Chair of Mining Engineering and Mineral Economics

Master's Thesis

Sublevel Stoping - The Field of Application in Europe and Canada & A State-of-the-Art Research

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Master-Thesis

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&

A State-of-the-Art Research

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Abstract

The utilization of sublevel stoping as a mining method has experienced a big upswing in recent decades. The reasons for this are manifold ranging from economic to rock mechanical aspects. The questions this thesis addresses are: “What are distinctive characteristics concerning the current field of application” and “Which differences / trends regarding the implementation of this method can be identified?” These questions are specifically related to European and Canadian mining operations.

In order to answer these questions, 8 to 12 sublevel stoping operations were researched in various topics. Furthermore, several related studies were used to obtain additional reference data. Using this established dataset, geometrical and rock mechanical parameters related to the deposit as well as design parameters related to the implementation, could be analyzed and compared with each other. Based on this process of analyzation, various differences could be highlighted and possible trends concerning the utilization of this method, identified.

A further aim of this thesis is to provide a detailed State-of-the-Art overview, concerning all major elements of a sublevel stoping operation. The relevant questions in that context are: “Which design methods are in accordance with today's State-of-the-Art and can be applied to the individual elements?” and “What are the key aspects / parameters influencing the design process of these subparts?” The answers regarding both questions are partly based on literature research and to a certain extent on real-world data / cases.

Zusammenfassung

In den letzten Jahrzehnten hat der Teilsohlenkammerbau immer mehr an Bedeutung für den Abbau von unterirdischen Lagerstätten gewonnen. Die Gründe dafür sind vielfältig und umfassen wirtschaftliche sowie gebirgsmechanische Aspekte. Die Fragen, mit denen sich diese Arbeit beschäftigt, lauten: „Welche Besonderheiten gibt es in Bezug auf den aktuellen Anwendungsbereich?“ und „Welche Unterschiede/Trends lassen sich bei der Umsetzung dieser Methode feststellen?“ Beide Fragen beziehen sich konkret auf europäische und kanadische Bergbaubetriebe.

Um eine aussagekräftige Antwort zu erhalten, wurden je nach Themengebiet und Datenlage, 8 bis 12 Bergbaubetriebe, welche Abbaumethodisch in diese Kategorie fallen, untersucht. Weiters wurden bereits bestehende Studien herangezogen um zusätzliche Referenzdaten zu erhalten. Anhand dieser erhobenen Daten konnten geometrische und gebirgsmechanische Parameter der Lagerstätten, sowie bestimmte Auslegungsparameter von einzelnen Elementen untersucht und miteinander verglichen werden. Im Zuge dieser Analyse ließen sich diverse Unterschiede aufgezeigt und Tendenzen bei der Anwendung dieser Abbaumethode feststellen.

Ein weiteres Ziel dieser Arbeit ist es, einen detaillierten Überblick über den Stand der Technik zu vermitteln, in dem alle wichtigen Elemente eines Teilsohlenkammerbaus beschrieben und dargestellt werden. Die relevanten Fragen in diesem Zusammenhang lauten: "Welche Entwurfsmethoden entsprechen dem heutigen Stand der Technik und können auf die einzelnen Elemente angewendet werden?" und "Was sind die wesentlichen Aspekte/Parameter, die den Entwurfsprozess dieser Teilelemente beeinflussen?“ Um diese Fragen zu beantworten wurde eine umfassende Literaturrecherche durchgeführt sowie Daten/Fällen aus der Praxis analysiert und gegenübergestellt.

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1 Introduction

In order to capture the current state-of-the-art as well as the field of application properly, this master's thesis discusses the underground mining method known as sublevel stoping, on a broad scale.

To get an initial impression, important basic information will be provided in the first section. This includes general facts such as the background of sublevel stoping, a comparison to other mining methods involving certain key mining aspects and last but not least a short discussion on the topic of deep mining.

Having elaborated on the basics, the "Field of Application" will be next. Here various European and Canadian stoping mines are researched and a comparison between specific geometrical and rock mechanical parameters will be made. The objective hereby is to identify and discuss trends, commonalities and differences between the individual deposits and the applied stope design.

Last but not least, the overall layout as well as the individual elements of a sublevel stoping operation are the center of discussion. Besides a graphical illustration and an elaboration on the purpose of each element, the design aspects are addressed on a broad basis. In addition, the data from chapter 3 is used to obtain reference information for the individual elements. The target of chapter 4 is to present and discuss current possibilities concerning the design process and to highlight the dependencies / interconnectedness of these elements.

2 Basics of sublevel stoping

This entry chapter is divided into three sections. The first topic which will be illuminated, is the historical background of this method. To point out the major differences to other mining methods, this is then followed by short comparison involving sublevel caving and cut and fill mining. Last but not least, it is necessary to underline the general difficulties and circumstances which emerge with increasing depth. Therefore, a general definition of “deep mining” will be the last topic of chapter 2.

2.1 Historical background

The concept of sublevel open stoping, also known as long-hole or blast-hole stoping, was originally developed in the first half of the 20th century, in Michigan. It originated from a mining method referred to as “short hole bench mining”, which was used at that time for the extraction of iron ore (Haldar, 2018). During this time period the newly evolved concept only slowly caused attention in the mining industry. In the 1970s, cut and fill was one of the most important mining methods for underground metalliferous deposits. It was very popular around the world and commonly used in countries like Canada, Australia and Scandinavia (Villaescusa, 2014). However, the rapid surge in demand for metals, called for an increased production in the mining industry, which simply was not possible with cut and fill mining.

For this reason, many company’s in various countries were forced to consider a change in their current mining concept, to enhance their production. The solution for most companies, especially in Australia and Canada, was a transition to sublevel stoping.

In the course of time and due to technological advancement, numerous versions of open stoping emerged. In particular the area of drilling and blasting as well as the use of backfill has undergone significant improvements, over the past decades. This was made possible by the ongoing research in these sectors and the emerging science of rock mechanics.

2.2 Mining method comparison

In this section a general comparison between sublevel stoping and alternative mining methods like cut and fill and sublevel caving, is made. The main reason why these methods are used for comparison, is the overlap concerning their area of application. All three methods can be applied to steeply dipping, potentially deep, tabular orebodies. However, despite this great commonality, there are many aspects in which these methods differ significantly. To get a broad overview, three main areas which are essential for these methods are going to be discussed. Through this process of analyzation, important differences, which in a further sense result in advantages and disadvantages can be identified. The points addressed by this comparison involve certain economic aspects, fundamental rock mechanical differences concerning needed environment and last but not least the area of application in regards to the deposit geometry.

