Numerical studies of mining geometry and extraction sequencing in Lappberget, Garpenberg

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Abstract

The core justification of this thesis was to delay expected failures by means of studying different sequencing patterns. This thesis reviews, analyzes and compares some of extraction methods and sequences used in steep orebodies. The subject of the research was proposed and supported by Boliden Mines and was conducted as a Master thesis in Luleå University of Technology.

Models were calibrated using data from observed failures on levels 880, 896 and 926 in Lappberget orebody. Hoek-Brown criterion with modified parameters for brittle rock and Stacey’s extension criterion were used in the process of analysis.

One major objective of the thesis is to decide the size of the stopes. For the purpose of comparison an index for crack propagation depth was introduced, the origin of the index is ELRD (equivalent linear relaxation depth) which was proposed as a tool to quantify the relaxation zone around excavations. Some modification was done on ELRD so it can be used for the goals of the project. Six variation of stope dimension was modeled in an isolated model. The comparison of stope dimension was studied in different sequencing patterns as well.

Finally, seven different sequencing methods were suggested and studied for exploiting the ore. The sequencing methods include primary-secondary patterns with three alterations of stope size, tertiary patterns with two variations of stope size, a pillarless pattern and a two-way pattern (top-down and bottom-up).
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1 Introduction

1.1 Background

Mining is, to a large extent, about moving enormous amounts of rock as economically as possible. This definition makes ore an economic concept which can be defined as concentration of minerals that can be exploited and turned into a saleable product to generate a financially acceptable profit under existing economic condition. It is essential to understand that ore does not properly exist until it has been labeled as such. The costs of extraction and market prices affect the volumes of ore and waste directly. Since the changes in market are not easy to predict, mining companies try to focus on minimizing the cost of production with the aim of gaining more profit. Several factors influence mining expenses e.g. mining technology, processing methods, mining method etc.

This thesis describes and explains different sequencing patterns for excavation of Lappberget orebody in Garpenberg, Sweden.

Once an ore body has been probed and outlined and sufficient information has been collected to warrant further analysis, the important process of selecting the most suitable method of mining can begin. The mining method is adapted to the rock conditions, shape, dimensions, strength and stability of the orebody. In addition to that, choice of an underground mining method must be tailored to the ground conditions if the mining operation is to be successful. The emphasis should be put upon objectively assessing suitable mining methods and choosing the method most compatible with ground conditions.

Currently transverse open stoping is in use in Lappberget, the thesis mostly concentrates on different stope sequencing and room sizes and the effect of variations of those factors on geotechnical attributes of the mine.

Detailed information about the method currently in use in Lappberget is presented in following chapters.
1.2 Introduction to Boliden group

Boliden Mines, commonly referred to as Boliden, is a Swedish mining company mostly concentrating on production of copper, zinc, lead, gold and silver. The company has approximately 4400 employees. The name comes from the Boliden mine located 30 km northwest of Skellefteå city where gold was found in 1924.

Boliden is one of the leading European metal companies in exploration, mining, smelting and recycling. Boliden operations are shown in Figure 1.1.

![Figure 1.1: Boliden operations map (Boliden, 2012)](image)

Aitik, situated outside the town of Gällivare in the very north of Sweden, is Sweden's largest copper mine and being operated as an open pit. Tara underground mine located in Ireland is Europe's biggest zinc mine and the world's ninth largest. The Boliden area is located in the mineral-rich Skellefte field, where almost 30 mines have been operated since the 1920s. Garpenberg area is located in Dalarna region of central Sweden and contains several orebodies including Lappberget which is the subject of this thesis.

Boliden mines production in year 2011 is shown in Table 1.1.
Table 1.1: Production of Boliden operations (after Boliden 2011)

<table>
<thead>
<tr>
<th>Mine</th>
<th>Milled Ore (ktonnes)</th>
<th>Metal content (t)</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Zn</td>
<td>Pb</td>
</tr>
<tr>
<td>Tara</td>
<td>2486</td>
<td>163935</td>
<td>19787</td>
</tr>
<tr>
<td>Garpenberg</td>
<td>1456</td>
<td>81068</td>
<td>28330</td>
</tr>
<tr>
<td>Boliden</td>
<td>1677</td>
<td>38214</td>
<td>1360</td>
</tr>
<tr>
<td>Aitik</td>
<td>31541</td>
<td>66876</td>
<td>2.44</td>
</tr>
</tbody>
</table>

1.3 Objective and scope of thesis

The thesis includes a study of the mining geometry and extraction sequences using linear elastic model by the help of Examine3D software with the purpose of acquiring the most productive and logical excavation room size for the new working level starting at 1250 to level 1100. Moreover, calibration of input data, modeling strategy and results by simulating previously mined area was conducted.

The failure observed in levels 926 and 896 caused the mine to lose its production, the idea of the thesis is to study the new ore block with the aim of identifying the failure and attempting to avoid, arrest or at least predicting the level that failure is expected to occur again by changing the mining sequencing plan. The study is based on numerical modeling leading to a choice of appropriate size and rapture effect for the rooms, and also the sequences of the mining method in order to achieve a high and safe production.

The work consists of following parts:

- Description of mining method in Lappberget and previous stress analysis calibration.
- Construction of numerical models.
- Study of stope geometry alternatives and their influence on stress distribution.
- Analysis of different excavation sequences and their effect on production rate.
- Study of production versus stress field/mine condition.
- Evaluation of results and preparation of recommendations.
1.4 Definition of mining sequencing

One of the main phases of mine design is to develop a plan for the orderly and optimal extraction of the reserve. The plan includes information about the sequencing of mining activity throughout the overall orebody. Steeply dipping orebodies provide the advantage of accessing separate areas of the orebody individually, which leads to more flexibility in the plan. Sequencing in mining defines the way in which mining progresses. Different factors play roles in designing mine sequencing and there are several goals to fulfill such as minimizing ground control issues, creating a sustainable extraction rate etc. Considering today’s competitive market, most managers agree that the overall objective of any long term mining plan is to maximize the net present of the mining investment. To make a plan more beneficial, following objectives are essential:

- To maximize the overall extraction percentage of the mineral resource.
- To minimize the initial development time and costs.
- To minimize or delay inevitable ground control costs and problems.

These factors can and will be in conflict. To maximize the economic returns, all of those objectives should be noted in the design procedure.

The in-situ stresses may require that stopes be mined in pre-determined pattern. This is especially critical for all mines when mining at depth. Mining stopes in sequences ensures that the stresses in the rockmass propagate away from the working heading. This is done to prevent high stress regions from developing in the unmined stopes, which can lead to rockburst or ground failure. Such an event could result in worker injury, damage to equipment, loss of recoverable ore or delay in production (Vick, 1983).

1.5 Methodology

Since there is almost always lack of geotechnical information, it is impossible to decide upon a mine sequencing until observations about the rockmass’ reaction to mining are made. With the use of failure observation in previously mined areas of the orebody, it was made possible to determine the optimal mine sequencing for this particular mine. The research was
conducted by primarily reviewing information available from observed failures in Lappberget mine and calibration of models using numerical analysis. A site visit was carried out to the mining areas in Lappberget. In addition, a thorough literature review was carried out on factors which influence the outcome of mining activities within the underground hardrock mining industry and also on historical and contemporary mine sequencing activities. Moreover, different failure criteria were examined to reveal which one shows better agreement with reality of the rockmass behavior. Afterwards several different strategies to extraction sequencing were studied and compared to develop an understanding of the effect of changing sequencing pattern and room sizes on expected failures in the mine.

Numerical modeling was carried out using Examine\textsuperscript{3D}. EXAMINE3D is a computer-aided engineering analysis package for underground structures, developed by the Rock Engineering Group, Department of Civil Engineering, University of Toronto. It currently includes modules for geometric modeling, surface meshing, elastic stress analysis based on the boundary element method, and data visualization/interpretation. (Rocscience, 2003)

1.6 Outline of the thesis

The outline of thesis is shown in Figure 1.2.

![Figure 1.2. Outline of the thesis.](image)

Introduction is followed by an overview of Lappberget, historical background of the area, location, geology, geotechnical environment and current conditions of the mine. Stress conditions and conducted measurements are presented in chapter 3, while chapter 4 discusses different failure criteria suitable for the resolution of the thesis. Chapter 5 encompasses
studies on calibration of models. Chapter 6 deals with estimation of optimal stope sizes and also defines sequencing patterns that were used in the research. Finally, Chapter 7 contains the discussion, conclusions and further suggestions for the project.

1.7 Contribution to original knowledge

Comparison of different sequencing patterns in transverse open stoping is not a widely discussed subject. Several researches, mostly by Potvin, were conducted about variety of patterns but not with the aim of comparing them; however, that is not the case for stope dimensions. Considerable work has been carried out regarding optimization of stope dimensions mainly based on Mathew’s method and stability graphs.

Anders Nyström (2005) conducted numerical analysis using Examine$^3$D with considerable variations of dimensions of the stopes to find an optimized room size for Lappberget but the research only focused on a primary-secondary pattern.

Marteen van Koppen (2008) also did his Master thesis about “Estimation of the Risk for Mine Induced Seismicity in Large Scale Mining in the Garpenberg Mine” which contains valuable information about the geology and history of the mine that were used in this thesis.

C. W. Pelley in his PhD thesis (1994) with the subject of “study of sequencing strategy for steep tabular orebodies” studied both historical and contemporary mine sequencing strategies and evaluated them in geotechnical, financial and operational objectives manners.
2 A review of the Garpenberg mine

2.1 Garpenberg mining history

Garpenberg area is located in Dalarna region of central Sweden, shown in Figure 2.1.

![Figure 2.1: Location of Garpenberg, the red balloon is the indicator (Google Maps 2012)](image)

Discovery of Garpenberg dates back to 13\textsuperscript{th} century. That makes it one of the oldest mines in Sweden. Historical evidences found in the area indicate that the area have been hosting mining activities since 380 BC. A major discovery was also made in Lapphyttan, approximately 40km south of Garpenberg, where a complete medieval industrial complex was found. The complex dates back to the 12\textsuperscript{th} century and had ore and coal storage facilities, smelters, blacksmiths, warehouses and accommodation facilities. Although iron and silver were mined too, the mining was mainly for cooper.

Before 1724, the extraction was made by using fire to heat the rock and pouring cold water afterwards over the rock to produce cracks inside the rock and make it more brittle which led to easier hammering it off the rock. Finally in 1724 explosives were introduced to the mines, this advancement in extraction brought the disadvantage of high risk in using explosives, until in 1860 Alfred Nobel stabilized nitroglycerin and introduced dynamite. Garpenberg mine was producing approximately 60 tonnes annually of raw copper in the middle of the 17\textsuperscript{th} century,
which made Garpenberg the second largest copper producer in Sweden. In 1848 all mining stopped in Garpenberg due to poor quality of the ore and in 1878 the smelter was shut down (Andersson, 2007).

In 1906 mining started over and in 1923 Zinkgruvor AB became the new owner of the Garpenberg operation. Boliden acquired Garpenberg in 1957 and exploration work has since resulted in a substantial increase in its ore reserves. Complex ore that contains zinc, copper, lead, gold and silver is being mined at Garpenberg. Since then, explorations conducted on the area led to discovery of several orebodies in the region, refer to Figure 2.2.

Figure 2.2: Aerial view of the Garpenberg operations. The main ore bodies and the connection drift are indicated. (After Söderman, 2011)

Figure 2.3 illustrates most of the orebodies in the area, the two major orebodies in Garpenberg are Dammsjön and Lappberget. The latter is the focus of this thesis.
Lappberget orebody was discovered in 1998. First ore was extracted from Lappberget in 2003. In 2005 Kvarnberget was discovered in Garpenberg.

A decision to carry out a major expansion of operations at Garpenberg was made in January 2011. A total of 3.9 billion SEK will be invested and will increase ore production at Garpenberg from the current level of 1.4 million tonnes to 2.5 million tonnes per annum. The expansion will be carried out between 2011 and 2014, with production successively increased, starting in early 2014. The mine is expected to reach full production capacity by the end of 2015.

2.2 Lappberget orebody

The Lappberget Zn-Pb-Ag sulphide orebody at the Garpenberg Mine comprises 20 Mt of massive and disseminated sulphide ore and 13 Mt of additional mineral resources. The deposit stood for 900 kt of the total 1.375 Mt production at Garpenberg in 2010. The Lappberget orebody was discovered in 1998 by following up an off-hole EM-anomaly from the previous year’s drilling (Allen et al, 2003). In 1997 a borehole was drilled from the ramp at 820 level in the southern direction. It hit a mineralized zone that later became the Kaspersbo orebody. The borehole was 100 meters long and it showed an anomaly in the borehole electromagnetic
survey (BHEM) at the end of the hole. The borehole unexpectedly encountered a wide limestone section that caused the drill hole to get such a length. An exploration drift – the U-drift – was driven on 910 level to further investigate the future Kaspersbo orebody and the anomaly afterwards.

In 2001 the exploration drift had come to a stage where the anomaly could be drilled. Three boreholes with the same azimuth were drilled towards 800, 900 and 1000 m in the direction of the anomaly. Each of those holes hit the Lappberget orebody. The BEHM surveys could neither indicate the top or the bottom of the ore. Drilling continued in 2002 and eventually BEHM surveys indicated the top of the mineralization to be around 450 to 500 meters elevation. Exploration drilling continues to present day both in the upper part and the deeper part of Lappberget. In addition to that, drilling is going on in the known areas to fine-tune the economic boundaries for stope and infrastructure design. (Fagerström, 2007)

Today Lappberget is estimated to be about 250 meters along the strike and ranging from 15 to 120 meters in width.

