASTM, 2012).

LKAB has a requirement that the shotcrete used for dynamic rock support in the underground mines must have a minimum toughness of 490 J at 40 mm deflection when tested according to ASTM C1550 (LKAB, 2010). The value of 490 J at 40 mm deflection is valid for test panels were the shotcrete is sprayed in the molds. For ASTM C1550 test panels where the shotcrete is cast in the molds, the minimum value of the toughness is set to 650 J instead. The higher value is used in order to correct for e.g. the fiber orientation in the shotcrete. When shotcrete is cast, the orientation of fibers tends to be more uniform than it is when the shotcrete is sprayed. A random orientation of the fibers provides more homogeneous properties for the shotcrete and gives a high strength for an arbitrary load direction.

The test is performed by applying a point load centrally on a circular panel of sprayed or cast shotcrete (Figure 23) with a thickness of 75 mm (-5/+15 mm) and diameter of 800 (± 10 mm). The load on the shotcrete panel is applied vertically from above through a hemispherical-ended piston that advances at a constant speed of about 4 mm/min. During the test, the applied load is recorded together with the deflection at the edge of the piston. The shotcrete panels are supported by three symmetrically positioned supports with pivot joints (Figure 24) in the test rig. The supports are positioned along a circle with a radius of 375 mm, with 120° spacing between each support; see Figure 24 (ASTM, 2012).
Figure 23. The testing equipment used for the ASTM C1550-12 tests at LKAB Berg och Betong.

Figure 24. The pivot supports used to support the shotcrete panels during the ASTM C1550 tests (from ASTM, 2012).

Figure 25. The test panels with point load and supports. The radius of the panel was 400 mm, and the support radius was 375 mm.
The test panel undergoes bi-axial bending in response to the central point load. Also, the use of three supports with pivot joints results in determinate out-of-plane reactions before cracking of the shotcrete occur. The energy absorbed by the shotcrete panel represents the ability of fiber-reinforced shotcrete to redistribute the stresses following cracking of the concrete (ASTM, 2012).

All tested shotcrete panels were prepared and measured in accordance with the ASTM standard, with exception for the shotcrete panels where the thicknesses was 50 or 100 mm in addition to the standard 75 mm thickness. Clearly, the use of welded steel mesh installed externally on the shotcrete panels or embedded inside the panels also deviate from the ASTM C1550 standard. However, the testing procedure and equipment specified in the ASTM standard was still used in order to be able to compare the results with standard ASTM C1550 tests. Full specifications for the ASTM C1550 testing method can be found in ASTM (2012).

5.2 Preparation of tests

All shotcrete panels used in the tests were cast in molds consisting of a hard-plastic bottom plate onto which a metal ring was mounted to create walls for the mold. In total, four different series of panels were cast according to Table 3. In order to obtain consistent quality and repeatable results, it was decided to cast the shotcrete in the molds instead of spraying it with shotcrete spraying equipment from the mine. Since no spraying equipment was used, there were no accelerating chemicals added to the shotcrete. After casting and dismantling of the molds, the shotcrete plates were stored dry at a temperature about 20°C. All panels cured for about 28 days before testing.

<table>
<thead>
<tr>
<th>Specifications</th>
</tr>
</thead>
<tbody>
<tr>
<td>Fiber reinforced shotcrete (FRS)</td>
</tr>
<tr>
<td>FRS + External mesh</td>
</tr>
<tr>
<td>FRS + External mesh</td>
</tr>
<tr>
<td>FRS/plain shotcrete + Embedded mesh</td>
</tr>
</tbody>
</table>

Table 3. Cast series of testing panels for the ASTM C1550 tests.

The steel mesh used in the tests was identical to the mesh used in the mines and had a wire diameter of 5.5 mm, 75x75 mm wire spacing and wires made of stress-relieved, annealed steel with properties presented in Table 4. The panels were cast with steel fiber reinforced shotcrete in accordance to the standard recipe used in the Kiirunavaara mine. The steel fibers in the shotcrete were of the model DRAMIX 65/35 manufactured by Bekaert. All shotcrete that was used in the casting of the panels came from batches intended to be used in the mine. Due to the limited number of casting molds, each series was cast from separate shotcrete batches.

The compressive strength for cast cubes of shotcrete in this quality is normally 50 MPa. The shotcrete recipe is presented in Table 5.
Table 4. Strength parameters for the welded steel mesh used by LKAB, given by the supplier of the mesh.

<table>
<thead>
<tr>
<th>Ultimate tensile strength (MPa)</th>
<th>Tensile failure strain</th>
<th>Yield strength (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>360-370</td>
<td>&gt;30% (A10)</td>
<td>300</td>
</tr>
</tbody>
</table>

Table 5. Standard recipe for fiber reinforced shotcrete used in the Kiirunavaara mine.