2.2.1 Economic aspects

Since the economic aspects of large mining operations are very diverse and cover many subjects / issues, it is simply not possible to address all of them in detail. However, there are certain basic topics which highlight the differences between these mining methods quite clearly. In that context, the first point to be discussed concerns the productivity and efficiency of the individual methods. Following on from this cost related aspects, specifically the initial investment, will be highlighted. Last but not least the topic of 'dilution management' is briefly analyzed.

2.2.1.1 Productivity / Efficiency

The productivity of a mine strongly depends on the size of the ongoing operation. In most cases this implies: The larger the scale of the mining procedure, the better the efficiency and therefore the lower the mining costs. To be able to compare the productivity in connection with the efficiency, commonly the indicator "extraction costs per ton" ($/t) is used. In this regard typical values vary from under 10$ per ton to over 200$ per ton. The following graph developed by (SRK Consulting, 2019) provides an insight into how underground mining costs correlate with the production quantity.

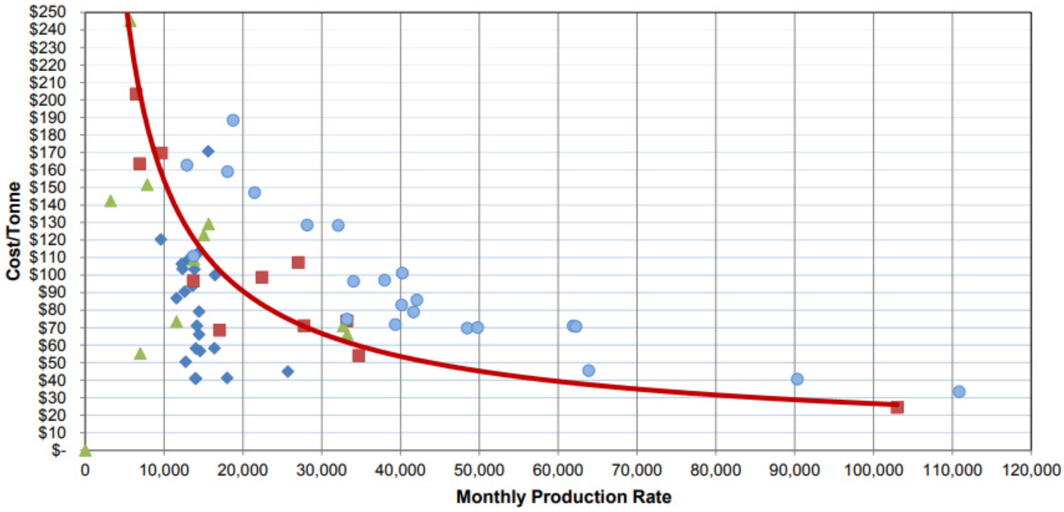


Figure 1: Underground mining costs - Production rate (SRK Consulting, 2019)

The tendency of this graph reveals the improved efficiency of large-scale mining operation. Unfortunately, there is no further information concerning the applied mining methods of these cases. However, what can be stated is that sublevel caving as well as sublevel stoping is usually characterized by a much larger production, in comparison to cut and fill mining. For example, 8 out of 10 researched sublevel stoping mines (Figure 86), indicate a production of 100,000t per month or more. According to this graph this would set the mining costs to approximately 28$ per ton. This also agrees with reports submitted by 2 of these mines. The Nerves-Corvo mine indicated 27$ per ton (Newall et al., 2017), while the Zinkgruvan mine stated mining costs of 35$ per ton (Daffern, 2017). Mines applying the sublevel caving method are usually likewise located in the lower section of the displayed cost range, as the production is similarly high, in many cases even higher. Cut and fill mining is characterized by a lower production. A detailed cost model created by (Costmine, 2021), stated that a cut and fill operation with a production of ~50.000 tons per month (which can be considered as fairly large) could indicate costs of 48$ to 72$ per ton, depending on the level of mechanization. In that example the extraction takes place at a depth of 800 meters, while hoisting is performed by shaft. Based on this condition, a fairly suitable comparison example.

One of the main reasons for the efficiency-advantage of stoping and caving operations, can be found in the larger proportions of utilized extraction units as well as in the overall less labor-intensive production procedures (none cyclical production). Furthermore, through a better possibility of high degree mechanization and automation, labor costs in relation to excavation and logistic equipment, can be reduced to a greater extend. This however is usually tied to higher initial expenses, which will be discussed next.

2.2.1.2 Initial Investment

The initial investment is defined as the required amount to start a mining operation. Depending on the used mining method as well as the scale of the operation, the difference can be quite substantial. Looking at the individual production layouts of these methods, it is apparent that sublevel stoping and -caving are characterized by a more complex production infrastructure, as much larger cutoffs have to be planned. This also correlates with the time required to start the overall mining process, as the production related preparations tend to be more time-consuming.

This indicates that sublevel stoping as well as -caving, feature a disadvantage in comparison to the cut and fill method, as not only more infrastructure is needed in the beginning, but the time from which revenues can be generated is usually inferior as well. Furthermore, depending on the level of mechanization and automation, additional time and investment is needed. This however also applies to the cut and fill method. Another important issue that is related to the higher initial investments in sublevel stoping and -caving operations, is the increasing relevance of technical knowhow, in many different sectors. This specifically concerns complex topics such as rock mechanics, blast geometry, backfill behavior and dilution control.

2.2.1.3 Dilution control

The last point which will shortly be elaborated upon, is the issue of dilution. As this is a very broad topic, ranging from insufficient experience with technical equipment to difficulties in the rock mass itself, only a few key points will be addressed. In regards to stoping the main difficulties emerge particularly at the boundaries of the stope. Inaccurate drilling and blasting, can cause the neighboring waste rock or backfill to dilute the blasted ore, leading to low grade material (Villaescusa, 2014). C&F on the other hand has the advantage of more flexibility and selectivity. The smaller extraction units naturally lead to a more accurate mining process. Furthermore, in contrast to stoping there are no dependencies to certain stope dimensions which also reduces unnecessary overcuts. In caving operations dilution control is even a more complex challenge, as this method is dependent on the gravity flow of blasted ore and caved waste rock. To predict the dilution in this mixed rock flow, complex measurements are necessary. This requires a lot of research and testing, which makes dilution control more time- and cost consuming than in cut and fill and stoping operations.

2.2.2 Rock mass environment

Most mining methods share the commonality that they can only be used under certain circumstances. The reasons for this are specific requirements which limiting the use of individual methods. An important factor for instance, is the size as well as the geometrical structure of the deposit. This criterion will be discussed separately in the next section. Another factor, which is one of the most significant for sublevel stoping and -caving operations with regards to planning and execution, can be found in rock mechanics. This should not mean that this point is irrelevant for cut & fill mining, however it does have a slightly different status.