Mining of Lappberget began in March 2003 by cut and fill mining on the 870 level. In February 2007 longhole mining was introduced between 1060 and 1080 levels as the first transverse open stopes were excavated (Koivisto, 2008). The transverse open stoping completely replaced the cut-and-fill mining method around the end of 2010.

Exploration of Lappberget orebody has added significantly to available ore reserves. In 2005, Garpenberg produced 1.1Mt of ore grading 5.75% zinc, 2.28% lead, 0.09% copper and 117g/t silver. 40% of the produced ore came from Lappberget. Location of the orebody is shown in Figure 2.4
The ore is hauled from the production levels via footwall drifts located north of the orebody to the central crusher positioned at level 800 where it is connected through the north skip shaft to surface. Upon reaching on surface the ore is transported to the mill at the south shaft by tracks.

2.3 Mining methods

Several mining methods were applied in Lappberget throughout its life; a part of the orebody was excavated using Rill mining, a method similar to Avoca mining. Cut and fill mining was also used at level 881 to 822. From 2012 transverse open stoping is the only mining method in Lappberget. The methods are described further more in this section.

2.3.1 Post-pillar cut-and-fill mining levels 881-822

Extraction was planned to transpire by using cut and fill mining starting from level 881 and continuing to elevation 822. The method was fully replaced by transversal open stoping at the beginning of 2012. Figure 2.5 shows the area mined by cut and fill mining.
Post room and pillar mining is a combination of room-and-pillar and cut-and-fill stoping. With this method, ore is recovered in horizontal slices starting from bottom and advancing upwards. Pillars are left inside the stope to support the roof, pillars are shaped as vertical beams across the orebody and continue through several layers of fill. Pillars and the fill contribute to the pillar's supporting ability. Mined out stopes are backfilled and the next slice is mined by machines working from the fill surface. Figure 2.6 illustrates a post-pillar mining operation.

2.3.2 Transverse open stoping

Transverse open stoping is an open stoping method and is considered more efficient for mining wide steep orebodies such as Lappberget. In sublevel open stoping (SLOS) the orebody is divided into separate stopes, between which ore sections are set aside for pillars to support the hanging wall. Pillars are normally shaped as vertical beams across the orebody. Horizontal sections of ore are also left, as crown pillars (Atlas Copco, 2003).
As outlined above, when the orebody becomes too wide to be self-supporting in a single cut, the stopes are oriented perpendicular to the strike as transverse stopes.

In Lappberget, drifts are being developed parallel to the ore in the footwall (footwall drifts). The ore is reached by constructing cross drifts perpendicular to the footwall drift. Access is created to the stopes on the bottom to allow the mining of the stope. After the development, a box hole cut is made to provide a free face for the longhole blasting. The machineries in Garpenberg are capable of handling both down holes as well as uppers. Remote mucking takes place in the bottom drifts to prevent risky unsupported areas. Stopes are paste filled after excavation. Currently transverse stoping is applied by following a primary-secondary approach in Lappberget, stopes are 20 to 30 meters high and the width differs from 10 to 15 for primary and 15 to 20 meters for secondary stopes.

Figure 2.7 shows an illustration of mining sequences in Lappberget from level 1080 to 896.

Figure 2.7: mining sequencing in Lappberget LVL 1080

Figure 2.8 displays the same approach used in Williams mine in Ontario.
It should be mentioned that paste fill is used in primary stopes where for secondary stopes waste rock is being dumped into the rooms. Recovery of this approach is expected to be 100% while dilution is assumed to be close to 10% (Kuiper, 2008).

The paste fill requires a curing time of 7 to 10 days before it can be accessed on top. The cycle repeats on the stope above once the paste is sufficiently cured. The mining is scheduled in such a way that blasting of secondary stopes is always a long time after the neighboring primary stopes have been backfilled. The paste is produced at the paste plant on surface near the Garpenberg Norra shaft and is pumped to the stopes underground. The paste fill is a combination of the tailings produced by the mill and a binder. The tailings are pumped as slurry from the mill to the paste plant where the excess water is taken out (Marklund, 2002).

More information about the sequencing can be found in Chapter 6.

2.4 Geology

This section includes a brief description about the geology of the region. Since the author is not a geologist, presented information were directly translated from the work of Pia Fagerström, summary of earlier work by Allen et al. (2003) shown on the maps and interpretation by Franz Vyskytensky, Maarten van Koppen’s thesis is the main reference for the current section.
2.4.1 Geology of Lappberget

The Garpenberg area is thought to be a volcanogenic hydrothermal ore deposit that has been formed by a stratabound replacement process below the sea floor in a caldera vent of a larger shallow marine rhyolite-dacite volcano. The cross section on 4000 Y is shown in Figure 2.9 and shows the Lappberget anticline, but at the time of the analysis there was less information on this area as drill hole information was limited. (Allen et al, 2003)

![Cross section at 4000Y, Lappberget](image)

The exact folding mechanism of the Lappberget anticline is not entirely sure, the extreme height of the Lappberget anticline (as it is more than a kilometer) is, as of yet, unique to the Garpenberg syncline. Relatively high contents of copper and gold in joint infillings and a higher concentration of remobilized ore veins on the southern end of Lappberget suggest
tectonic shear zones in that area, similar to shear zones found towards the north. (Allen et al., 2003, Allen, 2008)

The general stratigraphy is shown in Figure 2.10. The general sequence in Lappberget from the top is:

- Young volcanogenic rocks and volcanic/clastic sediments
- Limestone, which is in contact with the ore.
- Ore, mainly sulphides, skarn and impregnated quartzite
- Old volcanogenic rock types, often fine-grained.

**Stratigraphy in Lappberget, starting at surface**

<table>
<thead>
<tr>
<th>Rock type</th>
<th>Grain size</th>
<th>Description</th>
<th>Transformation / Structures / Mineralization</th>
</tr>
</thead>
<tbody>
<tr>
<td>Pumice breccia</td>
<td></td>
<td>The pumice contains fragments of varying quantity</td>
<td>Red breccia and brown sulphide transformation, strong lithification and texturing</td>
</tr>
<tr>
<td>Glacial sediment</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Rhyolitic tuff</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Breccia conglomerate</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Andesite dyke</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Breccia conglomerate</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Basaltic pyroclastic</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Breccia conglomerate</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Tuff in beds</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Limestone breccia</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Limestone</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Limestone - dolomite</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Dolomite</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Massive sulphides</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Skarn</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Stilbite</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Phlogopite quartzite</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Several types</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Biotite quartzite</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>The pumice breccia contains fragments of varying quantity.</td>
<td>Red breccia and brown sulphide transformation, strong lithification and texturing</td>
<td></td>
<td></td>
</tr>
<tr>
<td>The pumice breccia contains fragments of varying quantity.</td>
<td>Red breccia and brown sulphide transformation, strong lithification and texturing</td>
<td></td>
<td></td>
</tr>
<tr>
<td>The pumice breccia contains fragments of varying quantity.</td>
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<td></td>
<td></td>
</tr>
<tr>
<td>The pumice breccia contains fragments of varying quantity.</td>
<td>Red breccia and brown sulphide transformation, strong lithification and texturing</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

**Figure 2.10: Stratigraphy in Lappberget. not to scale and grain size is relative (after Fagerström, 2007)**

The first rock type is the pumice breccia which is rhyolitic in chemical composition and is therefore very hard. The rock is brown-greyish but various transformations are known. The breccia-conglomerate has varying thickness up to several tens of meters. The fragment sizes vary from a few centimeters up to several decimeters. The main part of the rock type is limestone and it often occurs in large fragments. Within the breccia-conglomerate distinct layers of fine-grained grey siltstone layers consisting of quartz, feldspars and biotite are
commonly found without any apparent mechanism. The thickness of these layers varies from 5 centimeters up to 2 meters.

Sulphides are commonly present in the matrix of the breccia-conglomerate, especially in the bottom part of the formation that indicates the proximity of the ore mineralization. The sulphides appear irregularly as an impregnation of iron sulphides (pyrite, pyrrhotite) and zinc sulphides ( sphalerite). Some massive sulphides have been found in drill cores (often high grades of sphalerite), but it is unclear where or when those sulphides originate.

Most heterogeneous clastic sediments occur in the syncline south of the Lappberget anticline, stratigraphically under the pumice formation. The transition between the clastic sediment and the pumice is often unclear. Clastic sediments occur as layers in the breccia-conglomerate as well.

Dacite is a half understood sub-volcanic rock type that has intruded through the hanging wall up to the end of the fault structure. The dacite looks homogeneous and grey with fine flat grains, but often porphyritic with plagioclase crystals up to 1 mm and mafic crystals up to 2 mm.

Next rock type is limestone breccia, which is the transition between the breccia-conglomerate and the limestone. The breccia has fragments from a single source and is often brecciated in-situ. The matrix is a fine-grained silt material, sometimes with irregular quartz concentrations or clasts.

Limestone is a medium grained white, clean, metamorphically recrystallized rock type that really should be called marble. In some places it shows orange to salmon-red colors. The limestone can have small layers of siltstone material and in some places quartz minerals mixed with calcite can be found. (Fagerström, 2007)

The preparation of ramps, trails and cross-towns will be performed in the limestone that can be classed as good to very good rock. (Nyström, 2005)

There is a coarse metal zoning in the massive sulphide ore in Lappberget. It is rich in silver in the top part, dominated by zinc in the middle and possibly higher grades of copper and gold in the deeper part. Otherwise the zinc-lead-silver ore is generally found in the entire Lappberget orebody as this is the main type of ore.
2.4.2 Geological zones

The orebody can be divided to three zones according to their geological features. Zone A and B are massive sulphide ores and are very similar in mineralogy, stratigraphy and in grades. The C zone is in another stratigraphic region and has lower grades. Plan view of zone locations are shown in Figure 2.11. (Nyström, 2005)

![Diagram showing the locations of A, B, and C zones](image)

Figure 2.11: Location of distinct A, B and C on level 900. Red is compact ore, blue is limestone and yellow is quartzite or tuff. (After Fagerström, 2003)

The massive A zone is richest and most persistent in both height and width of the three ore zones. The A zone is confirmed between 650 and 1300 elevation, but core drilling is on-going to further define the ore body outline. The A zone is 150 meters long and 20 to 40 meters wide on 900 elevation for example. The mineralization took place in the limestone/dolomite and in the contact underneath the limestone.

The massive B zone is similar to the A zone in texture, mineralogy and stratigraphic position (the same limestone/dolomite base). The B zone is a separated area (as indicated on Figure 2.11), probably caused by folding or shearing mechanisms in the limestone.

On the hanging walls’ ore contact at both the A and C zone there are some deformation zones that contain talc as well as mica minerals such as biotite and chlorite (Figure 2.12). These zones are typically 1 to 5 meters wide and impose significant issues with the hanging wall stability. (Fagerström, 2007)
2.5 Geotechnical environment

Geotechnical characteristics of the rock mass are considered to be vital information in order to make an acceptably precise model.

Nyström (2005) provided analyzed results of 28 cores with total length of 5849 meters gathered from holes with different locations and orientations spread over Lappberget.

Point load testing was conducted on the cores gained from three holes. Testing was done by diametrical loading only; results are presented in Figure 2.13. Some weaker rock types were found throughout the test but majority of the results show UCS of 100 MPa and higher.

In order to obtain a point load index following equations were used:

\[(Eq. \ 2.1) \quad I_s = \frac{P}{D_n}\]
Where $D_e$ is equivalent core diameter calculated by Equation 2.2 for diametric test and $P$ is the recorded load.

(Eq. 2.2) \[ D_e = D^2 \]

Where $D$ is the core diameter. The obtained point load index, $I_s$, is valid for the tested core diameter. To simplify comparison with other tests with different core diameters, the point load index is corrected to a standard size — a 50-mm diameter core. This standard diameter point load index is written as $I_{s(50)}$. The corrected point load index is calculated from:

(Eq. 2.3) \[ I_{s(50)} = F \cdot I_s \]

(Eq. 2.4) \[ F = \left( \frac{D_e}{50} \right)^{0.45} \]

The corrected point load index can be approximately correlated to uniaxial compressive strength, as follows:

(Eq. 2.5) \[ \sigma_c = 22 \cdot I_{s(50)} \]

![Figure 2.13: UCS gained from diametrical point load testing (After Nyström, 2005)](image-url)
Figure 2.14 shows the RMR of mapped cores, the overall results of the core logging can be found in Table 2.1

<table>
<thead>
<tr>
<th>Class</th>
<th>RMR</th>
<th>% of total mapped length</th>
</tr>
</thead>
<tbody>
<tr>
<td>Very poor rock</td>
<td>0-20</td>
<td>0</td>
</tr>
<tr>
<td>Poor rock</td>
<td>20-40</td>
<td>1</td>
</tr>
<tr>
<td>Fair rock</td>
<td>40-60</td>
<td>20</td>
</tr>
<tr>
<td>Good rock</td>
<td>60-80</td>
<td>39</td>
</tr>
<tr>
<td>Very good rock</td>
<td>80-100</td>
<td>40</td>
</tr>
</tbody>
</table>

In addition to provided data, the results from 1250 meters of core, analyzed by point testing for the purpose of conducting a feasibility study on a new shaft close to Lappberget is shown in Table 2.2. Results indicate high strength of the rock that can be estimated in the range of 100-250 MPa. RMR mapping points out that most of the examined rock can be categorized as good rock (refer to Table 2.3).
Table 2.2: Point load test along shaft outline. (Nyström, 2008)

<table>
<thead>
<tr>
<th>Rock Type</th>
<th>No of Samples</th>
<th>UCS (MPa) + St. Dev.</th>
</tr>
</thead>
<tbody>
<tr>
<td>Tuffite</td>
<td>6</td>
<td>191 ± 52</td>
</tr>
<tr>
<td>Dacite</td>
<td>1</td>
<td>176</td>
</tr>
<tr>
<td>Sandstone</td>
<td>3</td>
<td>207 ± 58</td>
</tr>
</tbody>
</table>

Table 2.3: RMR-rating from logging of drill hole 1757 and 1774 (Nyström, 2008)

<table>
<thead>
<tr>
<th>Class</th>
<th>RMR</th>
<th>% of total mapped length</th>
</tr>
</thead>
<tbody>
<tr>
<td>Very poor rock</td>
<td>0-20</td>
<td>0</td>
</tr>
<tr>
<td>Poor rock</td>
<td>20-40</td>
<td>0</td>
</tr>
<tr>
<td>Fair rock</td>
<td>40-60</td>
<td>1</td>
</tr>
<tr>
<td>Good rock</td>
<td>60-80</td>
<td>70</td>
</tr>
<tr>
<td>Very good rock</td>
<td>80-100</td>
<td>29</td>
</tr>
</tbody>
</table>

Joint mapping was carried out in 2003 in the area. Observations showed that generally speaking, zone A has very few joints. The weaker zones are steeply dipping and are commonly found near the hanging wall contact. These zones flank zone B on both sides. In zone C no strong weakness zone was identified other than the jointing parallel to the foliation. Two sub-vertical joint concentrations were identified. They are consistent with the change of orientation of the foliation from zone A to zone C; these have the orientations of N46E/90 and N72E/90. Some locally sub horizontal joints were also found (Nyström, 2005).