<table>
<thead>
<tr>
<th>Materials</th>
<th>Amount (kg/m³)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Aggregate (0-4 mm)</td>
<td>1295</td>
</tr>
<tr>
<td>Aggregate (4-8 mm)</td>
<td>97.75</td>
</tr>
<tr>
<td>Water</td>
<td>230</td>
</tr>
<tr>
<td>Cement</td>
<td>500</td>
</tr>
<tr>
<td>Silica</td>
<td>20</td>
</tr>
<tr>
<td>Water reducing admixture</td>
<td>2.08</td>
</tr>
<tr>
<td>Admixture</td>
<td>1.92</td>
</tr>
<tr>
<td>Steel fibers</td>
<td>40</td>
</tr>
</tbody>
</table>

5.2.1 Series 1: Fiber reinforced shotcrete panels
In the first series, four panels were cast for each of the thicknesses 50, 75 and 100 mm, giving a total of 12 panels. The panels were allowed to cure for 29 days before testing. Figure 26 shows the preparation of the panels in the first test series.

Figure 26. Casting of the shotcrete panels in test series 1.

5.2.2 Series 2: Panels with external welded steel mesh
Four panels, with a thickness of 75 mm, were tested in the second series, where the purpose was to determine the most suitable method for attaching the external welded steel mesh to the testing panels. The mesh was cut into quadratic pieces measuring 750 mm x 750 mm in order to simplify the handling of the mesh during attachment to the panels and testing (Figure 27). The mesh was attached to the shotcrete panels by first folding the corners of the mesh around the panel after which four expanding bolts of dimension M8 and a length of 75 mm was installed through the mesh, 40 mm deep into the panel. A metal washer and a nut mounted on each bolt secured the mesh in the correct position.
Figure 27. One of the steel meshes in series 2, prepared to be attached underneath a shotcrete panel.

One panel was prepared with bolts installed from the bottom side of the panel and two panels were prepared with the bolts installed from the upper side, with the bolts positioned in the up-folded corners of the mesh, as shown in Figure 28a. The fourth panel was also prepared with the bolts positioned on the upper side, but each bolt was installed through a shackle fastened to the up-folded corners of the mesh in order to simulate the situation for the 100 mm thick panels where the corners of the mesh would not reach over the edges of the panel, as shown in Figure 28b. The purpose of the shackles was therefore to act as an extension of the mesh to the bolts. The panels were allowed to cure for 27 days before testing.

Figure 28. Shotcrete panels in series 2 equipped with external steel mesh a) attached with an M8 expanding bolt and b) attached with an M8 expanding bolt through a shackle.
5.2.3 Series 3: Panels with external welded steel mesh
In the third series, a total of twelve panels were cast with thicknesses of 50, 75 and 100 mm, four panels of each thickness. All panels in the third series were equipped with external welded steel mesh of the same dimensions as the mesh used in the second series. The mesh attached on the 50 and 75 mm thick plates were attached to the panels with expanding M8 bolts, installed from the upper side of the panel through the mesh at the up-folded corners (Figure 28a). Shackles were added between the bolts and the mesh on the 100 mm thick panels since the up-folded mesh corners were unable to reach over the edges of the panel (Figure 28b). The bolts installed in the 50 mm panels were positioned about 50-80 mm from the edges of the panels. Corresponding distance for the 75 mm panels were 30-45 mm and for the 100 mm panels the distance were 50-60mm.

Since no earlier tests had been performed in the laboratory on 50 mm thick panels, there was a concern that the installed expanding bolts would cause the 50 mm thick panels to crack during the tests. Because of this, the 50 mm thick panels were reinforced with brackets of 8 mm diameter rebar cast in the shotcrete inside the panels where the bolts would be positioned. Identical brackets were cast inside the 75 mm thick panels in order to make it possible to compare the behavior of these panels with the panels tested in the second series and hence conclude if the brackets had a large influence on the performance of the panels. An example of a bracket is shown in Figure 29.

![Figure 29. One of the brackets used as reinforcement in the 50 and 75 mm thick shotcrete panels in series 3.](image)

The intention was to compare the results between the first and third series in order to distinguish the effect of the external steel mesh. The panels were allowed to cure for 28 days before testing. Figure 30 shows the third series after casting and installation of the steel meshes.
Figure 30. The mesh equipped panels in series 3 before testing. From the left: 100 mm, 75 mm and 50 mm thick shotcrete panels.

5.2.4 Series 4: Panels with embedded steel mesh

The final, fourth series consisted of six shotcrete panels with a thickness of 75 mm. All of these panels were reinforced by embedded welded steel mesh of the same wire dimensions as used in series 2 and 3, but with the mesh cut to fit inside the circular molds for the panels. The steel mesh was positioned in the middle of the panels during casting. Three of the panels were cast with unreinforced shotcrete, while the remaining three panels were cast with ordinary steel fiber reinforced shotcrete. The purpose of this was to determine the influence of the steel fibers. The panels were allowed to cure for 28 days before testing.
6 Results

6.1 Results from series 1: Fiber reinforced shotcrete panels

The results from the first testing series are shown in Figure 31 and Figure 33. Each panel is designated with three numbers; the first number is the panel thickness in millimeters. The second number corresponds to the test series while the third number represents the individual panel of each thickness. For example, “50 1-2” designates the second 50 mm thick panel in the first series. As can be seen in Figure 31, the thickest panels were capable of carrying the highest loads. With decreased thickness, the peak loads for the tested panels decreased. All panels of the same thickness showed a similar behavior during loading, with some increased scattering for the thicker panels. At deflections reaching 80 mm all panels in the series showed similar load carrying capacities. The sudden dip of strength for panel 100 1-1 after 10 mm deflection was caused by an error in the testing rig. An example of a shotcrete panel from series 1 after cracking is shown in Figure 32.