In sublevel stoping, cavities, which are characterized by fix dimensions (also known as stopes), are used to extract an orebody. The top priority in this regard is that these stopes must ensure stability over a certain period of time. To determine whether this is achievable, it is of utmost importance to estimate the quality of the rock mass, as certain preconditions should prevail. Various example values are presented in chapter 3. Depending on the results of the conducted rock mass classification, it is then possible to evaluate whether and to what extent stoping can be utilized and how much support is necessary. To be more precise, not only the rock mass quality of the ore is of great importance, but also of footwall and hanging wall. For example, accurate estimations concerning the behavior of the hanging wall, can lead to better stability and the prevention of excessive dilution as support items like anchors and cable bolts can be used more accurately. The footwall on the other hand is usually used for the infrastructure of the mine. In that sense, access drifts, footwall drifts and pillars have to be designed and placed in such a manner, that these structures are capable of withstanding stress redistributions caused by ongoing mining activities.

In sublevel caving, the quality of the rock mass is equally relevant, however for other purposes. This method aims to achieve a controlled collapse of the hanging wall, by blasting the ore as well as the orebody boundary. Through this complex blasting / fragmentation process, it is possible to excavate the ore, by drawing it off from the established draw point. In contrast to the “stability” aim of stoping, the main objective of sublevel caving is to proper dimension the blasting / fragmentation function in connection with the gravity flow, by analyzing numerous rock mass characteristics. As in stoping, the mine infrastructure is usually constructed within the footwall. For this reason, the rock mass quality of all three aspects (Ore, HW, FW) must be considered.

Cut and fill mining is slightly different when it comes to the prerequisite of a certain rock mass quality. This method does not aim to create large cavities (compared to stoping), nor does it try to trigger a controlled collapse. One main reason for the rock mass assessment is to ensure safety and stability by estimating the needed support for the excavation process. In general cut & fill mining is very flexible in terms of rock mass quality. Especially underhand cut and fill, where the mining is carried out under an artificial roof, is unique in this context. Using this approach, it is possible to extract deposits showing even very poor rock mass conditions. However, as the costs for needed support increases, the efficiency of this method decreases. From a rock mechanical standpoint, cut and fill mining is therefore suitable for potentially deep deposits, which are characterized by difficult rock mass conditions.

The next topic to be discussed is the deposit geometry.

2.2.3 Deposit geometry

This chapter is divided into two sections in which the topic “deposit geometry” will be discussed. The first point to be addressed concerns the general framework conditions of all geometrical parameters, which should prevail for the utilization of sublevel stoping. In the second section, basic characteristics of orebody’s are outlined and a discussion concerning which mining methods would be applicable and also reasonable for an extraction, takes place. In this context once again, sublevel caving as well as cut and fill mining are brought into the discussions. It should be noted that the decision, of the mining method to be applied, cannot be based on the deposit geometry alone, as the rock mechanical aspect is of equal importance. However, by focusing for the most part on the geometrical aspects in this section, it can be demonstrated under which circumstances, sublevel stoping would be appropriate from a geometrical standpoint.

2.2.3.1 Geometrical framework conditions

The general field of application concerning stoping is rather large, as this method can be adapted to a variety of deposit geometries. In the majority of use cases however, the deposit features a massive or a tabular structure with a steep inclination between 60 and 90 degrees and a constant thickness of 10 to 100 meters. What should be noted is that these two value ranges should only be seen as an indicator, as there are mines which utilize sublevel stoping for deposits with a constant thickness of less than 10 meters. The same applies to the above-mentioned inclination range.

Thickness

In regards to the orebody thickness, it is important to note that this parameter is decisive for the orientation of the stope. Basically, there are two possible stope configurations which can be applied. The most common type nowadays is the transverse stope design. The second possibility is to arrange the stopes according to the strike of the deposit. This type of mining is also known as longitudinal stoping. These longitudinal stopes, are usually applied if the orebody or specific parts of the orebody, feature a low thickness. According to (Villaescusa, 2014) in most cases the critical value lies around 15 meters. This value was also confirmed by the conducted research in chapter 3.

Depth

Since this method is heavily dependent on the quality of the rock mass, prevailing stresses and many other geological factors, the typical depth at which sublevel stoping can be applied vary to a large scale. According to the data gathered from the researched mines, deposits located at depths between 0 and 1500 meters are commonly extracted. However, this should not indicate a boundary, as there are various mines operating at 2000 meters and below. Although stoping becomes increasingly difficult with advancing depth, a fundamental limitation could not be determined for this mining method. But more on this topic in the chapter 2.3.

To summarize this section, the following table presents all above-mentioned geometrical parameters, including example values that fall within the discussed ranges.

Strike length [m]	Thickness [m]			Dipping angle [°]	Depth [m]	Depth extension [m]
	Maximum	Minimum	Average	Dip		
800	60	5	35	70	300	1100

Figure 2: Example deposit - Geometrical dimensions

A visual impression of a deposit featuring these values is illustrated in the following figure.