Only available data regarding elastic measurements of the rock types in Lappberget comes from Dahle (2005) and Nilssen (2004). Young’s modulus, Poisson’s ratio and UCS values were measured using biaxial test, results are presented in Table 2.4 and Table 2.5.

Table 2.4: Biaxial test results on limestone (After Nilsson, 2004)

<table>
<thead>
<tr>
<th>Location</th>
<th>E-modulus (GPa)</th>
<th>Poisson’s ratio</th>
<th>UCS (MPa)</th>
<th>Density (kg/m³)</th>
</tr>
</thead>
<tbody>
<tr>
<td>LVL 883</td>
<td>55</td>
<td>0.17</td>
<td>73</td>
<td>2714</td>
</tr>
<tr>
<td>LVL 967</td>
<td>60</td>
<td>0.12</td>
<td>100</td>
<td>2722</td>
</tr>
</tbody>
</table>

*Locations of the holes are shown in chapter 3, Figure 3.3 and Figure 3.4

Table 2.5: Biaxial test on the ore (After Dahle, 2005)

<table>
<thead>
<tr>
<th>Location*</th>
<th>E-modulus (GPa)</th>
<th>Poisson’s ratio</th>
<th>UCS (MPa)</th>
<th>Density (kg/m³)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Hole 1</td>
<td>84.7 ±10.4</td>
<td>0.15 ±0.03</td>
<td>196 ±65.1</td>
<td>3331</td>
</tr>
<tr>
<td>Hole 2</td>
<td>90.7 ± 15.6</td>
<td>0.15 ± 0.04</td>
<td>146 ±14.0</td>
<td>3270</td>
</tr>
</tbody>
</table>

*Location of the holes are shown in chapter 3 figure Figure 3.6
3 Stresses

Rock stress measurements provide input data and can be used for calibration of numerical models. For such application, the common practice is to locate the measuring sites remote from the stope areas, so that the measured stresses are not affected by mining activities. Determined virgin stresses are then used directly as input for the numerical models (Borg et al, 1984).

In order to estimate stresses in Lappberget, results from strain measurements were used to back calculate the stresses using elastic relationships. By rotation of the stress components the orientation and size of the principal stresses (either three-dimensional or two-dimensional in the plane of the measurements) were determined. Two series of stress measurements have been taken inside or in close proximity to Lappberget by SINTEF from Norway. The measurements will be described later in this chapter.

It should be noted that results from undertaken stress measurement program cannot be defined by a single stress state but a range in a stress tensor, and also in case of deep excavations large scale back analysis methods often provide more reliable results (Martin et al, 2003).

Martin et al. (2003) suggests that during the preliminary design stages of a project it is often adequate to use stress information that has been accumulated on previous projects to estimate the regional stress state. For that purpose, results from both overcoring and hydrofracturing measurements databases that have been established for the Scandinavian shield can be used (data from Scandinavian shield extends to depth of 800 meters). Measurements are mostly located near the ground surface and few measurements at depths below 500 meters. Hence using databases trends to predict in-situ stresses at depth can be statistically challenging.

Both measured stresses in the mine and stresses according to the Scandinavian shield database are presented in this chapter.

3.1 Stress estimation from databases

Vertical stress at a depth $D$ is gravitational and is a product of depth and the unit weight of the overlying rockmass. The density $\rho$ for Lappberget varies between 2700 kg/m$^3$ and 3300 kg/m$^3$. The gravitational stress is often estimated from the following equation:
(Eq. 3.1) \[ \sigma_v = \rho g D \]

While the vertical stress can be estimated by the weight of the overlying rocks, significant deviation from this mean should be anticipated and this deviation can be less than the weight of the overburden.

In the Scandinavian Shield, the coefficient of variation for the linear relationship given in Eq. (3.1) varies significantly with depth (Figure 3.1). At depths greater than 500 m COV is less than approximately 20% but exceeds 100% at more shallow depth. The range of anticipated values can only be narrowed by stress measurements and/or back-analysis.

![Figure 3.1: Comparison of the average vertical stress gradient with the mean, using the vertical stress from the Scandinavian shield stress database (After Martin et al, 2003)](image)

Estimation of the horizontal stresses are much more difficult to predict compared to vertical stresses, Figure 3.2 shows the magnitude of in-situ horizontal stresses data for the Scandinavian shield.
Anticipated values for stresses according to above diagrams are presented in Table 3.1. Vertical stresses calculated using average density of 3000 kg/m$^3$.

Table 3.1: Stress calculation according to Scandinavia shield database

<table>
<thead>
<tr>
<th>Level</th>
<th>$\sigma_V$ (MPa) ± 20% Dev.</th>
<th>$\sigma_H$ (MPa)</th>
<th>$\sigma_h$ (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>883</td>
<td>26.5±5.3</td>
<td>36.3</td>
<td>19.9</td>
</tr>
<tr>
<td>967</td>
<td>29.0±5.8</td>
<td>39.4</td>
<td>21.6</td>
</tr>
<tr>
<td>1250</td>
<td>37.5±7.5</td>
<td>49.5</td>
<td>27.3</td>
</tr>
</tbody>
</table>

3.2 Stress measurements

Several measurements were conducted in regions nearby Lappberget using overcoring and doorstoppers, detailed information are presented in following section.

3.2.1 Overcoring

The stress measurements were carried out using the overcoring technique by SINTEF on levels 883 and 967; sample cores were taken from the actual measuring holes. The diameters of the samples were 62 mm and a length/diameter ratio was 2.5. Samples were tested using biaxial loading (Nilsson, 2004).
Location of overcoring is shown in Figure 3.3 and Figure 3.4 for levels 883 and 927 respectively.

Figure 3.3: Location and orientation of stress measurement level 883. (After Nyström, 2005)

Figure 3.4: Location and orientation of the measurement at level 967 (After Nilsson, 2004)

Figure 3.3 shows the location of an electric workshop where measurement took place. Previously studies using two-dimensional numerical analysis revealed that the excavation did not influence horizontal stresses as for the vertical stresses the value increased by almost 10% due to disturbance of stresses after excavation. Measurement was approximately perpendicular to the drift orientation (N150E) with a 6 degree inclination and a length of 18.5 meters. (Nilsson, 2004)
Measurement on level 967 was conducted in foliated limestone with few joints. The foliation has the approximate orientation of the orebody with the strike and dip of N060E/75SE. The orientation of the hole was identical to the hole in level 883. A sketch of the location can be seen in Figure 3.4.

Summarized stress results are shown in Table 3.2 while Figure 3.5 illustrates the orientation of stresses. By comparing the results from measured stresses and calculated stresses according to Scandinavia shield presented in Table 3.1 it can be seen that for level 883 estimated values for vertical stress and minor horizontal stress are close to measured. However for major horizontal stress, measured value is higher than anticipated. As for level 967 measured values are significantly lower than calculated ones and although the measurement was conducted on a lower level, stresses reduce with increasing depth.

<table>
<thead>
<tr>
<th>Level</th>
<th>( \sigma_V ) (MPa)</th>
<th>( \sigma_H ) (MPa)</th>
<th>( \sigma_h ) (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>883</td>
<td>23.8</td>
<td>45.2</td>
<td>20.2</td>
</tr>
<tr>
<td>967</td>
<td>17.8</td>
<td>26.2</td>
<td>10.7</td>
</tr>
</tbody>
</table>

Figure 3.5: Stereographic representation of the states of stress at 967 (left) and 883 (right). (Nilsen, 2004). Orientations with respect to the true North, not local mine coordinate system.

It is not easy to explain the reason of stress deviations and the large differences between 967 and 883 elevations. Induced stresses from nearby excavations might play a larger role than expected. There could be a significant difference between the elastic properties of the massive sulphide ore and the host rock that could cause stress accumulations in certain areas in the
host rock near the ore contact. From experience in Scandinavia, it is very common to see the main principal stress component in (sub-) horizontal direction (Nilssen, 2004).

### 3.2.2 Doorstoppers

In doorstopper measurements method, the strain is converted to stress after a sample is distressed. Elastic relationships and laboratory test results of rock samples are needed for calculation of stresses (Dahle, 2005). It should be mentioned that the state of stress at the end of the borehole is not equal to in-situ stress as the borehole itself imposes, albeit small, induced stresses. Correction factors based on the Poisson’s ratio are available and should be applied.

The measurements took place in two post-pillars on 852 elevation in the cut-and-fill post-pillar mining part of Lappberget (see Figure 3.6). First hole is 6.5 meters deep and measurements were conducted in 8 points from 0.4 to 5.6 meters along the hole. As for the second hole the depth is 7 meters with 7 reading points between 0.6 and 6.5 meters along the hole. The measurement at 6.5 meters was unsuccessful and is therefore discarded in the calculations of the total load on the pillar. The cores recovered from the holes showed only one major joint in the first hole. This joint had clay infilling (Dahle, 2005).

![Figure 3.6: Location of the doorstopper measurements at level 853. (After Dahle, 2005)](image)
Uniaxial loading test was done on recovered cores. The average values of the different parameters are summarized in Table 3.3 (Dahle, 2005).

<table>
<thead>
<tr>
<th>Location</th>
<th>E-modulus (GPa)</th>
<th>Poisson’s ratio</th>
<th>UCS (MPa)</th>
<th>Density (kg/m³)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Hole 1</td>
<td>84.7</td>
<td>0.15</td>
<td>196</td>
<td>3331</td>
</tr>
<tr>
<td>Hole 2</td>
<td>90.7</td>
<td>0.15</td>
<td>146</td>
<td>3270</td>
</tr>
</tbody>
</table>

These values are to be used with some caution as the Young’s modulus varied over the length of the borehole. The results provide a good impression of the state of stress in the pillars nonetheless (see Figure 3.7) (Dahle, 2004).

![Figure 3.7: Stress profiles along hole 1 (left) and hole 2(right) (After Dahle, 2005)](image-url)
4 Failure criteria

Failure criterion provides a formula enabling prediction of the level at which failure will occur, therefore, right choice of the criterion is the essence of the project and regarding this decision, the reliability of results and its consistency with reality can be affected.

A failure criterion is merely a way to estimate the time of instability occurrence. For a failure criterion to be useful, it should be based on the correct mechanism. Failure mechanism attempts to describe the process leading to the failure. Failure mechanisms are seldom evident and, hence, need to be identified from observations, literature surveys, knowledge of the material characteristics, measurements and past experience (Sjöberg, 1993).

In hard rock, failure around the openings is a function of stress magnitudes and the properties of the rockmass. The failure process is controlled by the continuity and distribution of the natural fractures in the rockmass for shallow excavations, however as in-situ stress magnitude rises with the depth increase; failure process is dominated by new stress induced fractures growing parallel to the excavation boundary, and the fracturing is generally referred to as brittle failure (Martin et al, 1999). Furthermore, the failure process in hard rock may occur in a violent manner with sudden release of stored strain energy. This process is often referred to as violent spalling or strain bursting. Knowing the likely failure process can aid the designer in choosing an appropriate strategy for support design and the excavation sequencing (Martin et al, 2003).

The most common failure mode in hard brittle rock in high stresses is spalling which is more or less independent from geology, where thin slices split and fall out from the stope roof. It should be mentioned that numerical modeling of spalling suffers from particular problems, since the failure is progressive and current designing software are not capable of simulating the ongoing process, the correct representation of the mechanisms behind failure cannot be obtained. Failure is most likely to start as extension strain fracturing through intact rock (Sjöberg, 1993).

In addition to spalling, rockburst and violent failures were observed on the pillars and in cross cuts in Lappberget, more detailed information about observed failures is presented in chapter 5.
Several failure criteria were examined throughout the study to have a good understanding of the failure and to find out the most correspondent criteria to reality of the mine. Eventually Stacey’s extension strain criterion (Stacey, 1981) and Hoek-brown modified brittle parameters (Martin et al, 1999) were chosen for the purpose of the study. Martin’s criterion is based on stresses while Stacey’s is built upon extension strains.

4.1 Hoek-Brown failure criterion with Martin’s modified parameters for brittle rock

Attempts to predict either the onset of brittle failure process or the maximum depth of propagation, using traditional failure criteria based on fictional and cohesive strength mobilized simultaneously have not met with success (Wagner 1987; Pelli et al. 1991; Martin 1997).