![Figure 31. Load - deflection diagram showing the ASTM C1550 test results from the first series.](image-url)
The energy absorbing capacity, described in section 3.2.3, for the tested panels in series 1 is shown in Figure 33. The thicker panels displayed higher initial energy absorption than the thinner plates. With increasing deflection however, the growth of absorbed energy decreased for all thicknesses. The 50 mm thick panels were not able to absorb the 650 J requested by LKAB, even at a deflection of 80 mm.

**Figure 33. Energy absorption (toughness) of the shotcrete panels in series 1.**

### 6.2 Results from series 2: Panels with external welded steel mesh

Figure 34 and Figure 35 shows the results from series 2. All panels had the mesh attached to the underside of the shotcrete. Panel 75 2-1, 75 2-3 and 75 2-4 had the external mesh attached with expanding bolts from the upper side of the panel, where the steel mesh was folded over the corners of the panels. Panel 75 2-2 had the bolts installed from the underside of the shotcrete panel.
The first panel was tested until reaching a deflection of 80 mm. The remaining panels were loaded until reaching a deflection of 100 mm, since the residual strength of the first panel was higher than expected. The load-deflection diagram in Figure 34 indicates that the panel 75 2-2, with the mesh bolted from the underside, had the lowest load carrying capacity at large deflections. During loading, the second panel also showed extensive cracking close to the bolts while the panels bolted from the upper side showed no signs of cracking. The sudden drop of load carrying capacity for panel 75 2-4 close to 100 mm deflection was the result of a mesh wire slipping on one of the pivot supports on the test rig. The absorbed energy, as presented in Figure 35, is similar for all four panels.

Figure 34. The load capacity of the mesh equipped test-panels in series 2.
Figure 35. Energy absorbing capacity of the mesh equipped panels in series 2.
6.3 Results from series 3: Panels with external welded steel mesh

In Figure 36 the load-deflection curves for the panels in series 3 are shown. Figure 39 shows the energy absorbing capacity of the panels. All panels tested in series 3 showed an initial strength drop when the shotcrete cracked. When the deflecting shotcrete came in contact with the underlying steel mesh, the load increased to a higher level than it was at the cracking of the shotcrete. The distance between the shotcrete panel and the underlying steel mesh was in general between 0 – 10 mm since the steel mesh was very stiff and the attachment of the mesh to the panels was done manually.

![Figure 36. The load capacity of the mesh equipped panels in series 3.](image)

As can be seen in Figure 36, the 100 mm thick panels withstood the highest loads in the series before cracking of the shotcrete occurred. Shortly after cracking of the shotcrete, the load was increased to a higher value than the shotcrete cracking load. However, the 100 mm thick panels experienced a decrease of strength until the load once again started to increase for three of the panels when the deflection exceeded 60 mm, perhaps as a result of the steel mesh becoming more activated. The panel 100 3-2 suffered from a sudden drop of load at 73 mm deflection caused by an unfavorable fracturing of the shotcrete. Two large pieces of shotcrete were formed together with a smaller third piece below the loading piston, which lost its contact with the steel mesh and became unable to carry any load whereupon the test was aborted. The other panels formed three pieces of even sizes after cracking. Figure 37 shows an example of a 100 mm thick panel during the testing.
The 75 mm thick panels showed a behavior similar to the 100 mm panels, with a decrease of strength in the residual loading phase before an increase of load begun after a deflection of about 60 mm. When the 50 mm panels were tested, they all displayed a relatively plastic behavior after cracking of the shotcrete. At least three of the 50 mm panels showed tendencies to a load increase after 60 mm deflection, but this increase was fairly small.

Some panels in series 3 showed signs of shearing failure in the shotcrete due to the loading. This was especially evident for the 50 and 75 mm thick shotcrete panels, and certain panels also displayed punching shear of the shotcrete directly under the piston. An example of these shear failures is shown in Figure 38.

The energy absorbing capacity was steady during loading for all panels shown in Figure 39. The 100 mm thick panels showed a slightly higher energy absorbing capacity during the initial loading and after about 80 mm deflection, compared to the other thicknesses. Even though the 50 mm thick panels were supported by the steel mesh, they were unable to absorb 650 J after 40 mm deflection.
6.4 Results from series 4: Panels with embedded steel mesh

The results from the fourth series are presented in Figure 40 and Figure 42. As can be seen in Figure 40, the shotcrete cracked at about 20-25 kN for all panels. However, the panels made of fiber reinforced shotcrete displayed a significantly higher strength after the cracking of the shotcrete compared to the panels without fibers, and was able to carry up to about 14 kN higher loads after cracking of the shotcrete. The fiber reinforced shotcrete panels were also able to undergo larger deflections before the individual wires of the embedded steel mesh began to fail. The failures of the steel wires in the mesh are shown in the diagram of Figure 40 as sudden drops of load, starting at around 50 mm deflection for panel number 4-6.