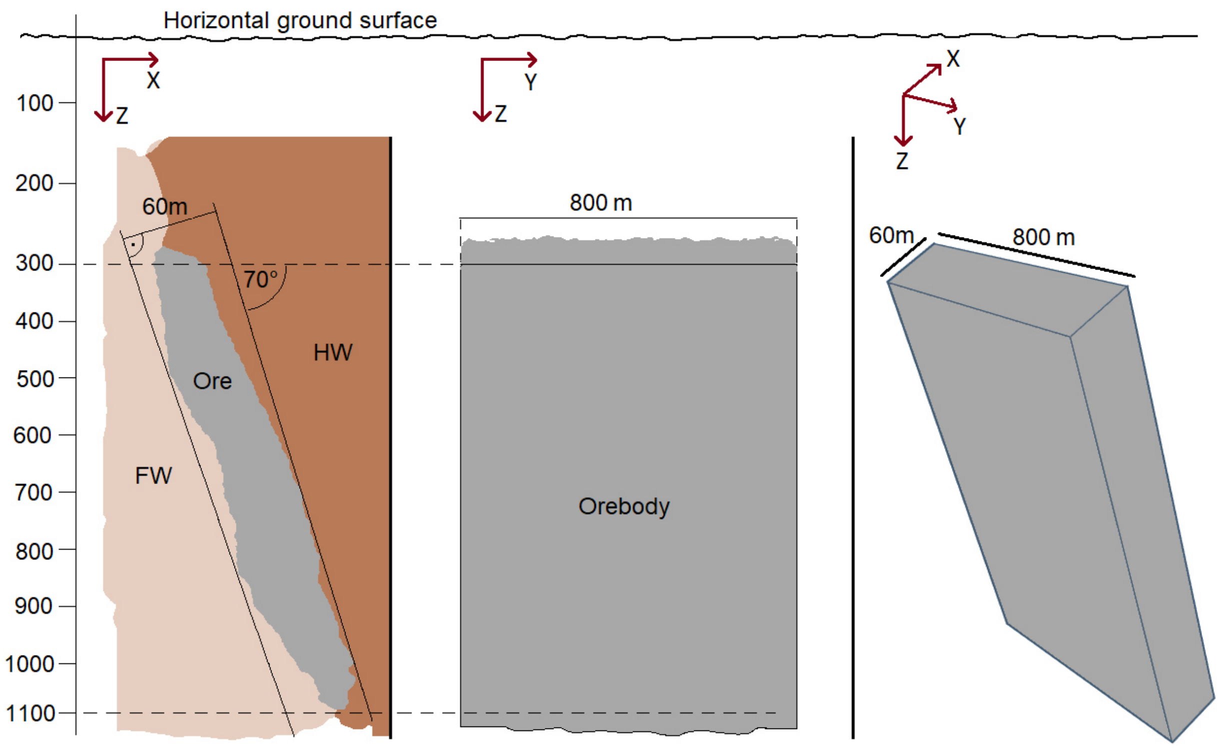


Figure 3: Example deposit – Illustration

2.2.3.2 Deposit geometry - Discussion

To get a more detailed overview concerning the area of application of the sublevel stoping method, cut and fill mining and sublevel caving are brought into discussion. For illustration purposes and as a basis for the discussion, three example contours are outlined in the following figure.

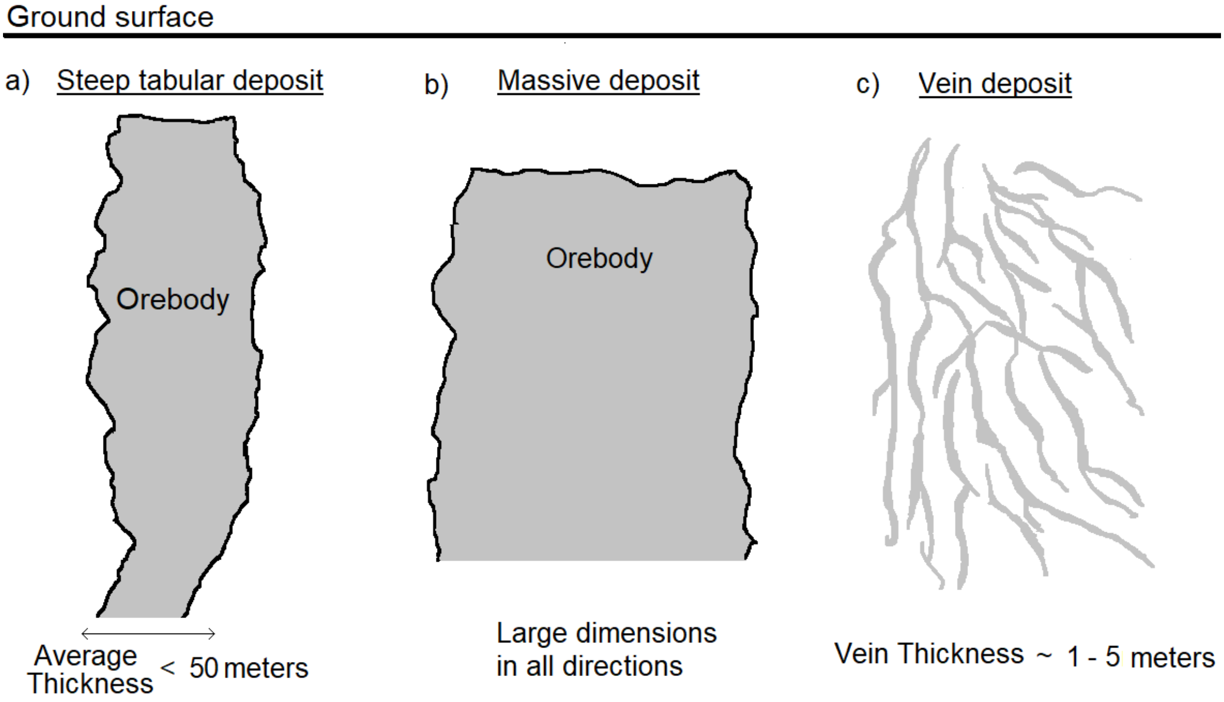


Figure 4: Deposit geometries - Discussion

Observing this figure, the characteristics shown in case “a)” can be reasonable for all three mining methods. In order to analyze which extraction method would be most appropriate for a deposit that resembles this structure, geometrical criteria such as the continuity, the main trend of alternation as well as the average thickness, are important to research. As sublevel stoping lacks in flexibility compared to cut and fill mining, large variation in the orebody thickness are generally more difficult to manage. For instance, if the thickness varies significantly between 3 and 12 meters, the extraction infrastructure including the stope orientation, might require frequent adjustment. If large parts of the orebody are characterized by low thickness (less than 7 meters) and strong irregularities, a more efficient approach could potentially be a cut and fill operation. The higher flexibility and selectivity can prove to be extremely useful under such circumstances. A similar situation arises when the deposit shows a fairly large thickness. As the efficiency of sublevel caving rises with increasing orebody thickness, the higher infrastructural effort can also be compensated more quickly. Therefore, sublevel caving would be an appropriate option if the average thickness stays above a certain level for the most part. However, since each deposit has its own unique characteristics, the question of which mining method to select can only be answered for each orebody individually.

The next deposit type to be addressed is known as massive deposit and outlined as case “b” in the figure above. In general, there are many different mining methods that can be applied to extract this type of orebodies. Since there are no major geometrical restrictions compared to tabular or vein deposits, methods that feature high efficiency / productivity are typically preferred. This is one reason why large block- and sublevel caving operations are often applied for such structures. However, sublevel stoping can be a rather good alternative if the rock mass is characterized by suitable rock mass conditions. The use of cut and fill mining is also possible, as this method has less limitations than any other method. However, since the efficiency is much lower, it would only make sense as a last resort in massive deposits.

Last but not least, case “c” (outlined in the figure above) will be discussed. In general, it can be stated that the more irregular the course of veins appear, the more difficult it becomes to efficiently recover ore using sublevel stoping. To justify the planning of static structures such as stopes, some kind of continuity should be present within an orebody. Furthermore, in order to deal with strong irregularities, it is advantageous to have a very well-researched and explored deposit to minimize the occurring dilution. Overall it very much depends on the degree of structural variability, the thickness of the veins, the ore grade and the rock mass conditions, whether sublevel stoping can be applied profitable in such an environment. A method which is utilized quite commonly for vein type deposits is cut and fill mining. The better selectivity reduces the occurring dilution to a minimum. Furthermore, through the advantage of more flexibility, emerging issues which are related to changes concerning geometrical or rock mechanical parameters, can be handled faster and more effectively. In regards to sublevel caving, it can be stated that the infrastructure of this extraction method is very suitable for orebody’s featuring large / broad dimensions, but not for small constantly changing vein structures. However, a further possibility to extract a vein type deposit, is to treat it as a massive deposit. In this case a considerable amount of exploration would be necessary to review the profitability.