Several studies on different rockmass types and conditions revealed that the initiation of brittle failure and forming of microscale fracturing occurs when the ratio of maximum tangential boundary stress to the laboratory unconfined compressive strength exceeds 0.4±0.1. For both Mohr-Coulomb and Hoek-Brown criteria, it is assumed that the cohesive (c or s) and the frictional (Φ or m) strength components are mobilized simultaneously. However Martin et al. (1999) proposed that in the brittle failure process peak cohesion and friction are not mobilized together, and that around excavations in brittle rock, the initial damage process is dominated by loss of the intrinsic cohesion of the rock such that the friction strength component can be ignored for estimating the depth of brittle failure.

Hoek-Brown empirical failure criterion is expressed as below:

\[
\sigma'_1 = \sigma'_2 + \sqrt{m \sigma_c \sigma_3 + s \sigma_c^2}
\]

\[
m = m_t \exp \left( \frac{RMR - 100}{28} \right)
\]

\[
s = \exp \left( \frac{RMR - 100}{9} \right)
\]

Where \(\sigma'_1\) and \(\sigma'_2\) are the maximum and minimum effective stress at failure, \(\sigma_c\) is the laboratory uniaxial compressive strength, \(m_t\) is the value for intact rock and RMR is the rock mass rating, where \(m\) and \(s\) are empirical constants based on the rock mass quality. For the boundary of an excavation, where \(\sigma_3=0\), Eq. 4.1 reduces to:
(Eq.4.4) \[ \sigma_1 = \sqrt{s \sigma_c^2} \]

When failure occurs in intact rock (s=1), \( \sigma_1 \) should be approximately equal to \( \sigma_c \). However according to Read and Martin (1996), in near boundary condition failure occurs when \( s=0.25 \sim 0.2 \) such that \( \sigma_1 \approx 0.5 \sigma_c \). Martin (1997) suggests that the difference between the laboratory strength and in-situ strength is a result of difference in loading path and that \( \sigma_c \) is not a material property but is controlled by the boundary condition of the measured sample.

Martino and Chandler (1994) also showed that formation of extension cracks reduces the intrinsic cohesion of the intact rock by about 70% as the friction is fully mobilized. The constant stress equation proposed by Martin can be expressed in term of the Hoek-Brown parameters as:

(Eq.4.5) \[ \sigma_1 = \sigma_3 + \sqrt{s \sigma_c^2} \]

By setting frictional constant \( m \) to zero to reflect that the frictional strength component has been mobilized, Martin (1999) suggests that by assigning \( s=0.11 \), the failure envelop shows a good agreement with observed failures in brittle rocks. Eq. 4.5 implies that the stress-induced brittle failure process is dominated by cohesion loss caused by the growth of extension cracks near the excavation boundary. Stacey (1981) found that for most brittle rocks the critical strain for extension fracturing was only slightly dependent on confining stress and occurred in the region of \( 0.3 \sigma_c \). Hence, both Martin’s and Stacey’s criteria are based on the same mechanistic model.

It is essential to understand that Equation 4.5 only describes the locus of damage initiation, and does not describe the limit of damage evolution, i.e. the extent of slabbing process. Equation 4.5, therefore, provides an estimate of the limiting depth to which slabbing can propagate but not the shape of slabbing region.

### 4.2 Stacey’s extension strain criterion

Previously Nyström (2005) used Martin’s criterion to predict the stress state at which crack forming in the orebody of Lappberget initiates, in the same report calibration of the used criterion and actual results from the nearby workshop was included which indicated good agreement with reality. On the other hand, Borg et al. (1984) used strain failure criterion to
predict the stability of Zinkgruvan mine, they found using elastic stress analysis and an extension strain criterion anticipates the potential fracture zones around large underground excavations with better agreement to the observed fracture zones than those predicted conventionally. An earlier attempt to predict the depth of brittle failure around a tunnel in massive quartzite was carried out by Stacey (1981) and led to forming Stacey’s extension strain criterion which is particularly applicable in areas of low confining stresses such as boundaries of an excavation.

Fracture is the failure process by which new surfaces, in the form of cracks, are formed in a material or existing crack surfaces are extended. The formation of fractures within a material does not imply failure of that material. According to Stacey (1981) “Fracture of brittle rock will initiate when the total extension strain in the rock exceeds a critical value which is characteristic of that rock type.” The previous statement can be presented mathematically as follows: fracture initiates when

\[ e_t > e_c \]

\( e_c \) symbolizes the critical value of extension strain. The fracture will form in planes normal to the direction of the extension strain, which corresponds with the direction of the minimum principal stress. For a material with ideal deformation behavior, the strain in this direction is related to the three principal stresses by the following expression:

\[ e_3 = \frac{1}{E} [\sigma_3 - \nu(\sigma_1 + \sigma_2)] \]

Where \( \sigma_1, \sigma_2 \) and \( \sigma_3 \) are the major, intermediate and minor principal stresses, \( E \) is the modulus of elasticity and \( \nu \) is Poisson’s ratio. From this it can be seen that extension fractures can also form when all three principal stresses are compressive. Moreover, the effect of the intermediate principal stress is taken into account unlike Martin’s criterion.

The critical value of extension strain can be obtained from the tests representing the strain at which extension fractures initiate. In addition to that Stacey (1981) gathered the critical magnitudes of extension strain for various rock types based on laboratory test which are summarized in Table 4.1. The value of critical extension strain can be obtained by using back calculation and calibration of the model with existent failures. Moreover, the critical extension strain can be estimated from the following equation (Ndluvo, 2007):

\[ e_c = \frac{f + UCS + \nu}{E} \]
Where E is Young’s modulus, ν is Poisson’s ratio and \( f \) is a correction factor that varies between 0.25 and 0.3 for crack initiation stress.

<table>
<thead>
<tr>
<th>Rock Type</th>
<th>Specimen length/diameter</th>
<th>Critical extension strain</th>
</tr>
</thead>
<tbody>
<tr>
<td>Quartzite A</td>
<td>2</td>
<td>0.000120</td>
</tr>
<tr>
<td>Quartzite B</td>
<td>2</td>
<td>0.000109</td>
</tr>
<tr>
<td>Quartzite C</td>
<td>2</td>
<td>0.000081</td>
</tr>
<tr>
<td>Quartzite D</td>
<td>2</td>
<td>0.000107</td>
</tr>
<tr>
<td>Quartzite E</td>
<td>2</td>
<td>0.000130</td>
</tr>
<tr>
<td>Lava A</td>
<td>2</td>
<td>0.000152</td>
</tr>
<tr>
<td>Lava B</td>
<td>2</td>
<td>0.000138</td>
</tr>
<tr>
<td>Diabase</td>
<td>2</td>
<td>0.000175</td>
</tr>
<tr>
<td>Norite</td>
<td>2.5</td>
<td>0.000173</td>
</tr>
<tr>
<td>Conglomerate A</td>
<td>2</td>
<td>0.000086</td>
</tr>
<tr>
<td>Conglomerate B</td>
<td>2</td>
<td>0.000073</td>
</tr>
<tr>
<td>Conglomerate C</td>
<td>2</td>
<td>0.000083</td>
</tr>
<tr>
<td>Sandstone</td>
<td>2</td>
<td>0.000090</td>
</tr>
<tr>
<td>Shale A</td>
<td>2</td>
<td>0.000116</td>
</tr>
<tr>
<td>Shale B</td>
<td>2</td>
<td>0.000150</td>
</tr>
<tr>
<td>Shale C</td>
<td>2</td>
<td>0.000095</td>
</tr>
</tbody>
</table>

The level of critical strains are very low and in a uniaxial compressive test correspond to a stress level in the region of 30% of the UCS which agrees with suggested value of stress for crack initiation by Martin. At higher values of macro extension strain, continuous fractures are likely to form, causing the progressive failure of brittle rock.

Including Martin (1997), Martin et al. (1999), Diederichs (2003) and Eberhardt et al. (2004) are agree to use an extension strain criterion while others like Kuijpers (2000) and Ryder and Jager (1999) have questioned the use of extension strain criterion. Martin (1997) noted that although the application of the extension strain criterion was successful, it is still not obvious what fundamental mechanisms control the failure of brittle rock.

For the purpose of this thesis both Martin’s criterion and extension strain criterion were studied to determine the reliability of these criteria and their consistency with reality of Lappberget.
5 Model calibration

The numerical analysis can assist in simulation of forced conditions on the rockmass and its response behaviors; this brings the advantage of predicting possible failures by identifying the observed mechanisms that govern failure in similar conditions. Calibrating against an observed failure mode provides a tool which can be used to establish guidelines for numerical modeling and choice of input parameters for future design of the mine. While a failure criterion is merely a way to calculate when failure will occur, the process of selecting suitable models and determining input parameters can develop a good understanding of the governing mechanisms (Sjöberg, 1993).

In January 2012 two separate failures were observed by Anders Nyström during a visit to Lappberget on levels 896 and 926 (Pictures of the failures are available in Appendix A).

On level 896 failure was observed as initial cracks on the floor of one of the footwall drifts which in a short time deformed to a step with almost one meter height and several meters strike length. The area was still seismically active at the time of author’s visit several months later. In addition to that, limited slabbing on the roof of the footwall was also observed. The size of the boulders found on the mentioned level varied from basketballs to one meter in each dimension.

A characteristic of stress-induced failure of excavations in brittle rock is the notched-shape of the failure region which is the case for level 926 on the roof of primary stopes. The caving in some stopes reaches the height of 12 meters at the vertex of the notches.

It should be mentioned that the failure has a progressive manner, more realistic modeling approach is to remove all of the failed elements according to the failure criteria, although that is not the strategy used in this thesis as a result of lack of time and limitations of Examine\textsuperscript{3D}.

The easiest way to obtain a correlation between failure modes and stress conditions is to use a simple linear elastic numerical model and to calculate stresses correspondent to observed failures. Martin (1997) showed that the brittle failure process initiates near the excavation face and hence is three-dimensional. Since the stress analysis itself is very important especially when it is disturbed by complex mining activities and also due to lack of post-peak strength parameters of rockmass, a 3-D elastic analysis can was chosen to perform the analysis.
Examine\textsuperscript{3D} as an elastic three dimensional analysis code was used for the numerical analysis software.

These newly formed failures on levels 896 and 926 were decided to be used for calibration of input data in the model generated for analysis in Examine\textsuperscript{3D}, so the results from the model can be compared to reality. In order to achieve this goal, two branches of models for each failure simulation was studied; first model branch was constructed considering the condition of excavated and working stopes at the time of failure occurrence and the second model branch was built with the assumption of failure in secondary stopes, the reason of making a model with excavated secondary stopes is the limitations of elastic analysis; in reality some of the secondary stopes will fail or lose a part of their bearing capacity, but since the software is not capable of simulating the phenomenon, secondary stopes have been modeled as excavated, which means no load will be carried by them.

Different uniaxial compressive strengths for the intact rock were assumed in the range of 120 to 200 MPa to determine the probable UCS. Furthermore, several failure criteria were used in order to study their agreement with observed failure.

Two approaches were used for loading, constant loading by assigning the actual measurements from the mine to the model (refer to Table 3.2) and gravitational loading with varying K-ratio; the gravitational loading was defined according to following equations:

\begin{align*}
(Eq.5.1) & \quad \sigma_v = \sigma_h = \rho gz \\
(Eq.5.2) & \quad \sigma_H = K \sigma_v
\end{align*}

Eventually the calibration of recent failures in addition to previous studies by Nyström (2005) helps the model to simulate the failure mechanism and predict its chance of occurrence with more reliability and consistent with expectations in reality. Although the results of calibration cannot be verified fully due to lack of measurements, it can be used to simulate a condition acceptably close to reality.

5.1 Footwall drift on level 896

Calibration with floor heaving can be useful to rectify in-situ stresses and UCS that is used as the input data for modeling. As it was mentioned before, two different conditions were
considered for calibration; firstly the situation in which all the excavated and ongoing excavation platforms were modeled and secondly an approach with secondary stopes modeled as excavated to simulate the failure in secondary stopes surrounded by excavated rooms. In Figure 5.1 gray area represent excavated stopes, red characterizes secondary stopes that are assumed to be failed, and green signifies currently open stopes.

Figure 5.1: Stope condition at the time of failure in level 896

Figure 5.2 demonstrates the location of observed failures in the mine while Table 5.1 provides details about the type of failure in limestone footwall; different locations are referred to by Roman numerals.
Figure 5.2: Location of observed failures on level 896

Table 5.1: Observed failures on level 896

<table>
<thead>
<tr>
<th>Number</th>
<th>Failure Indicator</th>
</tr>
</thead>
<tbody>
<tr>
<td>I</td>
<td>Spalling on the roof/ small slabs</td>
</tr>
<tr>
<td>II</td>
<td>Violent failure on pillar nose/rockburst/ small slabs to big boulders/ejection of grouted rebar into the access drift</td>
</tr>
<tr>
<td>III</td>
<td>Progressive heaving on the floor/ almost 1 meter</td>
</tr>
<tr>
<td>IV</td>
<td>Big failure on the face of the drift/ small slabs to big boulders</td>
</tr>
<tr>
<td>V</td>
<td>Pillar nose destruction/ Depth of failure near 1 meter</td>
</tr>
<tr>
<td>VI</td>
<td>Spalling and rockburst from 1 meter above the floor</td>
</tr>
</tbody>
</table>

Figure 5.3 and Figure 5.4 show the model used for analysis in Examine^3D^:
Figure 5.3: Examine 3D model for studying level 896 – Actual mined stopes

Figure 5.4: Examine 3D model for studying level 896 – secondary stopes failed

It should be mentioned that mined areas above level 896 in the model had been hidden in the following figures in order to provide explicit representations of the results.