All tested panels in the fourth series experienced several wire failures in the embedded mesh. After testing of panel number 4-3, the shotcrete around the cracks of the panel was removed in order to expose the failed mesh. An examination of the exposed mesh revealed that 10 out of 15 wires intersecting the cracks in the panel had failed. The remaining five intact wires showed indications of beginning failures. Figure 41 shows a panel after testing, where the steel wires of the mesh had failed.
Figure 40. Load capacity of the mesh reinforced panels in series 4.

Figure 41. The condition of fibers and mesh in a reinforced panel from series 4, after testing. The steel wires in the mesh that crossed the crack of the panel showed clear signs of tensile failure.

The energy absorbing capacity of the panels made of fiber reinforced shotcrete, shown in Figure 42, were higher than for the panels without added fibers.
Figure 42. Energy absorbing capacity for the mesh reinforced panels in series 4.
7 Analysis of results
Beside an analysis of the obtained results for each series, the results were used to forecast the properties of shotcrete panels of greater thicknesses. The different types of panels were compared with each other in order to clarify the differences in load capacity and toughness between the panel types.

7.1 Analysis of series 1: Fiber reinforced shotcrete panels
The panels tested in series 1 showed that all panels, regardless of thickness, approached a residual strength of about 3 kN at 80 mm deflection. This was probably caused by the angle of the cracks in the shotcrete, which was increasing together with the deflection of the panel (Figure 43). When the angle of the cracks became larger, the remaining amount of steel fibers anchored in the shotcrete at both ends was reduced as more fibers were drawn out from the shotcrete. Therefore, even though the thicker panels had a larger total amount of steel fibers redistributing the stresses in the panel, the number of load bearing fibers at large deflections became increasingly more equal between the panels regardless of their thickness. This reasoning can be proved by using an equation presented by Bernard (2004), where the average crack rotation angle of an ASTM C1550 panel can be determined from

\[ \varphi = \sqrt{\frac{3\gamma}{r}} \]  

(7.1)

where \( \varphi \) is the average crack rotation angle in radians, \( \gamma \) is the post-crack deflection of the panel and \( r \) is the radius of the supports during the test, equal to 375 mm (Figure 43).

\[ x = 2t \cdot \sin \left( \frac{\sqrt{3\gamma}}{2} \right) \]  

(7.2)

where \( t \) is the thickness of the shotcrete panel and \( x \) is the crack opening at the bottom of the panel as shown in Figure 43. Insertion of the three panel thicknesses into equation (7.2) gives the corresponding crack openings shown in Table 6 for different deflections. The values indicate that the crack opening at the bottom of the 75 and 100 mm panels are significantly larger than for the 50 mm panels at equal deflections.
### 7.1.1 Peak load related to panel thickness

By inserting the measured peak loads from series 1 into equation (3.1) it was possible to obtain the stresses of the outer tensile fiber $\sigma$ at the corresponding peak loads by solving equation (3.2). This way the actual tensile strength for the used shotcrete could be determined. Table 7 shows the peak loads of the tested panels in series 1.

#### Table 7. Average peak loads of the tested panels in series 1.

<table>
<thead>
<tr>
<th>Panel type</th>
<th>P max (kN)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Series 1 50 mm</td>
<td>12.2</td>
</tr>
<tr>
<td>Series 1 75 mm</td>
<td>30.4</td>
</tr>
<tr>
<td>Series 1 100 mm</td>
<td>48.1</td>
</tr>
</tbody>
</table>

Solving equation (3.1) and (3.2) with $R = 0.4$ m, $r = 0.375$ and $d =$ panel thickness, the mean tensile strength $\sigma$ for all thicknesses was 5.4 MPa at cracking load of the shotcrete matrix. By using the mean value of $\sigma$, the cracking load for panels of thicknesses up to 150 mm was determined using equation (3.1) and (3.2), see Figure 44.
Figure 44. The peak load after cracking of the shotcrete based on measurements from series 1.

A comparison of the calculated peak loads and the peak loads measured in series 1, Figure 44, shows a relatively good correlation between the values. A more accurate comparison would be obtained with more test results and by testing panels of other thicknesses.

7.1.2 Toughness related to panel thickness

The toughness of fiber reinforced shotcrete panels of varying size were estimated using the results of series 1. The mean load bearing capacity for each thickness of the tested panels were studied at every tenth millimeter starting at 10 mm deflection. Mean load capacities for the 75 and 100 mm thick panels were divided by the mean load capacity for the 50 mm panels to obtain the relation between the values.

When the ratios between the load capacities were drawn in a diagram, it was possible to relate a unique logarithmic trend to the ratios for each deflection. Theoretical ratios for panels up to 150 mm were then determined by these trends. By multiplying the theoretical ratio for each panel size with the measured load capacity of the 50 mm thick panels, it was possible to obtain a prediction of the residual load capacity for each panel (Figure 45).
Figure 45. Comparison between residual load for ordinary shotcrete panels and the measured mean load of the panels in series 1.

From the average load carrying capacities in Figure 45, it was possible to estimate the energy absorbing capacities for panels of different thicknesses in the deflection interval 10-80 mm. As shown in Figure 46, the energy absorption becomes less influenced by the panel thickness as the thickness increases.