To get a more detailed impression which deposits are suitable for stoping operations, chapter 3 contains numerous researched real-world use cases.

2.3 Deep underground mining

Sublevel stoping is a mining method that can be utilized in shallow as well as in deep deposits. An interesting question in this context is: “At which depth is a mine considered to be classified as deep?” To answer this question, various aspects of “deep mining” have to be examined. The first thing what naturally comes to mind, is that the simplest form of classification, could be a categorization according to the depth of the mine. An example, for such a classification system, can be found in the Chinese coal industry. Based on different circumstances like geological conditions, support and mining method, the upper limit for deep coal mines in China, is set to between 600 and 800 meters. The lower limit on the other hand is fixed to 1200 meters, with some subdivisions in-between. (HU She-rong, 2010) A mine exceeding 1200 meters in depth, is defined as ultra-deep. (Wang, 2019) Such a categorization can be seen as useful, if mines of the same type, with similarities in geology and rock mass, are compared to each other. The downside however, is that it can hardly be used for an overall classification.

The main question that arises in this context is: “What is the core issue, related to increasing depth?” According to (Wagner, 2019) the most apparent effect of depth, which commonly influences mining structures, is the rise of rock pressure. However, he further stated that rock pressure related mining problems are also dependent on other factors. One significant parameter which was mentioned in that relation is the strength of the rock mass. Due to these circumstances similar “bad” or “good” conditions can therefore prevail at different depths, depending on the geology and rock mass of a deposit. Another factor, which is also significant for the prevailing mining conditions, are the thermal properties of the rock mass. To illustrate why this aspect must be taken into account, (Wagner, 2019) made a comparison of a coal mine and a South African gold mine. While the coal mine indicates a temperature increase rate of approximately 3°C per 100-meter depth, the south African gold mine only measures a rate of 1°C per 100-meter depth. The much higher rate in the coal mine leads to earlier difficulties regarding heat, ventilation and other thermal related problems. This is a further reason, why most definitions of “deep mining” refer to changes in mining conditions instead of fixed depths. (Wagner, 2019)

Considering these points, the overall mining conditions which prevail in a “deep mining” environment can be described as followed. Rock mass which is located in “high” depth, is distinguished by high temperature, high water pressure and high in-situ stresses. Mining within these constant framework conditions, lead to the circumstances that rock mass failures as well as rock bursts are more likely to occur. Furthermore, such environmental conditions naturally result in greater physiological strain for workers.

For these reasons, it is not useful to define “deep mines” by their actual depth on a general basis. However, by comparing various mines in geologically and geographically different locations, it is possible to determine several relative circumstances which emerge at a certain depth. These prevailing conditions can then be used to categorize a mine as “deep”. However, in consideration of all these aspects, a typical depth at which deep mining problems (conditions) start to emerge more commonly, lie around 1500 meters, in hard rock mines. (Wagner, 2019)

3 Field of application

In this chapter, real-world examples of mines from different parts of Europe and Canada which apply sublevel stoping as their mining method, are presented (Section 3.2 and 3.3)

To illustrate general differences as well as the commonalities of these operations, a comparison concerning various geometrical and rock mechanical parameters is made later on (Section 3.4). First, however, a brief but important comment on the data that was collected for this comparison.

3.1 General information on the researched data

In regards to the researched data a few remarks have to be made. The information gathered in this chapter originated from a wide variety of sources involving technical reports, master theses, scientific papers, conference papers, general studies as well as online databases. For this reason, the provided information on some mining operations is more accurate as well as more traceable than on others. Another important point is that some of these reports / papers are rather old which indicates that parameters could be quite different today. Furthermore, the presented data concerning stope design usually relates to certain main areas of the individual mines and should only be seen as “typically used” values. As environmental conditions vary, stope dimensions have to be adapted during the mining process. The same applies to the rock mass quality. The overall situation of a mine cannot be represented by generalized values from a few measurements which are publicly available. However, what this gathered data does demonstrate, is a general overview concerning the field of application of sublevel stoping as well as an insight into the differences and trends concerning applied stope designs.

3.2 European mines

To get a broad impression how the sublevel stoping method is applied in European mines, this chapter will highlight various stoping operations. Thereby each mine will be described in the topics - “Deposit Characteristics”, where general information about the orebody is presented - “Stope Dimensions”, in which geometrical data concerning the infrastructure is summarized and last but not least “Rock mass data”.

3.2.1 Magnesite deposit – Austria (Breitenau mine)

The main mining method that was applied for the extraction of this magnesite deposit, is called post pillar mining. The decision to use this method, was based on the lens-shaped structure and the irregularity of the orebody. However, in later stages, a change of the mining method, took place. The reasons for the modification of the excavation method, were rock pressure related and it was decided to use stoping in combination with backfill on the deeper levels. (Wagner, 2015)

Deposit characteristics

With a dipping angle of only ~25° the overburden is rather small in the beginning, but increases rapidly due to the mountainous circumstances. The thickness varies from 50 to 200 meters and the strike length extends to a maximum of 500 meters. The length of the deposit in dip direction is estimated to be 2000 meters. (Henjes-Kunst, 2014)

The following figure visualizes the described deposit.

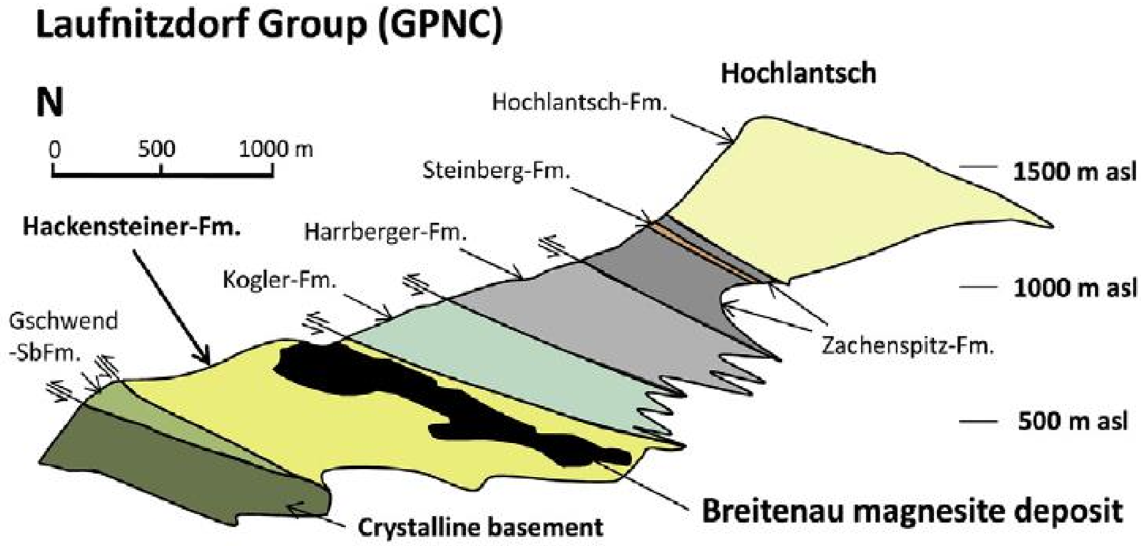


Figure 5: Breitenau mine, Austria (Henjes-Kunst, 2014)