Parameters sensitivity studies were conducted on the models by changing the value of UCS and K-ratio; Table 5.2 presents the model schemes. Young’s modulus and Poisson’s ratio were decided upon the information provided in Table 2.4.
Table 5.2: List of model schemes for study of the footwall drift

<table>
<thead>
<tr>
<th>Scheme</th>
<th>UCS (MPa)</th>
<th>E (GPa)</th>
<th>v</th>
<th>K-Ratio</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>100</td>
<td>55</td>
<td>0.17</td>
<td>1.5</td>
</tr>
<tr>
<td>2</td>
<td>150</td>
<td>55</td>
<td>0.17</td>
<td>1.5</td>
</tr>
<tr>
<td>3</td>
<td>200</td>
<td>55</td>
<td>0.17</td>
<td>1.5</td>
</tr>
<tr>
<td>4</td>
<td>100</td>
<td>55</td>
<td>0.17</td>
<td>1.8</td>
</tr>
<tr>
<td>5</td>
<td>150</td>
<td>55</td>
<td>0.17</td>
<td>1.8</td>
</tr>
<tr>
<td>6</td>
<td>200</td>
<td>55</td>
<td>0.17</td>
<td>1.8</td>
</tr>
<tr>
<td>7</td>
<td>100</td>
<td>55</td>
<td>0.17</td>
<td>2</td>
</tr>
<tr>
<td>8</td>
<td>150</td>
<td>55</td>
<td>0.17</td>
<td>2</td>
</tr>
<tr>
<td>9</td>
<td>200</td>
<td>55</td>
<td>0.17</td>
<td>2</td>
</tr>
<tr>
<td>10</td>
<td>100</td>
<td>55</td>
<td>0.17</td>
<td>Constant*</td>
</tr>
<tr>
<td>11</td>
<td>150</td>
<td>55</td>
<td>0.17</td>
<td>Constant*</td>
</tr>
<tr>
<td>12</td>
<td>200</td>
<td>55</td>
<td>0.17</td>
<td>Constant*</td>
</tr>
</tbody>
</table>

*Density of limestone in footwall is 2720 kg/m³

The results were firstly examined by using Stacey’s criterion. Critical extension strain was assumed equal to 0.00009, 0.00014 and 0.00018 (according to equation 4.8) corresponding to UCS’ equal to 100, 150 and 200 MPa respectively. The value of \( f \) was assumed to be 0.3. Since the positive direction is assigned for compressive stress in Examine³D, negative values for minor strains should be considered as results. The isosurfaces obtained from the software are shown in Table 5.3. More pictures regarding results from the software can be found in Appendix B.

Table 5.3: Isosurface of failure according to Stacey’s criterion for footwall drift- Level 896

<table>
<thead>
<tr>
<th>K-Ratio</th>
<th>( e_c )</th>
<th>Isosurface</th>
</tr>
</thead>
<tbody>
<tr>
<td>1.8</td>
<td>0.00009</td>
<td><img src="image" alt="Isosurface Image" /></td>
</tr>
<tr>
<td>1.8</td>
<td>0.00014</td>
<td><img src="image" alt="Isosurface Image" /></td>
</tr>
</tbody>
</table>
It should be noted that floor heaving was observed between drifts R13 and R9, as the confinement of the ore is greater on this side of the orebody (refer to Figure 5.2, Border of zone A and B). According to Larsson (2004) when the stiffness of the excavated rock is lower than the stiffness of the host rock a stiff loading system is simulated and the failure process is more likely to be non-violent (Figure 5.5) which is the case for drifts number R14 to R23.

Figure 5.5: Two distinct loading systems caused by difference in stiffness (After Larsson, 2004)
In order to study the sensitivity of K-ratio and critical extension strain parameters versus the depth of failure, Figure 5.7 is presented; readings were extracted from reference point shown in Figure 5.6:

![Figure 5.6: Location of reference point for reading of failure depth](image)

![Figure 5.7: Parameter sensitivity analysis effect on depth of failure – Stacey’s criterion](image)

The constant loading used in above diagram was defined according to Figure 3.3 as \(\sigma_H = 45\) MPa, \(\sigma_V = 24\) MPa and \(\sigma_h = 20\) MPa. The result implies that critical extension strain of 0.00018 in combination with constant loading and K-ratio of 1.8 have the best agreement with the actual failure.
Same approach was conducted using the models with excavated secondary stopes to examine the relevance of the assumption of failed secondary rooms. Results are presented in the same manner in Figure 5.8, as it can be seen the changes are not significant.

Hoek-Brown modified criterion for brittle rocks was also used to predict the failure in the model. Several intact rock UCS were assumed for calibration, which can be used to obtain a domain of possible uniaxial compressive strengths of the rock that lead to failure, although the exact extent of failure is not known and the calibration was limited to approximately one meter of heaving on the floor of footwall drift, therefore the precise value for uniaxial compressive strength cannot be gained. The isosurfaces restricted to safety factor of one are shown in Table 5.4. The pictures show the top view of footwall drift on level 896.

![Figure 5.8: Parameter sensitivity analysis effect on depth of failure – Secondary failed](image)

Table 5.4: Isosurface of failure according to Martin’s criterion for footwall drift- Level 896

<table>
<thead>
<tr>
<th>Loading</th>
<th>UCS</th>
<th>Isosurface</th>
</tr>
</thead>
<tbody>
<tr>
<td>$\sigma_H=45\text{MPa}$</td>
<td>$\sigma_{h}=20\text{MPa}$</td>
<td>$\sigma_v=24\text{MPa}$</td>
</tr>
</tbody>
</table>
As it can be seen, Martin’s criterion leads to bigger volume of failure with higher concentration of yield volume on the top of the drift comparing to Stacey’s, for instance by using UCS=100 MPa allied to K=1.5, results show extensive failure on the roof reaching to cut and fill mined area above, no failure on the floor of the drift was indicated. The results from the models with failed secondary stopes showed almost the same results as the models with actual excavated rooms. Since in this case Martin’s criterion failed to simulate the failure shape, further investigation was skipped by the author. The closet situation was UCS=200 MPa associated with K=1.8 and the constant loading.

Another interesting result that can be studied is stored strain energy in the rockmass. The strain energy can be used to examine the possibility of rockburst; the occurrence of rockburst depends on both the property of storing strain energy in the host and the environment of strain energy accumulation in mining. Sudden relief of accumulated energy leads to a strong shock that is more essential under uniaxial compression which is similar to the stress condition on the boundary of excavation. Wang and Park (2001) proposed that as maximum energy exceeds 1.0x10^5 J/m^3 the probability of sudden release of strain energy increases. Isosurface of strain energy higher than 1.0x10^5 J/m^3 is presented in Figure 5.9 with indicators for the observed violent failures on level 896. (For failure descriptions refer to Figure 5.2 )
5.2 Workshop on level 880

Nyström used the data of pillar breakage in level 880 to calibrate his model (2005) using Martin’s criterion. The data can be used for the purpose of this thesis in order to validate the assumed value for critical extension strain for the limestone by comparing the analysis result with depth of crack that was measured in the pillars on level 880. Samples were made in two 4 meters long boreholes. The first hole was drilled in one of the narrow pillars and the other in a wide column. Boring samples showed that the narrow pillar was cracked up basically all along the borehole by extensive fracturing near the rock surface. In the wide column, only a small number of near-surface cracks were observed, see Figure 5.10 and Figure 5.11.
By using critical strain of 0.0001, good agreements were gained with the observed failure. See Figure 5.12.
5.3 Stope roof failure on level 926

Roof failure on level 926 can be used to obtain a domain for uniaxial compressive strength of the ore. Again, calibration was done by considering two different situations for the model; one with actual excavation at the time of failure and another with assumption of secondary stopes that were surrounded by primary excavations had failed. In Figure 5.13 gray areas represent excavated stopes and red characterizes secondary stopes that are assumed to be failed.
The failure includes fall of rock boulders in addition to spalling and caving in the shape of V-notches on the back of stopes near the hanging wall. Caving along footwall in R11 was also observed while the hangingwall in the same stope is relatively intact. Other failures such as caving along footwall contact and along blind opening (used for first blast) in R13 and caving along weak hangingwall in R15 were also witnessed. It should be noted that R17 was filled along the hangingwall with paste fill after mucking, the aim was to prevent fall off from hangingwall.

Figure 5.14 gives a hint of the location and the observed failure in the area of interest while Table 5.5 contains description of the failures.

Figure 5.13: Stopes condition used for calibration on level 926
Figure 5.14: Location of failures on level 926

Table 5.5: Observed failures on level 926

<table>
<thead>
<tr>
<th>Number</th>
<th>Failure Indicator</th>
</tr>
</thead>
<tbody>
<tr>
<td>I</td>
<td>Caving of the roof with depth of several meters/ fallouts/ spalling</td>
</tr>
<tr>
<td>II</td>
<td>Major fallouts/ visible spalling</td>
</tr>
<tr>
<td>III</td>
<td>Fallouts at the cutoff</td>
</tr>
<tr>
<td>IV</td>
<td>Major fallouts/ visible spalling</td>
</tr>
<tr>
<td>V</td>
<td>Paste fill at the end of the stope/ only drift is excavated/ visible soft schistosity on the walls of drift</td>
</tr>
<tr>
<td>VI</td>
<td>Paste fill at the end of the stope/ only drift is excavated/ visible soft schistosity on the walls of drift</td>
</tr>
</tbody>
</table>

Figure 5.15 and Figure 5.16 show the model used for analysis in Examine<sup>3D</sup>. 
It should be mentioned that the drifts were eliminated from the model in favor of simplification; moreover a denser mesh was assigned to elevations 926 and 956. The longest vertex in dense mesh area was about 6 meters.

The conditions that result in failure of a rockmass surrounding an excavation can be either caused by structure or stress. In order for the structure controlled failure to occur, loss of stresses are essential in most cases as steep wedges (and of course shallow wedges as well)
which would normally have been clamped in place can be liberated and destabilized causing serious and often catastrophic failures (Diederichs and Kaiser, 1999).

Modified Hoek-Brown criterion with Martin’s values for $m$ and $s$ was used to predict the formation of cracks. The main areas of interest for calibration were stope R11 where caving was observed all along the back of the stope in the footwall shaped as a V-notch with highest vertex of 12 meters and no caving along hangingwall, Stope R13 and R15 that suffered from caving along the hanging wall with 10 meters length and 7 meters height. A survey conducted using CMS scanning in stope R11 is presented in Figure 5.17.

Figure 5.17: Caving survey in stope R11, level 926
The basic model was made with the assumption of $E=85$ GPa, Poisson’s ratio equal to 0.15 and density of $3300 \text{ kg/m}^3$, the loading was assigned as constant with the value of $\sigma_H=45\text{ MPa}$, $\sigma_H=20\text{ MPa}$ and $\sigma_V=24\text{ MPa}$. Table 5.6 shows the model schemes.

Table 5.6: List of model schemes for study of level 926

<table>
<thead>
<tr>
<th>Scheme</th>
<th>UCS (MPa)</th>
<th>$E$ (GPa)</th>
<th>$\nu$</th>
<th>K-Ratio</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>100</td>
<td>85</td>
<td>0.15</td>
<td>1.5</td>
</tr>
<tr>
<td>2</td>
<td>150</td>
<td>85</td>
<td>0.15</td>
<td>1.5</td>
</tr>
<tr>
<td>3</td>
<td>200</td>
<td>85</td>
<td>0.15</td>
<td>1.5</td>
</tr>
<tr>
<td>4</td>
<td>100</td>
<td>85</td>
<td>0.15</td>
<td>1.8</td>
</tr>
<tr>
<td>5</td>
<td>150</td>
<td>85</td>
<td>0.15</td>
<td>1.8</td>
</tr>
<tr>
<td>6</td>
<td>200</td>
<td>85</td>
<td>0.15</td>
<td>1.8</td>
</tr>
<tr>
<td>7</td>
<td>100</td>
<td>85</td>
<td>0.15</td>
<td>2</td>
</tr>
<tr>
<td>8</td>
<td>150</td>
<td>85</td>
<td>0.15</td>
<td>2</td>
</tr>
<tr>
<td>9</td>
<td>200</td>
<td>85</td>
<td>0.15</td>
<td>2</td>
</tr>
<tr>
<td>10</td>
<td>100</td>
<td>85</td>
<td>0.15</td>
<td>Constant*</td>
</tr>
<tr>
<td>11</td>
<td>150</td>
<td>85</td>
<td>0.15</td>
<td>Constant*</td>
</tr>
<tr>
<td>12</td>
<td>200</td>
<td>85</td>
<td>0.15</td>
<td>Constant*</td>
</tr>
</tbody>
</table>

Critical extension strain was anticipated to be in range of 0.00005 to 0.0001 using equation 4.8. The isosurfaces are shown in Table 5.7.

Table 5.7: Isosurface of failure according to Stacey’s criterion for stopes in level 926

<table>
<thead>
<tr>
<th>K-Ratio</th>
<th>$e_c$</th>
<th>Isosurface</th>
</tr>
</thead>
<tbody>
<tr>
<td>1.8</td>
<td>0.00005</td>
<td><img src="image" alt="" /></td>
</tr>
<tr>
<td>1.8</td>
<td>0.00008</td>
<td><img src="image" alt="" /></td>
</tr>
</tbody>
</table>
As it seen, in all the cases the results indicate total failure of secondary stopes which is anticipated to be the actual condition of those areas. Results for sensitivity of K-ratio and critical extension strain were conducted at a reference point on stope R11 showed in Figure 5.18. Figure 5.19 illustrates the comparison.
Depth of failure in all cases is less than the actual observation, the reason for that lies in progressive nature of the failure. The parameters sensitivity analysis was continued by studying the models with failed secondary stopes, see Figure 5.20:
To simulate the exact failure shown in Figure 5.17 is almost impossible due to limitations of the software, progressive nature of the failure and absence of geological futures of the surrounding rock in the software. The results for the models with excavated secondary stopes indicates closer agreement to the observed failure however because of lack of continuous survey of the caving, it cannot be judged which model shows the initiation of the failure. More accurate study would be gained by modeling the progress of the failure according to caving surveys conducted in a certain time spans.