Figure 46. Theoretical toughness of fiber reinforced shotcrete panels in the deflection interval 10-80 mm.
7.2 Analysis of series 2: Panels with external welded steel mesh

The results from the second series showed that the attachment of steel mesh with bolts on the upper side of the panel was more advantageous than installing them on the underside. Since the bottom sides of the panels were subjected to tensile stresses during loading, the holes for the bolts created weak zones through which cracking occurred. The panels with and without shackles was compared regarding the load carrying capacity in order to determine if the shackles influenced the load bearing capacity of the panels. However, this comparison did not show any significant influence from the shackles regarding both bearing and energy absorbing capacity.

7.3 Analysis of series 3: Panels with external welded steel mesh

Once fracturing had occurred in the shotcrete of the panels in series 3, the load bearing capacity increased to a higher value than when the shotcrete cracked. For most panels the shotcrete cracked at a deflection around 3-5 mm while the maximum strength was achieved at around 8-13 mm deflection. The apparent delay of peak strength might have been the result of the gap between the panel and the underlying mesh, being up to 10 mm for some panels. Because of this gap, the panels were allowed to deflect up to 10 mm before they came in contact with the underlying mesh and hence could utilize its strength. Still, this gap can be considered to be realistic since a similar gap is created when the mesh is installed in the mines.

The residual strength of the panels seemed to be strongly related to the thickness of the shotcrete since the loads carried at large deflections were distinctly different between the groups of different panel thicknesses (Figure 36). At 40-100 mm deflection, the 50 mm thick panels carried around 10-14 kN while corresponding loads at the same deflections were 17-20 kN for the 75 mm thick panels (with one exception) and 26-45 kN for the 100 mm thick panels.

When the load bearing capacity at large deflections is compared with the results from series 1, it is clear that the steel mesh does not give an equal load carrying capacity for all thicknesses. If the steel mesh alone would be the main source for load bearing capacity at large deflections, there would be smaller differences between the different panel thicknesses at large deflections. Instead, based on the results and observations from the tests, it is possible that the steel mesh acts as a confining element under the shotcrete panel and hampers the widening of the cracks. This might lead to a slower pull-out of the steel fibers and more energy absorbed as friction between the fibers and the shotcrete. Another factor contributing to the increase of load bearing capacity can be frictional and clamping forces between the mesh and shotcrete at the edges of the shotcrete panels.

All panels with external steel mesh showed an increase of load bearing capacity after relatively large deflections. This was particularly evident in the results for the 100 mm thick panels with external steel mesh, where the load increased about 10 kN for two panels after a point of lowest load at around 50-60 mm of deflection.
The indicated increase in strength for the mesh supported panels might be the result of the steel mesh becoming more mobilized as an increasingly larger area of the shotcrete came in contact with the mesh due to the deflection. Since the performed test on the mesh supported panels were stopped at deflections of 100 mm, it is not possible to determine the capacity for these panels at larger deflections. However, since the steel mesh did not show any visible signs of initiated failures, it is possible that the mesh would have been able to sustain additional deflection. Results from tests conducted by others, for example Player et al. (2008) mentioned earlier, indicate that the strength of a welded steel mesh is not utilized to any large extent at deflections smaller than 100 mm.

### 7.3.1 Peak load related to panel thickness

Due to the lack of an appropriate calculation model, the peak loads after cracking in relation to panel thickness were extrapolated from the measured values in series 3 (Figure 47). As shown by the extrapolated trend line, the growth of peak load related to the panel thickness follows a polynomial trend like the calculated values for series 1. A study of the measured peak loads in Figure 47 indicates that a decrease of shotcrete thickness reduces the strength of the panels. When the panel thickness is reduced from 100 mm to 50 mm, i.e. halved, the peak load is reduced to about 20% of the strength of a 100 mm thick panel.

![Figure 47. Peak loads for panels in Series 3 with extrapolated trend line.](image)

### 7.3.2 Toughness related to panel thickness

The toughness of the panels equipped with external steel meshes was studied in relation to the panel thickness. First, the contribution from the steel mesh to the load carrying capacity was examined. Since the mesh equipped panels in series 3 had equal thicknesses as the ordinary panels in series 1, it was possible to subtract the measured loads carried by the ordinary shotcrete panels from the mesh equipped panels at a corresponding deflection. The difference in load was thereby taken to be the direct effect of the attached meshes (Figure 48).
As shown in Figure 48, the apparent contribution from the steel mesh increased for thicker shotcrete panels. This clearly indicates that the thickness of the fiber reinforced shotcrete influences the load bearing capacity of shotcrete with external steel meshes. Figure 48 also indicates that the load bearing capacity assumes a nearly linear behavior for deflections exceeding 40 mm. Therefore, the growth of load capacity with increased shotcrete thickness was examined at deflections between 40 and 80 mm. The loads carried by the mesh of the 75 and 100 mm thick panels were divided with the corresponding loads for the 50 mm panels at each deflection to obtain the relation between the thicknesses. This resulted in a mean ratio of 2 (2.08) between the 75 and 50 mm panels, and a mean ratio of 3.5 (3.47) between the 100 and 50 mm panels.