Stope dimension

The stopes measure a width of 6 to 7 meters and a fixed height of 21 meters. The length usually varies between 70 and 80 meters. For stability reasons all stopes are separated by pillars showing a width of 7 meters. However, these pillars can usually be considered as secondary stopes. The working depth in these cases, is estimated to be 800 meters below surface. (Wagner, 2015)

What should be noted, theses stopes are oriented longitudinal. However, since the deposit measures a shallow angle and decent thickness, stopes are placed adjacent to each other.

Rock mass data

The strength of the rock mass is derived from back analysis of mine tunnels, which are situated in the deeper parts of the deposit. In these tunnels several 8.5m long horizontal core holes have been drilled and analyzed. In addition to the study of the core samples, numerical simulations have been conducted. The results indicated, that the core samples are consistent with a numerical analysis, based on a rock mass strength of 50 MPa. Furthermore, a calculation, using the RCF (Rock wall condition factor), was also carried out. This method for calculating the critical rock mass strength, which was initially developed for South African gold mines, resulted in a value of 47,25 MPa. (Wagner, 2015)

The values used for this calculation are shown below:

$$\sigma_{crit} = 0.7 * 0.5 * 135 = 47.25\ [MPa]$$

UCS = 135 MPa .....Uniaxial compressive strength
RCF = 0.7 ....Rock wall condition factor
F = 0.5 ....Strength reduction factor

Figure 6: Calculation of the critical rock mass strength

To obtain further comparative values, the rock mass was additionally estimated based on the Hoek-Brown method. The GSI value of the investigated location, was set to be between 65 to 75. The results can be seen in the following table. (Wagner, 2015) (Wagner, 1987)

	Disturbance factor D=0				Disturbance factor D=0.3			
GSI	σ_c MPa	σ_{cm} MPa	c MPa	φ °	σ_c MPa	σ_{cm} MPa	c MPa	φ °
60	14.5	29.5	7.9	33.5	11.3	25.6	7.2	31.3
65	19.2	33.3	8.7	34.9	15.4	29.3	7.9	33
70	25.8	37.9	9.6	36.3	21.1	33.8	8.8	34.8
75	33.6	43.9	10.8	37.7	28.8	39.6	10	36.4
80	44.4	51.6	12.3	39	39.2	47.2	11.5	38

Figure 7: Rock mass strength parameters (Wagner, 2015)

Concerning hanging wall and footwall the RMR was determined to be 30, which would categorize the rock mass as “poor” in terms of quality. (Schenkl, 2014)

3.2.2 Polymetallic deposit – Sweden (Boliden Garpenberg mine)

Garpenberg is the oldest mining area in Sweden and is known to be one of the most productive zinc mines in the world. The area of Garpenberg is divided into several mining sectors, containing numerous deposits. The two largest orebodies in the Garpenberg region, are Dammsjön and Lappberget. It should be noted that all of the following data in this section refer to the Lappberget orebody.

The main resource being extracted from this mine is zinc, but also lead, copper, gold and silver is recovered. Back in 2003 the mining process began, by using the cut and fill method for extraction. Several years later, a change of the mining method, to transverse sublevel stoping, took place.

Deposit characteristics

With a dipping angle of 80 to 90 degrees, the orebody is classified as steeply dipping. Furthermore, the thickness varies between 15 and 120 meters, while the strike measures a length of 250 meters. The orebody in general, is situated at a depth of 500 meters and extends to 1400 meters. (Souley, 2018) (Ghasemi, 2012)

The following figure illustrates the Garpenberg region.

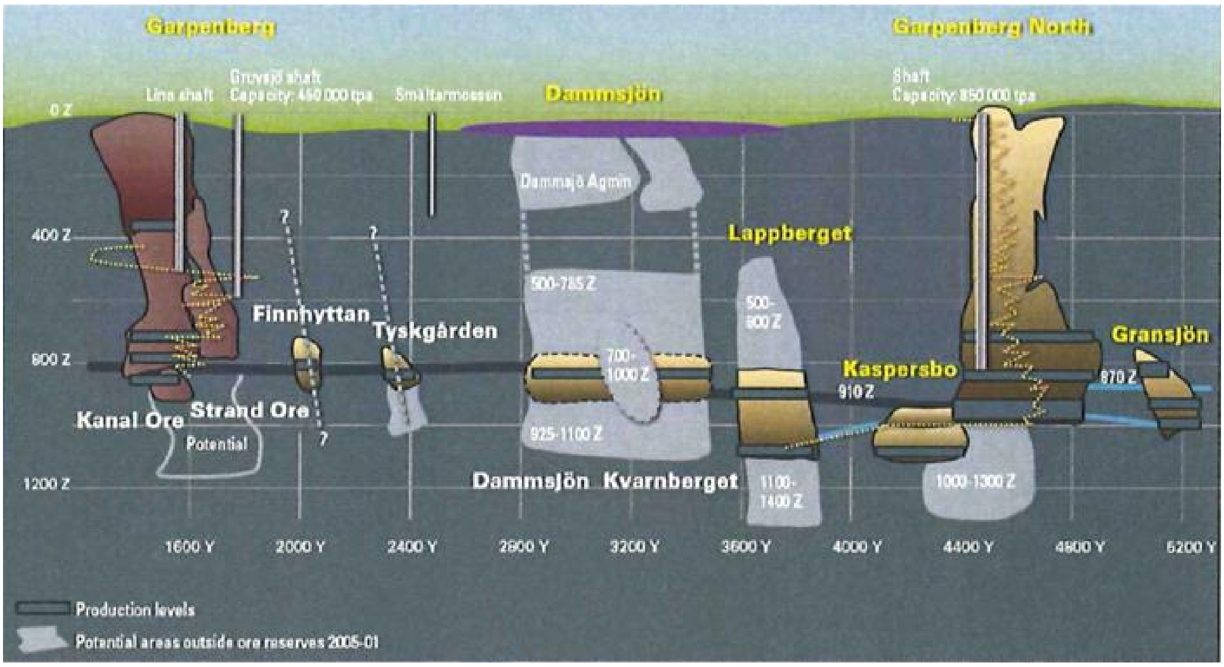


Figure 8: Garpenberg region (Van Koppen, 2008)

Stope dimension

The stopes used in the Lappberget orebody are arranged transverse, in a primary - secondary pattern. Additionally, a typical pyramidal mining structure is applied. Concerning the width, primary stopes vary between 10 and 15 meters, while secondary stopes are slightly larger showing a width of 15 to 20 meters. The length of the stopes is dependent on the orebody thickness and therefore also differ. However, an average value lies around 80 meters. The height of a stope is adapted to the current depth. This means that stopes blow the 1040-meter level (>=1040 m) measure a height of 20 meters while stops above the 1016-meter level (<=1016 m) have a height of 30 meters. In between the 1016 and 1040 level the height is set to 24 meters. (Van Koppen, 2008)

Rock mass data

Regarding rock mechanical data, several biaxial tests were carried out over the years. The results can be seen the table below.