Martin’s criterion was initially examined on the basic model and by studying the failure on the stope R11 in order to inspect the reliability of the criterion. In the base model reading of total 45 points were conducted in the model on the boundary of assumed failure according to the survey shown in Figure 5.17. Figure 5.21 illustrates the difference in envelopes drawn by Hoek-Brown criterion and Martin’s, triangular icons represent reading from safe points while crosses signifies failures. Hoek-Brown envelop was gained by using average RMR= 70 (see Table 2.1 and Table 2.3) which according to equation 4.2 and 4.3 consequences in \( m=5.13 \), \( s=0.035 \) (\( m_i \) assumed to be equal to 15).
The above diagram confirms that Martin’s criterion leads to acceptable agreement with the failure in mine. More schematic results are shown in Table 5.8. The criterion was used for several uniaxial strengths so a range for UCS can be obtained.
Table 5.8: Isosurface of failure according to Martin’s criterion for stopes in level 926

<table>
<thead>
<tr>
<th>K-Ratio</th>
<th>UCS</th>
<th>Isosurface</th>
</tr>
</thead>
<tbody>
<tr>
<td>1.8</td>
<td>100</td>
<td><img src="image1.png" alt="Isosurface" /></td>
</tr>
<tr>
<td>1.8</td>
<td>150</td>
<td><img src="image2.png" alt="Isosurface" /></td>
</tr>
<tr>
<td>1.8</td>
<td>200</td>
<td><img src="image3.png" alt="Isosurface" /></td>
</tr>
</tbody>
</table>
Assumption of UCS equal to 100 and 150 MPa points to extensive failure on top of the stopes reaching to cut-and-fill mined areas, while the caving was observed with height of 12 meters for stope R11, UCS of 200 MPa leads to more rational results, this value is reasonable judging by measurements done by Dahle (2005) which directed to expected uniaxial strength in range of 130 to 250 MPa (See Table 2.5). Similar to Stacey’s criterion, Martin’s criterion reveals total failure in secondary stopes; simulation of secondary stopes failure by excavating the stopes in the model however, demonstrates extensive failure all over the grid box as it is shown in Figure 5.22. Table 5.9 shows the effect of changes of K-ratio on depth of failure.

Table 5.9: variation of K-ratio effect on failure isosurface according to Martin’s criterion

<table>
<thead>
<tr>
<th>K-Ratio</th>
<th>UCS</th>
<th>Isosurface</th>
</tr>
</thead>
<tbody>
<tr>
<td>1.5</td>
<td>200</td>
<td></td>
</tr>
</tbody>
</table>
As it seen, the closest Isosurface to reality is the one with constant loading and the model with K-ratio equal to 1.8 with UCS of 200 MPa. In conclusion every failure mechanism requires interpretation with appropriate failure criterion, Martin’s criterion seems to show bigger volume of failure comparing to Stacey’s although acceptable results were obtained for both criteria. Both criteria were used in the rest of the study but only the results from Martin’s criterion are presented.
6 Mine sequencing

Sequencing defines the way in which mining progresses throughout the orebody. It is mainly driven by the ore grade (high grade first), but also used for the aim of stress management and minimizing ground control problems, governing a sustainable extraction rate and also for the purpose of overcoming operational issues (access, ventilation, filling, etc.).

To maintain stability in the rock mass it is essential that the void spaces are designed at stable dimensions, such that failures are arrested and/or delayed. Safe dimensions for excavation are many times smaller than the orebody itself. Consequently, a series of individual stopes must be excavated to achieve full orebody extraction which justifies the necessity of sequencing design. Therefore, extraction sequences are fundamental for achieving safe and economic production throughout a stoping life (Villaescusa, 2003). Stoping sources are usually scheduled from a number of locations and extraction methods, several sequencing strategies were studied for mining in Lappberget for the ore between levels 1250 to 1100. In case of Lappberget, to avoid the failure on the sill pillar below level 1080, the remaining ore requires to have great height, which means renunciation of the economic value of the ore. On the other hand, by leaving a short sill pillar, the failure will be inevitable thus one of the aims of sequencing design for Lappberget is to delay the failure in production cycle.

The extraction can be achieved by using multiple stopes (primary, secondary and when required tertiary) in conjunction with mass blasting and filling. The main concern is the sequencing of the stopes such that early overstressing of permanent pillars is avoided.

A technically sound strategy is to engineer the overall extraction sequences in a way that initial stopes are extracted within a chosen area of the orebody and subsequent stopes are retreated systematically towards orebody extremes, consequently avoiding blocks with high stress concentration.

As two or more stopes are excavated along a major principal stress trajectory direction, stress shadowing will occur. Sequencing, depending on the excavation plan, leads to leaving pillars (usually as secondary or tertiary stopes). In deep orebodies high stress is encountered due to the depth, it is of great importance to avoid overstressed pillars and hanging pillars, this goal can be fulfilled through use of pillarless or early pillar recovery mine sequencing strategy.
The excavation advancement direction can be chosen as center-out, top to bottom, bottom-up and from abutments towards the center. The bottom-up mining approach is very common for sequencing in high stress conditions, ideally extraction progresses from the bottom of mining block towards its top as the total extraction increases and the stress concentrates, the extraction horizon moves toward the shallower levels of the mine and towards the area of lower pre-mining stresses; as a result, excessive induced stress and deteriorating ground conditions are better managed (Potvin and Hudyma, 2000). The thesis mainly focuses on bottom-up approach in addition to a two-way excavation strategy from both bottom to top and top to bottom.

6.1 A study on stope dimensions

Stope size is very vital in terms of production rate and also safety of working environment by avoiding failures. Dimensions of the rooms have direct effect on costs associated with the mucking, haulage, crushing, hoisting, milling and treatment of waste rock, and the backfill. Larger stopes in economical aspect lead to increase of cost of backfill while at the same time it increases the production rate. As for the technical side, larger excavations cause further yielding in the surrounding rock which can be an undesirable effect, though in some cases controlled fracturing of the host rock may be considered as an advantage for easier extraction.

The initial design of an underground mining method requires that a stope dimension be chosen to form the basis of the feasibility study. By examining different sizes for the stopes and their effect on crack propagation depth in the host rock, it can be determined how large of an opening will be appropriate. This section deals with the results from analysis of various extraction dimensions in light of how they meet the objectives demanded for a successful mining operation.

No two stopes are the same, as each has numerous potential variables such as geological features and stress environment, which may impact recovery and crack initiation depth. The choice of room sizes are greatly influenced by factors such as equipment limitations and the rockmass stability.
Rockmass characteristics defined for models were obtained according to the result of calibration analysis. In situ stresses were used in terms of principal stresses and their associated orientations expressed in dip and dip direction according to a discussion with Anders Nyström and Per-Ivar Marklund (2012). Table 6.1 provides the input data for the base model generated in Examine\textsuperscript{3D}.

<table>
<thead>
<tr>
<th>E (GPa)</th>
<th>v</th>
<th>UCS (MPa)</th>
<th>σ\textsubscript{1} (MPa)</th>
<th>Dir/Dip</th>
<th>σ\textsubscript{2} (MPa)</th>
<th>Dir/Dip</th>
<th>σ\textsubscript{3} (MPa)</th>
<th>Dir/Dip</th>
</tr>
</thead>
<tbody>
<tr>
<td>85</td>
<td>0.15</td>
<td>150</td>
<td>55</td>
<td>345/0</td>
<td>30</td>
<td>0/90</td>
<td>24</td>
<td>75/0</td>
</tr>
</tbody>
</table>

Common stoping sequence in many Canadian mines employing blasthole methods uses a pyramidal or chevron mining front. Stopes are sequenced to maintain a triangular shape to the mined-out area by mining vertically with a lead stope, then outward along the rill of the triangle towards its base. The leading primary stope, subjected to elevated stresses as a result of the high level of confinement, creates a ‘bow wave’ effect that tends to distress adjacent primary stopes and shed stresses to the abutments (Board et al. 2001). Primary stopes are usually mined and filled for two vertical lifts before mining of the secondary stopes between them is started. In order to simulate the advancement of the primary stopes, models were constructed in three different stages, initial stope, two equally dimensioned stopes on top of each other and a block with height equal to three times the height of the initial stope (see Figure 6.4). Table 6.2 shows the dimensions used for modeling. Modeled stopes were assigned a height of 25 meters in every stage as it was chosen by Boliden as a common practice in Lappberget. Since the modeling includes three lifts for advancement of the stope, the height changes from 25 meters to 50 and 75 meters respectively. For this parameter study, stope dimensions changing in range of 10, 15 and 20 meters in width and the strike length of 20 and 40 meters were assessed.

<table>
<thead>
<tr>
<th>Width(m)</th>
<th>Length(m)</th>
<th>Height(m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>10</td>
<td>20</td>
<td>25</td>
</tr>
<tr>
<td>10</td>
<td>20</td>
<td>50</td>
</tr>
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<td>10</td>
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<td>20</td>
<td>50</td>
</tr>
<tr>
<td>15</td>
<td>20</td>
<td>75</td>
</tr>
<tr>
<td>15</td>
<td>40</td>
<td>25</td>
</tr>
</tbody>
</table>
6.1.1 Estimation of crack volume

Dunne and Pakalnis (1996) and Clark and Pakalnis (1997) proposed a way to express dilution on the walls of the stopes, same approach with some modification in failure criterion is adopted to achieve an index to direct crack depth on the walls of the stopes. The method used in this study is based on calculating average volume of cracked wall per square meters of roof ($m^3/m^2$), rather than length of crack out of the wall. The initial idea was to achieve an index by dividing the volume of cracked Isosurface by the volume of the stope. However since the height of the stopes is constant it was eliminated from the equation. This method was derived from ELRD (equivalent linear relaxation depth) value which was proposed as a tool to quantify the relaxation zone around excavations. Since the height is equal in all cases, the area of surface is considered as the surface of the roof. An advantage of using crack depth index (CDI) is that the effect of dimensions can be associated with the volume of formed cracked around the excavation rather than a length in only one direction. CDI was calculated as follows:

(Eq.6.1) \[ CDI = \frac{Volume \ of \ measured \ crack \ volume \ from \ the \ stope \ wall \ (m^3)}{Roof \ area \ of \ the \ stope \ (m^2)} \]

Crack depth was determined from Isosurface of strength factor of 1, located on a vertical plane positioned at the stope side wall and a horizontal plane on mid-height. For the purpose of the study, crack depth was defined by strength factor contours using Hoek-Brown criterion with Martins’ modified parameters for $m$ and $s$, measured from the center of the stope wall.

Using Examine$^{3D}$, strength factors were plotted onto grids placed on the wall and mid-height of the stope, as illustrated in Figure 6.1. Grid planes were orientated normal to the excavation-wall dip, and extend a distance of 100 meters away from the stope boundary. The extent of the potential cracked zone associated with a given stope geometry was determined from contours of strength factor less than 1.
The volume of potential cracks simulated by a 3-D elastic numerical model for a given 3-D stope geometry was estimated using an approach described by Pakalins et al. (1998) in which the volume was represented as the volume of half a prolate ellipsoid, shown in Figure 6.2. The volume of cracked region was calculated as:

\[
V = \left(\frac{2\pi}{3}\right)r_1r_2r_3 \quad (\text{m}^3)
\]

In Equation 6.2, \( r_1 \), \( r_2 \) and \( r_3 \) correspond to the perpendicular, horizontal and vertical radius distances from center (wall side and mid-height) of the stope wall contact.
A potential for failure exists within the envelope of formed cracks, defined by Hoek-Brown criterion with modified parameters for brittle rock. The extents of such envelope were determined from Isosurface of strength factor less than 1, located on vertical and horizontal planes located at the stope wall and mid-height, respectively, as illustrated in Figure 6.2.

To quantify modeled crack propagation from the 3-D simulations, the crack depth index (CDI) is introduced in this study to denote potential crack propagation; the index was used further in the study as a tool for comparing different stope sizes.

### 6.1.2 CDI relationship with stope dimension

There are different factors that influence the stability of an excavation. Unlike parameters such as depth of mining, geological structures and in-situ stresses which are out of control of mine operation, stope dimensions are variable factors that influence the stability and crack propagation depth which can be optimized in order to make a balance between productivity, safety and general financial and mechanical aspects of a mine. Selection of stope dimensions should be based on a compromise between “acceptable” overbreak and expenses due to establishing infrastructures and backfilling.
The effects of changes in stope width and length were assessed using Examine3D. When investigating the sensitivity of stope dimensions on crack formation, data from the volume of isosurfaces surrounding the stopes were collected in order to obtain an index that can be compared for different areas of stope roof (Equation 6.1).

Trends associated with CDI for the strength factor < 1 contour as a function of strike length and width of stopes are presented in Figure 6.3. The figure represents a design tool that can be used to determine the dimension of primary stopes which later on would be used as the base case for modeling different sequences of excavation. The effect of variation in stope sizes in different sequencing approaches on the sill pillar above the excavation area was also studied (More on this subject in 6.6).

![Crack Depth Index on wall of stopes](image)

**Figure 6.3: CDI corresponding with different stope dimensions**

Figure 6.3 shows six different schemes for stope dimensions in three advancing stages, the first stage - color coded with blue- was modeled as an isolated stope with height of 25 meters in the surrounding rock, the red bar represents the second stage that was simulated as a block with twice the height of the initial stope and finally the green bar displays the crack depth...
index for the third stage with modeled block of 75 meters of height, see Figure 6.4. Only three steps of mining advancements were modeled since it was assumed that the primary stopes will not advance more than three vertical lifts before excavation of the first secondary stope.