By plotting the mean ratios in a diagram, it was possible to draw a polynomial trend line for the ratios (Figure 49) with the equation

$$R = 0.0004t^2 - 0.01t + 0.5 \quad \text{(7.3)}$$

where $R$ is the ratio between a shotcrete panel of an arbitrary thickness and a 50 mm panel and $t$ is the thickness in millimeters of the shotcrete panel.
Figure 49. Ratios together with a polynomial trend line, showing the relative increase of load carrying capacity compared to 50 mm thick panels when external steel meshes are used.

The increase of load bearing capacity supplied from the steel mesh was determined for deflections between 40 and 100 mm, by multiplying the corresponding ratio $R$ obtained by equation 7.3 with the load differences for the 50 mm thick panel in Table 8. The total load bearing capacity for panels with thicknesses 50 – 150 mm was then obtained by adding the determined contribution from the meshes to the corresponding load bearing capacity of the ordinary shotcrete panels calculated for series 1 (Figure 50).

Table 8. Calculated differences in mean loads between series 3 and series 1, 50 mm thick panels.

<table>
<thead>
<tr>
<th>Deflection (mm)</th>
<th>Difference in load (kN)</th>
</tr>
</thead>
<tbody>
<tr>
<td>40</td>
<td>5.47</td>
</tr>
<tr>
<td>50</td>
<td>5.84</td>
</tr>
<tr>
<td>60</td>
<td>6.68</td>
</tr>
<tr>
<td>70</td>
<td>7.59</td>
</tr>
<tr>
<td>80</td>
<td>8.51</td>
</tr>
</tbody>
</table>
The toughness for shotcrete panels equipped with external mesh was determined by the average load capacity over the deflection interval 40-80 mm. As shown in Figure 51, the increase of toughness follows a polynomial trend as the panel thickness is increased.

Figure 51. Calculated toughness for shotcrete panels equipped with external steel mesh under the deflection interval 40-80 mm.
7.4 Analysis of series 4: Panels with embedded steel mesh

The results from the fourth testing series indicates that the 75 mm thick panels reinforced with both steel fibers and an embedded steel mesh performed best in response to loading. These panels reached the highest loads and underwent relatively large deflections under a plastic state. Still, the load bearing capacity decreased rapidly after deflections exceeding 50-60 mm.

Panels with embedded steel mesh but without steel fibers showed a behavior similar to the fiber and mesh reinforced panels. However, when no fibers was added to the shotcrete, the cracking load of the shotcrete was lower and the panels were unable to reach as high residual loads as the fiber and mesh supported panels. Also, the steel wire failures in the embedded mesh were initiated somewhat earlier for the panels without steel fibers. Hence, the steel fibers contributed to both increased peak load of the shotcrete and residual strength when embedded steel mesh were used.

7.4.1 Peak load related to panel thickness

The simplification of calculating the moment capacity as a traditional beam was considered to be sufficiently accurate for the mesh reinforced panels since the tests involved a point load on a statically determinate panel. The peak load after cracking was estimated for panels of different thicknesses in order to obtain a rough estimate of the influence from the panel thickness.

Equations (3.1) to (3.8) were used for the calculations with the values \( R = 0.4m, r = 0.375 \text{ m} \), \( f_{sr} = 360 \text{ MPa} \) and \( f_{cc} = 50 \text{ MPa} \). During the calculation it was assumed that the steel mesh would be positioned on the same distance from the bottom of the panel regardless of the panel thickness. The steel mesh was placed in the middle of the tested panels, i.e. at a depth of 75/2 mm. Thus, the value of \( d \) for each panel of thickness \( t \) was

\[
    d = t - 75/2
\]

The steel wire diameter of 5.5 mm gave the reinforcement area, \( A_s \), equal to 317 mm\(^2\)/m, i.e. the cross-sectional steel area per meter mesh. Estimated peak loads for different panel thicknesses are shown in Figure 52 together with the measured results from series 4. Clearly, testing of mesh reinforced panels of other thicknesses is required to confirm the validity of these estimated values.

The ratio between the mean values of peak loads for the two panel types in series 4 was 1.34, i.e. the panels reinforced by steel fibers and mesh carried 34\% higher peak loads than the panels reinforced by mesh alone. By multiplying this ratio with the estimated values for the reinforced panels, a simple estimate was given of the effect from steel fibers on the peak load (Figure 52).
The peak load in relation to panel thickness for the fiber reinforced panels in Figure 44 shows a polynomial growth. Considering this, it might be expected that the steel fibers in the mesh reinforced panels of series 4 would cause a similar behavior if the panel thickness were increased since the amount of steel in the steel mesh is constant. The peak loads for the mesh reinforced beams with fibers in Figure 52 should therefore be considered as rough minimum values.

### 7.5 Comparison of series 1 and 3

The mean values of the load carrying capacity for series 1 are compared with the mean values from series 3 in Figure 53. It is clearly shown that the external steel mesh gives significantly higher residual strength to the panels, compared to the panels without steel mesh.