Values From Depth [m]	Source	Density overburden [t/m³]	UCS [MPa]	E [GPa]	Poisson ratio
1155 - Ore:	Bouffier (2015)	3,03	150	45 and 64	0,23
852 - Ore:	Dahle (2005)	3,33	196 +-65	84,7	0,15
852 - Ore:		3,27	146 +- 14	90,7	0,15
883 - Limestone:	Nilsson (2004)	2,71	**73**	**55**	**0,17**
967 - Limestone:		2,72	**100**	**60**	**0,12**

Figure 9: Rock mechanical properties – Garpenberg

Furthermore, a geomechnical numerical modelling was developed, to evaluate seismic and aseismic rock deformation. The reason for this was to investigate the response of deep underground mining. The parameters retained for this numerical modelling, were as followed.

Materials	UCS[a] [MPa]	mi	GSI[b]	E [GPa]	ν	m	s
Ore	188	20	80	66	0.2	10	0.112
Limestone	110	20	80	57	0.18	10	0.112
Weak	30	9	38	20	0.3	1.0	0.001
Very weak	10	9	25	2	0.4	0.63	0.00024
Pastefill	–	–	–	0.5	0.2	–	–

Figure 10: Parameters for numerical modelling (Santis, 2020)

In 2005 a general analysis of 28 drill cores, from various rock types of the Lappberget orebody was carried out. The results showed that nearly 80% of the total mapped length, indicated an RMR higher than 60. The values can be seen in the following figure.

Class	RMR	% of total mapped length
Very poor rock	0-20	0
Poor rock	20-40	1
Fair rock	40-60	20
Good rock	60-80	39
Very good rock	80-100	40

Figure 11: RMR – Lappberget (Nystörm, 2005)

3.2.3 Gold deposit – Finland (Agnico – Kittilä mine)

The Kittilä mine, which started its production in 2009, is located in the northern part of Finland. The purpose of this mine is to extract the Suurikuusikko gold deposit, which indicates an average ore grade of 4,9 g/t. In the beginning of the operation open pit mining was applied to extract the Suuri and Roura orebody. Later on (in 2012) a conversion to underground mining took place. (Wyche, 2015)

The mining method which is applied so far, is longhole open stoping with delayed backfill. Thereby transverse as well as longitudinal stopes are used. The main criterion for the used orientation, depends on the thickness of the deposit. The separation value for this decision is determined to be at 7 meters. So transverse stopes are only used, if the thickness of the orebody exceeds this value. (Tommila, 2014)

Deposit characteristics

The deposit measures a strike length of 5000 meter and a varying thickness of 3 to 40 meters. The average value however is determined to be around 7 meters. In regards to the depth, it has been observed that the orebody extends to at least 1500 meters below surface, featuring a dipping angle of 70° to 90°. The deepest stopes in operation, are situated at a depth of 1175 meters. A longitudinal section of the deposit is shown in the figure below. (Coucet, 2009) (Tommila, 2014)

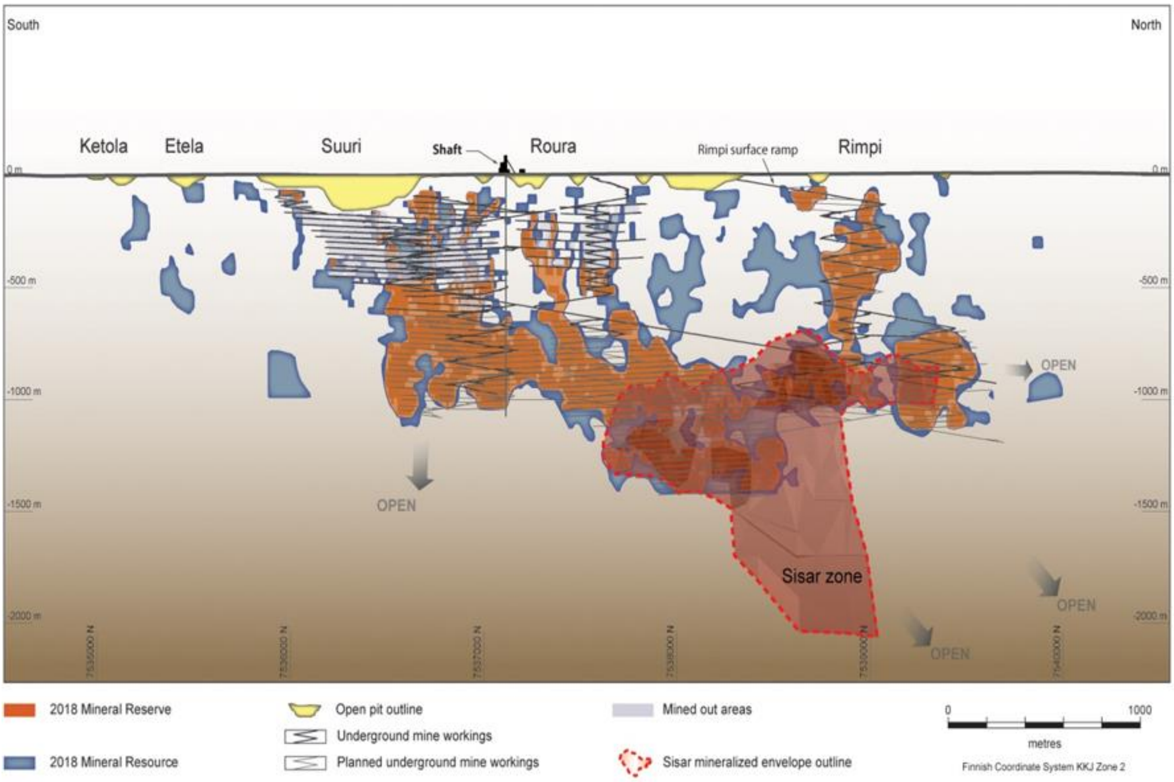


Figure 12: Longitudinal section – Kittilä mine (Agnico Eagle, 2020)