According to the results, the crack depth index is significantly lower when the ratio of length over width reaches 1.3 (for the case of 15 meters width and 20 meters length); in addition to that the diagram reveals that when the strike length increases to 40 meters, CDI changes slightly for the second and third stage and almost independently from the width of the slope. Eventually it can be concluded that as the face of the stopes become closer to a square shape, the stopes become more stable and the crack depth will be limited. In 3 dimensions, CDI value for a rectangular prism indicates less crack depth comparing to a cubic block however further research should be conducted to validate the last statement. Moreover interpretation of Figure 6.3 indicates that potential crack depth changes are less significant between different stope types in the first stage, although the alteration becomes more noticeable for the second and third stage.

Table 6.3 includes the data that were used for illustration of results showed in Figure 6.3.

Table 6.3: Measured crack radius from the wall of stopes

<table>
<thead>
<tr>
<th>X</th>
<th>Height</th>
<th>Y</th>
<th>R1</th>
<th>R2</th>
<th>R3</th>
<th>Volume (m³)</th>
<th>Area(m²)</th>
<th>CDI(m)</th>
<th>Stage</th>
</tr>
</thead>
<tbody>
<tr>
<td>10</td>
<td>25</td>
<td>20</td>
<td>7.62</td>
<td>7.2</td>
<td>15.5</td>
<td>1780.98</td>
<td>200</td>
<td>8.90</td>
<td>1</td>
</tr>
<tr>
<td>10</td>
<td>25</td>
<td>20</td>
<td>9.9</td>
<td>8.9</td>
<td>26.58</td>
<td>4904.77</td>
<td>200</td>
<td>24.52</td>
<td>2</td>
</tr>
<tr>
<td>10</td>
<td>25</td>
<td>20</td>
<td>10.2</td>
<td>9.06</td>
<td>39.63</td>
<td>7669.93</td>
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<td>15</td>
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<td>8.27</td>
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<td>7.2</td>
<td>28.91</td>
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<td>25</td>
<td>20</td>
<td>10.83</td>
<td>7.6</td>
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<td>25</td>
<td>20</td>
<td>8.5</td>
<td>12.82</td>
<td>16.86</td>
<td>3847.72</td>
<td>400</td>
<td>9.62</td>
<td>1</td>
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<tr>
<td>20</td>
<td>25</td>
<td>20</td>
<td>11.03</td>
<td>14.36</td>
<td>29.9</td>
<td>9918.36</td>
<td>400</td>
<td>24.80</td>
<td>2</td>
</tr>
<tr>
<td>20</td>
<td>25</td>
<td>20</td>
<td>11.7</td>
<td>14.87</td>
<td>41.97</td>
<td>15292.36</td>
<td>400</td>
<td>38.23</td>
<td>3</td>
</tr>
</tbody>
</table>
In Table 6.3, \( r_1 \), \( r_2 \) and \( r_3 \) correspond to the perpendicular, horizontal and vertical radius distances from center (wall side and mid-height) of stope wall; volume was obtained from equation 6.2.

The choice of stope dimensions should be in a way that the depth of cracks be reasonable for both mechanical aspect of the mining as well as financial side. Farther crack propagation into the secondary stopes has the advantage of making the excavation easier and reduces the chances of violent failures. On the other hand, more crack propagation increases the risk for fallouts which changes the planned geometry and could result in oversized blocks that requires secondary blasting. According to Figure 6.3 stope dimension of 15x20 meters for the primary stope is not efficient since it results in less volume of cracks comparing to other schemes, on the other hand choosing a big room for primary stopes increases the expenses for the backfill which has to have higher strength comparing to smaller voids, so a rational choice would be primary stopes with narrow width and high crack propagation index. The alternatives for primary stope dimension that fulfill these requirements are 15x40, 10x40 and 10x20 (width x strike length) meters in the order of the depth of crack propagation in the plane normal to the primary stope wall (\( r_1 \) in Table 6.3). The size of secondary stopes can also be decided accordingly to the dimensions of the primaries. The sequencing should be planned in a way that governs early extraction of secondary stopes right after mining of primary stopes in order to avoid rises in potential of violent failures, but still it should be late enough for secondary stopes to fail.
6.2 Primary and secondary stope excavation

The main advantages of primary and secondary sequencing method are an initial high degree of flexibility and productivity and low cost during primary stoping. The overall cost is minimized by the use of unconsolidated fill within the secondary stopes. A disadvantage is that stress re-distributions may cause rock mass damage within secondary pillars late in the extraction sequence. The effects of stress can be minimized by avoiding undercutting of individual stopes and by mass blasting those highly stressed regions within a stoping block. Multiple lift primary and secondary stopes have been used very successfully to achieve complete extraction with minimal dilution within the steeply dipping orebodies (Bywater et al, 1985).

Three sequencing patterns with different stope dimensions were studied using primary-secondary strategy to identify the optimum shape for extraction of the orebody; for this purpose an orebody of 165 meters width, 40 meters strike length and height of 150 meters was modeled and stopes were excavated according to each pattern separately, the level for the new excavation sector starts from 1250 and moves upward to 1100 leaving 20 meters sill pillar before reaching level 1080; the bottom of previous mining area. The height for every stage of excavation in all the patterns were restricted to 25 meters as it was decided by the company; strike length of stopes were also fixed by average length of the orebody (40 meters).

The sequencing approach was selected in a way that initial extraction begins with two stopes excavated in the center with a distance of one secondary stope to create a stress shadow for the remaining stopes. Using this alternative, the initial fill mass is not exposed simultaneously on both sides. Primary stopes were designed smaller than secondary stopes (pillars) to minimize the use of cemented fill and the secondary stopes are designed large enough to enable safe recovery between primary stopes.

6.2.1 Pattern 1, P10S15

The first studied pattern was a primary-secondary approach with primary stope width of 10 meters, the width of secondary stopes was chosen as 15 meters. Figure 6.5 illustrates the sequencing of the stages for this pattern. It should be noted that the numbers represent the stages of the sequences and it was assumed that secondary stopes at every stage are excavated after the pastefill for the previous stage had cured.
The excavation initially starts in S6 and S8, at the third stage first secondary stope will be mined in S7. Stage 5 provides 10 working stopes which administers high production rate for the mine.

6.2.2 Pattern 2, P10S20

Second pattern was designed with stope width of 10 and 20 meters for primary and secondary stopes respectively, stage numbers are shown in Figure 6.6.
Figure 6.6: Sequencing of Pattern 2- Primary 10m and secondary 20m

Since the width of the modeled orebody was fixed by 165 meters, changes in dimension of the stopes for different patterns result in variation of the size of the rooms on the far sides of orebody compared to the dominant size assumed for the rest of the stopes in that pattern, therefore for some patterns the size of first and last column of stopes are different. The variation of the stope size is not a part of the actual sequencing pattern and is solely applied for the purpose of easing comparison of the patterns.

6.2.3 Pattern 3, P15S20

Eventually the last pattern for primary-secondary sequencing approach was intended for primary stope width of 15 and secondary width of 20 meters. Larger primary stopes generally lead to higher production rate early in the mining cycle but at the same time most likely they jeopardize the safety of the rooms and also increase the cost of fill material for primary stopes. More detail about the pattern is presented in Figure 6.7.
6.3 Tertiary stope sequencing

A primary-secondary-tertiary triangular retreat shape can be used as a substitute for primary-secondary approach. The system has been successfully implemented at several mines in Canada. The main advantage of this method is that it allows a number of stopes to be mined simultaneously, hence increasing the productivity within a mining block. Because of the detrimental effects of stress re-distributions on the pendant pillars formed in the sequencing, secondary pillar stopes must be recovered as early as possible in the extraction sequence. In general, no more than two sublevels are mined ahead of a pillar before recovering it and both sides of a pillar cannot be mined simultaneously (Potvin and Hudyma, 2000). The main problem with this method is that the sequencing should be followed strictly during the entire extraction process or else bursting in pillars rather than gradual failing can be expected. In addition to that, the method causes rise in expenses of fill material since both primary and secondary stopes should be packed by pastefill while waste fill can only be used for tertiary stopes.
6.3.1 Pattern 4, 1-5-9 Sequencing

A variation to tertiary method is 1-5-9 stoping sequence. The general excavating sequence is from the center of orebody towards abutments, Stopes 1-5-9 (in this case S3, S7 and S11) are extracted as two lift primaries and filled with consolidated fill, stope S3 and S11 were lagged one stage so the overall excavation maintains a triangular shape which results in better redistribution of stresses. The excavation is followed by retreating stopes 3-7-11 (in this case S1, S5, S9, S13) as secondary, also filled with pastefill. First tertiary stope is excavated after stopes S1, S5 and S9 reach two lifts ahead. Excavation of teriaries creates a number of pendant pillars with many degrees of freedom that rely on the fill support. See Figure 6.8.

Figure 6.8: Sequencing of Pattern 4 (1-5-9)- Primary 15m and secondary 15m and tertiary 10m

A disadvantage of a 1-5-9 extraction sequence using short lift stopes (or 1-4-7 in that regard) is their inefficient stope mucking characteristics. The mucking has to be carried out in areas that had previously been subjected to stress distribution. Each stope access becomes a stope drawpoint and a significant amount of reinforcement using cablebolting is required in all the stope accesses and exposed backs. Reinforcement can be largely inefficient within the bottom of pendant secondary pillars where remote mucking is required for 100% of the tonnage. Furthermore, additional footwall development access in waste rock may be required on each
sublevel, as more than one access may be required for effective mucking of each individual stope (Villaescusa, 2003). Moreover, if the tertiaries were allowed to lag behind the secondary stopes, violent failure becomes a possibility.

6.3.2 Pattern 5, 1-4-7 Sequencing

Another deviation to tertiary sequencing is 1-4-7 which is very similar to 1-5-9 in basis. The extraction begins in stopes 1-4-7 (S4, S7 and S10) as primary stopes where S7 acts as the leading stope to compromise keeping the triangular shape of pattern. Extraction is followed by excavation of first secondary stope in S8; at the same time two tertiary stopes are being mined on both far sides of the orebody in S1 and S13. As it can be seen in Figure 6.9 symmetry is absent in this method, this results in different sizes for the designed rooms, therefore referred pattern contains one tertiary stope with the width of 10 meter which does not follow the rule of 15 meters width for the rest of tertiary stopes. The width of primary stopes was designed as 10 meters and the width for secondary stopes was decided as 15 meters.

![Figure 6.9: Sequencing of Pattern 5 (1-4-7)- Primary 10m and secondary 15m and tertiary 15m](image)

This strategy suffers from same disadvantages as 1-5-9. However, similar to pattern 4 it offers high production rate.
6.4 Pillarless, center-out sequence

The ideal stress management stoping sequence is a systematic retreat from the center out, without pillars. Although conceptually sound, the capacity of this mining system to produce ore is severely constrained by the stope cycle time. Stopes must be mined, filled and cured before the adjacent stope can be mind. With active mining on a large number of sublevels, there are substantial development, scheduling and logistic challenges (Potvin and Hudyma, 2000).

It can be debated that extraction of small stopes in the host rock should be considered as an advantage, since it minimizes the magnitude of local seismic events.

As it was mentioned before, time needed for the fill to cure causes delays in production cycle, one alternative to eliminate or fade out the interruption of production is to use rapidly curing cemented backfill with minimal drainage delays in all the stopes, which introduces new expenses to the operation. Furthermore usage of tight fill is necessary in all the stopes which is expensive and sometimes not practical. Also implantation the method requires extensive rock reinforcement since large crowns are exposed. The conceptual sequencing for the method is presented in Figure 6.10.

![Figure 6.10: Sequencing of Pattern 6 (Pillar less) – Stope width 15m](image)
6.5 Top-down and bottom-up sequencing

A study on two-way sequencing method was conducted as an alternative pattern for Lappberget; the pattern was configured as both top-down and bottom-up extraction. Excavation starts as a single column on level 1250 progressing upward reaching level 1175 while at the same time sequencing advances from level 1100 towards level 1150, at this point remained ore in between is excavated. The pattern may lead to some problems regarding scheduling and development. The main reason for studying this pattern is to obtain an idea of how a two way pattern will work. The benefit of the pattern is failure on the sill pillar will occur gradually while stresses are pushed towards the abutments. The width of stopes was decided as 10 and 15 meters for primary and secondary stopes respectively as it is shown in Figure 6.11.

The scheduling of development and extraction is quite challenging in this approach, it introduces additional expenses in early stages of mining since drifts should be developed in both top and the bottom of the block, plus excavation from the top starts in the failed area which requires extensive reinforcement. Excavation of the pillar in level 1175 will be
problematic and most probably results in extra dilution from the pastefill on the roof. Besides, all the stopes above level 1150 should be pastefilled and waste material cannot be used for filling secondary stopes in referred altitude.

6.6 Comparison of patterns

In order to compare the studied patterns and to reach a conclusion to find out the most optimized sequence, failure around the excavation in every stage of every pattern were measured to obtain the stage in which failure is anticipated to occur for each pattern. The base model was constructed using the assumption of Young’s modulus equal to 85 GPa, Poisson’s ratio of 0.15, UCS of 200 MPa and critical extension strain of 0.0001. Constant loading with parameters shown in Table 6.4 was introduced to the software.