Since the panels without external steel mesh were tested to a deflection of only 80 mm, a direct comparison cannot be made with the corresponding steel mesh equipped panels at deflections exceeding 80 mm. At 80 mm deflection, the loads carried by the 50 mm panels supported by external steel mesh were about 8 kN higher than the panels without external mesh, as shown in Figure 53. The corresponding differences for the 75 and 100 mm thick panels were 18 kN higher and 30 kN higher, respectively. By studying the trends of the load-deflection curves for the two panel types, it is likely that the differences in load will increase for larger deflections.

While the panels from the first series experienced a steady decrease in strength after cracking of the shotcrete, the mesh supported panels from series 3 underwent large deflections with significantly higher strength. The 50 mm thick panels supported by steel mesh behaved almost perfectly plastic when the mesh became loaded after cracking of the shotcrete, while
the thicker panels displayed a dip in the load bearing capacity until a growth of strength took place after a deflection of about 60 mm.

![Graph showing load vs. deflection for different series and thicknesses]

**Figure 53.** Mean load capacities for the tested panels in series 1 and 3.

When the energy absorbing capacities are compared for series 1 and 3 in Figure 54, it is clear that the panels supported by an external steel mesh were able to absorb larger amounts of energy.

![Graph showing absorbed energy vs. deflection for different series and thicknesses]

**Figure 54.** Mean energy absorbing capacity for the tested panels in series 1 and 3.
7.6 Comparison of series 1 and 4

Series 1 and 4 were compared to determine the difference between panels with and without embedded steel mesh. The mean test results for the 75 mm panels in series 1 and 4 are displayed in Figure 55. In the residual state, the two panel types with embedded steel meshes showed a higher strength than the panels without mesh. The panels from series 1 were not able to carry any significant loads after cracking of the shotcrete. Still, the peak loads at cracking were higher for these panels than the mesh reinforced panels without any steel fibers. This might indicate that the steel fibers contributed to both peak load capacity and residual strength of the panel when embedded steel meshes were used. In this comparison, the panels reinforced by both steel mesh and steel fibers showed the best performance since these panels were able to carry the highest loads during large deflections.

![Figure 55. Load carrying capacities for the 75 mm thick panels in series 1 and 4.](image)

As indicated by the results in Figure 55, the panels reinforced with steel fibers and steel mesh were able to absorb the highest energy, followed by the panels reinforced with only steel mesh (Figure 56). For deflections larger than 20-40 mm, the panels without embedded steel mesh experienced a decrease in energy absorbing capacity.
7.7 Comparison of series 3 and 4

In order to examine the differences between external and embedded steel mesh, the mean load carrying capacities for the 75 mm thick panels in series 3 and 4 are plotted in Figure 57. Only the panels with fiber reinforced shotcrete are displayed to clarify the impact of the mesh alone.

Figure 57 clearly show that the panels with embedded steel mesh were able to sustain higher loads, even at relatively large deflections. However, since the panels with embedded mesh experienced the wire failures in the mesh after about 60 mm deflection, the panels lost their capacity to carry loads at larger deflections. At a deflection around 70-85 mm the panels with external steel mesh showed a higher strength which also was increasing. For the same deflection, the strength for the panels in series 4 was decreasing as the steel wires in the mesh failed. A comparison of the energy absorbing capacity for the two panel types show that the mesh reinforced panels had the highest toughness, Figure 58. The decrease of strength at large deflections for the mesh reinforced panels is reflected as a decrease of toughness, while the opposite holds for the panels with external mesh.
Figure 57. Load carrying capacities for the 75 mm thick panels in series 3 and 4.

Figure 58. Energy absorbing capacities for the 75 mm thick panels in series 3 and 4.
7.8 Summary of estimated peak loads and toughness

A summary of the estimated and extrapolated peak loads for all panel types is given in Figure 59. The estimated values for the mesh reinforced panels with steel fibers are very uncertain since the interaction between the fibers and the mesh was difficult to determine from only one tested thickness. Thus, these values were excluded from Figure 59 to avoid confusion when the peak loads are compared.

![Figure 59. Estimated strength as a function of panel/beam thickness for fiber reinforced panels and mesh reinforced beams. The mean values from series 3 are displayed with an extrapolation for larger panel thicknesses.](image)

The estimated toughness for the different panel types are compared in Figure 60. The residual values for the mesh reinforced panels were difficult to estimate since only one panel thickness was tested. Thus, no calculations were made for the energy absorbing capacity of mesh reinforced panels. For the panels equipped with external meshes, the residual strength at smaller deflections was difficult to estimate due to the irregular strengths between the thicknesses. Because of this, the toughness was only compared for the deflection interval 40-80 mm.
Figure 60. Estimated energy absorbing capacity (toughness) as a function of panel thickness for fiber reinforced panels and panels equipped with external meshes.
8 Discussion and conclusions

8.1 Discussion

The results obtained through the laboratory tests indicate that the strength and toughness of shotcrete equipped with external steel mesh is strongly influenced by the shotcrete thickness. None of the tested 50 mm thick panels were able to reach the required toughness of 650 J absorbed energy at 40 mm deflection. A decrease of the shotcrete thickness will therefore lower the performance of the entire support system, assuming that the rock bolts are stronger than the surface support. The estimated toughness for panels of different thickness with external steel meshes does instead show that a better energy absorbing capacity is obtained with a thicker shotcrete layer.