<table>
<thead>
<tr>
<th>E (GPa)</th>
<th>v</th>
<th>UCS (MPa)</th>
<th>$\sigma_1$ (MPa)</th>
<th>Dir/Dip</th>
<th>$\sigma_2$ (MPa)</th>
<th>Dir/Dip</th>
<th>$\sigma_3$ (MPa)</th>
<th>Dir/Dip</th>
</tr>
</thead>
<tbody>
<tr>
<td>85</td>
<td>0.15</td>
<td>200</td>
<td>55</td>
<td>345/0</td>
<td>30</td>
<td>0/90</td>
<td>24</td>
<td>75/0</td>
</tr>
</tbody>
</table>

The indicator of failure was assumed to be propagation of cracks according to Hoek-Brown criterion with Martin’s modified parameters. Another indication can be extension of cracks above the roof of leading stope according to Stacey’s criterion. Mentioned indicators were acquired according to calibration of the models using observed failure on level 926. The contours of failure in the critical stage of each pattern are presented in Appendix C.

It is anticipated that the same failure occurred on level 926 happens in the new excavation block as the extraction altitude comes close to the planned sill pillar for the first six patterns, as for the seventh pattern failure can be expected at time of stope excavation on level 1175. The failure is inevitable and the aim of the study is to reach the failure as late as possible in the production cycle. Pattern 1 encounters the failure as the leading primary stope opens in level 1125, same statement is valid for patterns 2, 4 and 5 although tertiary patterns (4 and 5) show more limited failures comparing to patterns 1 and 2. Third pattern specifies the failure in an earlier stage as the leading primary stope extents to level 1150. The earliest indication of failure can be observed in pillarless method, pattern 6, in the third stage of excavation at level 1200. This proves that forming a wide crown on top of stopes increases the risk of failure. The last pattern is different from the others, for pattern 7 the area of interest is the remaining ore
between levels 1150 and 1175. The bold outline in Figure 6.5 to Figure 6.11 illustrates the shape of excavation at the time of failure occurrence.

According to obtained results it was revealed that the extent of failure is more sensitive to the reached level of leading stope rather than the width of the base of a pattern. In general, it can be concluded that occurrence of the failure is unavoidable as the mining reaches level 1125 for most patterns, as for pattern 7, since the first stope opens in level 1125 the failure in sill pillar occurs immediately after excavation.

The 3rd and 6th pattern show the least volume of failure and at the same time significantly less excavated volume comparing to other patterns. Pattern 4 and 5 indicated approximately same crack propagation which is less than Pattern 1 and 2; meaning tertiary patterns are better choices in terms of limiting the failure on top of the advancing primary stopes, however using tertiary patterns increases the risk of violent failures.

The efficiency of patterns can be measured by other aspects as well, for example by comparing the volume of excavated materials at the stage of failure. A pattern that results in higher production before reaching the failure is preferable since the expenses for stabilizing the failure will be postponed. In that sense applying a pattern with wider base will result in more production before reaching the expected failure although the problem with this approach would be dealing with more failed stopes at the same time which can lead to a significant delay in production.

Table 6.5 shows the volume of excavation corresponding to the stage of failure for different patterns. Figure 6.12 illustrates the tempo of excavation volume for different patterns and Figure 6.13 represents the volume of excavation at the stage in sequencing that failure on the roof of advancing primary stope is anticipated. In Figure 6.14 volume of total fill required for each pattern is shown. The fill volume calculated by assuming usage of fill in primary stopes for chevron patterns and required fill for secondary and primary stopes for 1-4-7 and 1-5-9 methods. For pillarless center-out sequencing it was assumed that fill will be used for all of the stopes and for pattern 7 the conjecture was usage of fill in primary stopes for bottom-up excavations and also filling of all the top-bottom rooms.
Table 6.5: Excavation volume at the stage of failure

<table>
<thead>
<tr>
<th>Pattern indic.</th>
<th>Width</th>
<th>Number of total stages</th>
<th>Stage of failure</th>
<th>Excavation volume (m³)</th>
</tr>
</thead>
<tbody>
<tr>
<td>P1</td>
<td>10 15 -</td>
<td>13</td>
<td>5</td>
<td>255000</td>
</tr>
<tr>
<td>P2</td>
<td>10 20 -</td>
<td>12</td>
<td>5</td>
<td>285000</td>
</tr>
<tr>
<td>P3</td>
<td>15 20 -</td>
<td>11</td>
<td>4</td>
<td>220000</td>
</tr>
<tr>
<td>P4</td>
<td>15 15 10</td>
<td>12</td>
<td>5</td>
<td>315000</td>
</tr>
<tr>
<td>P5</td>
<td>10 15 15</td>
<td>12</td>
<td>5</td>
<td>250000</td>
</tr>
<tr>
<td>P6</td>
<td>15 - -</td>
<td>15</td>
<td>3</td>
<td>120000</td>
</tr>
<tr>
<td>P7</td>
<td>10 15 -</td>
<td>12</td>
<td>4</td>
<td>140000</td>
</tr>
</tbody>
</table>

Figure 6.12: Excavation tempo for different patterns
Tertiary patterns 1-4-7 and 1-5-9 (P5 and P4) have the highest excavation volume followed by pattern 2. By comparing patterns 4 and 5 to pattern 2, it can be concluded that pattern 2 is more desirable since the expenses for the fill material is less for a primary secondary pattern comparing to a tertiary patterns, plus 4th and 5th patterns increase the risk of rockburst failure to occur.
Table 6.6 provides a comparison summary considering technical aspects of the methods.

**Table 6.6: Technical advantages and disadvantages of proposed patterns.**

<table>
<thead>
<tr>
<th>Methods</th>
<th>Pros</th>
<th>Cons</th>
</tr>
</thead>
</table>
| **Primary-secondary** | - High degree of flexibility and productivity.  
- Low cost during mining of primary stopes.  
- Usage of unconsolidated fill within the secondary stopes. | - Concentration of stresses may occur leading to rockburst. |
| **Tertiary**    | - Higher number of stopes can be mined simultaneously.  
- Higher productivity in a shorter span of time. | - Sequencing should be followed strictly.  
- Pastefill is required in both primary and secondary stopes hence more cost.  
- Higher risk of rockburst if the secondary stopes lag more than two lifts from primary excavation and also if tertiary lag more than two lifts from secondary stopes. |
| **Pillarless**  | - Smaller risk for rockburst.  
- Symmetric extraction pattern leads to better distribution of stresses. | - Longer cycle time for extraction.  
- Substantial development and scheduling is required.  
- Exposes unsupported pillar above the excavated area which requires extensive rock reinforcement. |
| **Two way mining** | - Failure occurs gradually.  
- Concentration of stresses in the sill pillar will be avoided. | - The scheduling of development is costly.  
- Dilution from the pastefill of above stopes is a risk and has to be evaluated by strength demand on pastefill.  
- Higher expenses for fill material. |
6.7 Parameter sensitivity analysis

In order to take into account the effect of uncertainties in the parameters used for modeling, it is necessary to conduct some parameter sensitivity analysis. In this section the effect of in-situ stress variations are presented. Moreover the consequences of changes in UCS on failure depth at top of the leading stope were studied for Patterns 1 and 2. The analysis was conducted at stage 5, where the occurrence of failure was anticipated. Results are presented in Figure 6.15 and Figure 6.16.

Figure 6.15: Sensitivity analysis for pattern 1

Figure 6.16: Sensitivity analysis for pattern 2
It should be mentioned that since the failure above the leading primary stope merged the failure below the mined area in level 1080, measuring the failure depth for UCS of 150 MPa for K-ratio of 1.8 and constant loading was not possible.

According to diagrams above, assuming uniaxial strength of 200 MPa for the ore will result in no failure for the second pattern in case of constant loading and k-ratio=1.5. As for UCS of 150 MPa the failure covers the whole remaining rock up to the mined area in level 1080.
7 Conclusion and recommendation

To understand failure mechanisms in a mine is of great importance since it enables the designer to predict potential failures and to tackle them in a practical way i.e. arresting the failure, avoiding it or simply to get prepared for its occurrence.

The failure that occurred on level 926 in Lappberget will most likely repeat itself on the next block of mining. The aim of the research was to find a safe stope dimension in a sequencing pattern that delays the failure in terms of production cycle as the necessity of high extraction rates early in a mine’s life is known to mine-planning engineers. The evaluation of final results from calibration of the model, stope dimension analysis and sequencing studies are presented in the following section.

7.1 Conclusion

The thesis was carried out to aid the understanding of how the changes in stope dimensions and sequencing effects the distribution of stresses in Lappberget.

Choice of failure criteria had to be made according to the mechanical properties of the rockmass as well as expected failure mechanism. The most accurate technique to examine reliability of a failure criterion is to use the criterion to simulate an observed failure. The calibration of the models for failures on level 880, 896 and 926 failures approves the expediency of Stacey’s and Martin’s criteria. Interpretation of result from calibration models indicate Martin’s criterion shows larger volume of failure comparing to Stacey’s.

In order to comprehend the effect of stope dimensions on volume of crack formation in the host rock using Martin’s criterion, an index was proposed in the thesis for measuring the volume of failure. The modeling was done for advancing of isolated stopes to three lifts. Comparison of several stope dimensions using the index showed that the crack index is lower when the ratio of length over width reaches 1.3. Crack propagation depth is more limited for stopes with square shaped face.

The effect of different stope sizes on the failure of the host rock was studied in association with various sequencing methods. Several aspects of using different sequencing methods were considered to affect the choice of optimal approach. Since the occurrence of the failure on the sill pillar above the extraction block is inescapable, implementing a sequencing strategy that
leads to failure in a later stage is more desirable, at the same time economical side of the choice of sequencing pattern plays a major role in decision making. In author’s opinion a primary-secondary pattern with primary stope width of 10 meters and width of 20 meters for the secondary stopes suits Lappberget orebody the best since it leads to a reasonable production before encountering the failure and also the expenses for the fill material are more limited comparing to other studied methods.

7.2 Future work

The main recommendations for future work arising from this thesis are as follows:

1. **Simulation of failure progress.** As it was mentioned before, brittle failure is mainly a process of progressive slabbing resulting in a revised stable geometry. To capture the essence of the failure mechanism and to validate the reliability of the models the calibration can be assessed using data obtained from continuous caving survey conducted in specific time spans. The numerical model should simulate the process of failure by removing failed elements.

2. **Choice of failure criterion.** Several attempts to predict failure in brittle rocks resulted in variety of proposed failure criteria. Hajiabdolmajid et al. (2002), Edelbro (2008) and many other researchers suggested that a cohesion weakening and frictional strengthening (CWFS) failure criterion shows more accuracy for prediction of the brittle rock failure depth. The failure criterion cannot be used in Examine\textsuperscript{3D} so to use the advantages of CWFS a substitute code should be considered as the modeling tool.

3. **Instrumentation programs.** To evaluate the numerical models, more and better instrumentation is required. Rockmass characteristics are essential for making a dependable model. Data obtained from stress measurements and deformations can be used for the input parameters in the model and also for calibration.
7.3 References


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APPENDIX A

Failure on levels 896 and 926

Figure A.1: Rockburst on pillarnose – LVL 896

Figure A.2: Ejected bolt from the wall – LVL 896
Figure A.3: Slabbing on the roof of access drift – LVL 896

Figure A.4: Heaving of floor in access drift – LVL 896
Figure A.5: Caving on the roof of stope– LVL 926

Figure A.6: Spalling on the roof of stopes– LVL 926
APPENDIX B

Plots of model calibration

Figure B.1: failure surface according to Stacey’s criterion—LVL 896—ec 0.00009, constant loading

Figure B.2: failure surface according to Stacey’s criterion—LVL 896—ec 0.00014, constant loading
Figure B.3: failure surface according to Stacey’s criterion—LVL 896 – ec 0.00018, constant loading

Figure B.4: failure surface according to Martin’s criterion—LVL 896 – UCS 100 MPa, K-ratio=1.8
Figure B.5: failure surface according to Martin’s criterion—LVL 896 – UCS 150 MPa, K-ratio=1.8

Figure B.6: failure surface according to Martin’s criterion—LVL 896 – UCS 200 MPa, K-ratio=1.8
Figure B.7: failure surface according to Stacey’s criterion– LVL 926 – ec 0.00005, constant loading

Figure B.8: failure surface according to Stacey’s criterion– LVL 926 – ec 0.00008, constant loading
Figure B.9: failure surface according to Stacey’s criterion– LVL 926 – ec 0.0001, constant loading

Figure B.10: failure surface according to Martin’s criterion– LVL 926 – UCS 150 MPa, constant loading
Figure B.11: failure surface according to Martin’s criterion– LVL 926 – UCS 200 MPa, constant loading
APPENDIX C

Sequencing patterns plot

Figure C.1: failure according to Martin’s criterion – Pattern 1 – UCS 150 MPa, constant loading

Figure C.2: failure according to Martin’s criterion – Pattern 1 – UCS 200 MPa, constant loading
Figure C.3: failure according to Martin’s criterion – Pattern 2 – UCS 150 MPa, constant loading

Figure C.4: failure according to Martin’s criterion – Pattern 2 – UCS 200 MPa, constant loading
Figure C.5: failure according to Martin’s criterion – Pattern 3 – UCS 150 MPa, constant loading

Figure C.6: failure according to Martin’s criterion – Pattern 3 – UCS 200 MPa, constant loading
Figure C.7: failure according to Martin’s criterion– Pattern 4 – UCS 150 MPa, constant loading

Figure C.8: failure according to Martin’s criterion– Pattern 4 – UCS 200 MPa, constant loading
Figure C.9: failure according to Martin’s criterion– Pattern 5 – UCS 150 MPa, constant loading

Figure C.10: failure according to Martin’s criterion– Pattern 5 – UCS 200 MPa, constant loading
Figure C.11: failure according to Martin’s criterion– Pattern 6 – UCS 150 MPa, constant loading

Figure C.12: failure according to Martin’s criterion– Pattern 6 – UCS 200 MPa, constant loading
Figure C.13: failure according to Martin’s criterion—Pattern 7 – UCS 150 MPa, constant loading

Figure C.14: failure according to Martin’s criterion—Pattern 7 – UCS 200 MPa, constant loading