The tests performed on shotcrete panels equipped with external steel mesh focused on the influence from shotcrete thickness on the panel strength. However, the steel mesh itself is another parameter that can be varied regarding wire diameter, wire spacing and steel quality. According to the existing literature, a common dimension for steel mesh used as surface support seems to be a wire diameter around 3.7-4.9 mm and a wire spacing of 100x100 mm. Hence, a weaker steel mesh should be studied in combination with shotcrete panels of different sizes to better determine the influence of the steel mesh alone. As long as satisfactory load performance is achieved for the dynamic rock support, weaker steel mesh can potentially reduce the material costs. Results from other authors regarding the strength of steel meshes with different dimensions indicate that the use of weaker meshes can be equivalent to stronger meshes at deflections below 60-70 mm (Figure 17). The advantage of heavier steel meshes is that these can undergo larger deflections before failure. Therefore, a detailed study on the required deformability of the support system should be made prior to reducing the capacity of the steel mesh.

The need for a profound understanding of the present rock deformations in the mines is also evident when embedded steel meshes are considered. Results from the laboratory tests indicate that shotcrete reinforced with embedded steel meshes can sustain larger peak loads than shotcrete equipped with external mesh, and also gives a higher residual strength. On the contrary, the results from the case studies show that embedded steel meshes can be more sensitive to large rock movements due to their stiffness. It is difficult to make any safe conclusions from the results of the panels with embedded steel meshes since only one panel thickness was tested. More tests are obviously needed on different panel thicknesses to be able to determine the influence of the steel fibers and the embedded mesh. Testing of different mesh dimensions might however be interesting for these panel types as well in order to obtain a better understanding of the interaction between mesh and shotcrete.

The underground mines in both Malmberget and Kiruna are likely to continue their mining at large depths. With the increasing depths come high stresses, which might result in an increased number of rockbursts. Higher stresses might also result in rockbursts with larger magnitudes than the magnitudes common in the mines today. In order to preserve the safety for the underground personnel and mining equipment, the importance of having a well-
performing rock support system is evident. A comprehensive understanding of the support capacity and performance requires additional testing to identify the key parameters that govern the performance of the entire support system.

**8.2 Conclusions**

Static point load tests based on the standard ASTM C1550 were performed on four test series of shotcrete panels. The purpose of the tests was to study the possibility of decreasing the thickness of the shotcrete layer in dynamic rock support. The thickness of the tested panels varied between 50, 75 and 100 mm.

One test series consisted of steel fiber reinforced shotcrete (SFRS) panels, and two series consisted of SFRS panels with underlying steel mesh. The fourth series consisted of SFRS panels and plain shotcrete panels with embedded steel meshes.

- The residual strength and energy absorbing capacity of a shotcrete rock support is enhanced significantly if a steel mesh is installed either externally or embedded inside the shotcrete. Embedded steel mesh gives a stronger support than when external meshes are used, for deflections up to about 80-90 mm. However, due to the limited possibility for the steel mesh to yield inside the shotcrete, rock support with embedded steel mesh will not tolerate excessive deformations.

- Shotcrete thickness seems to influence the strength of a rock support to a large extent even if external steel mesh is installed over the shotcrete. A reduced thickness of the shotcrete decreases the strength of a rock support consisting of shotcrete with externally installed steel mesh. This must be carefully considered before any decision is made to reduce the thickness of the shotcrete layer. Thus, a reduction of shotcrete thickness from the present 100 mm is not recommended unless a profound study is made regarding the effects from such reduction.

- Externally attached steel mesh does not increase the strength of a shotcrete support with a static value in the panel tests. In fact, the relative increase of strength caused by the external mesh is larger for thicker panels. It is possible that the mesh creates a confinement to the shotcrete which in turn enhances the strength of the shotcrete itself.
8.3 Suggestions for continued research

Based on the results obtained in this thesis, some suggestions are given below on further studies. A study of these questions would give a better understanding of the performance of the rock support system.

- Testing of panels with different shotcrete thicknesses and with different dimensions of steel mesh. The effects of a weaker steel mesh have not been studied for the combination shotcrete-mesh. At least for moderate deflections, it is likely that a weaker mesh could perform satisfactory.
- More tests on panels with embedded steel meshes to determine the influence fibers have on the performance of reinforced shotcrete.
- Testing of plain shotcrete panels (i.e. without steel fibers) in combination with external steel mesh to study the impact steel fibers have on the panel strength.
- Measure and determine the size of possible rock deformations in underground mines. Before any decisions are made to reduce shotcrete thickness or mesh dimensions, it is important to quantify the size of expected deformations in the rock as well as the deformation capacity of the rock support system.
- Measure and determine the deformation capacity for existing rock support systems.
- Dynamic tests in field and/or in laboratory to study the performance of the dynamic support system when exposed to dynamic loads. The results in this study were obtained from static tests. Dynamic tests on the support system together with rock bolts are needed in order to acquire a better understanding of the performance during seismic loads.
10 References

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