Stope dimension

As mentioned above, transvers and longitudinal stopes are utilized. In this context it should be noted, that the average thickness of this orebody (75%) lies below 10 meters, which classifies the deposit as narrow. This indicates that 75% of all transverse stopes feature a length of < 10 meters. The maximum length on the other hand is set to be 35 meters. Concerning longitudinal stopes, width and length are deliberately switched. Here, the width (orebody thickness) is set to a maximum of 7 meters and the length has been tested “with varying success” up to 60 meters. In regards to the height, both stope types apply a value of 25 meters, if the overburden exceeds 400 meters. In shallower parts of the mine, the height is set to 40 meters. (Tommila, 2014)

Rock mass data

To evaluate the rock conditions three different tests were carried out at the Aalto University: The Bond Ball Mill Test, the Drop Weight Test and the Point Load Test. The results of this investigation can be seen in the following table. It should be noted that the three-digit number of the sample code refers to the depth, while the letter indicates the orebody from which the sample originated.

(S = Suurikuusikko, R = Rouravaara)

Sample code	Bond Ball Mill Test BMWi (kWh/t)		Drop Weight Test Axb (-)		Point Load Test UCS (MPa)	
1S175	18.62	hard	33.52	hard	119,01	high strength
2S175	17.75	mod. hard	30.46	hard	113,33	high strength
1S325	19.38	hard	39.49	mod. hard	114,76	high strength
2S325	18.85	hard	38.01	hard	123,24	high strength
1R390	15.81	mod. hard	30.54	hard	85,43	medium
2R390	16.56	mod. hard	30.72	hard	112,83	high strength
1X900	20.40	very hard	45.93	medium	149,84	high strength
2X900	18.31	hard	33.41	hard	116,25	high strength

Figure 13: Drill core results (Lange, 2019)

Besides the data shown in previous figure, further tests have been carried out in the more distant past. Based on these older drill core samples (Eloranta, 2012), the value for weak and moderate rock mass was estimated. These samples originated from the top parts of the Suuri and Roura orebodies. The results were as followed.

Moderate rock mass	Weak rock mass
UCS = 132 MPa	UCS = 89 MPa
E = 69 GPa	E = 61 GPa
v = 0,25	v = 0,24

Figure 14: Estimation values for weak and moderate rock mass (Tommila, 2014)

In order to estimate the GSI value for weak and moderate rock mass, a correlation between RQD and GSI values was identified. The data used for this calculation, originated from geotechnical loggings of six boreholes.

The result was the following expression as well as the values obtained by it:

$$GSI = 0{,}33 * RQD + 37 \quad \text{(Tommila, 2014)}$$

	Moderate rock mass	Weak rock mass
RQD	49,8	14,2
GSI	53	42
mi = 17 , D = 0,7		

Figure 15: GSI values - Kittilä mine (Tommila, 2014)

3.2.4 Chromite deposit – Finland (Outokumpu's Kemi mine)

The Kemi chromite deposit, which is located in Finland (northeast to the town of Kemi) was discovered in 1959. Several years after the discovery, the production began by utilizing open pit mining. After reaching a depth of 200 meters (in 2005), a full conversion to underground mining took place. The method which is in use since then, is sublevel stoping with the utilization of backfill.

Deposit characteristics

The full length of the intrusion is known to be 15 km. However, the thickness only reaches centimeters, towards both ends. The more interesting (thickened) part of the intrusion, which strikes approximately 4 km, features a maximum width of 100 meters. The average thickness regarding this section lies around 40 meters. With a dipping angle, ranging from 60° to 70°, the orebody is defined as steeply dipping. The current section of the deposit, which is already in extraction since the last decades, is located between the former open pit (220 m below surface) and a depth of around 550 meters. The depth extensions of the deposit are estimated to be 1000 to 2000 meters. In regards to this, exploration is going on in the deeper parts of the mine. The following figure displays a cross section of the Kemi deposit. (Huhtelin, 2015)

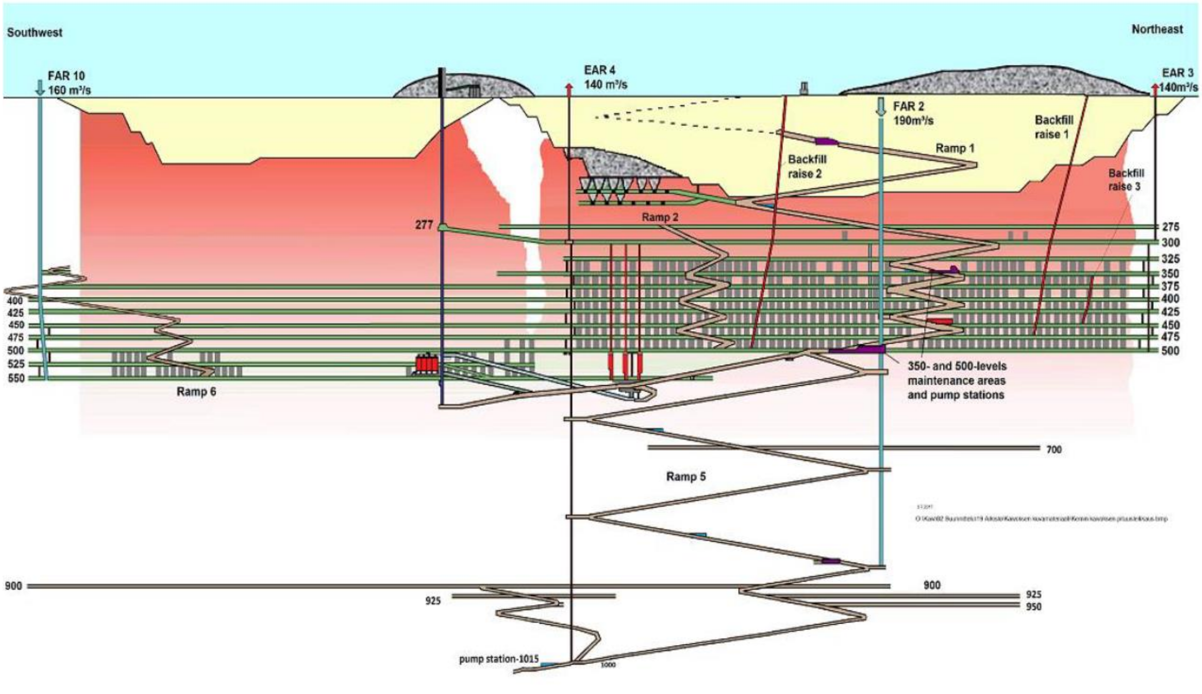


Figure 16: Longitudinal section - Kemi Chromite deposit (Outokumpu Chrome Oy, 2018)

Stope dimension

The stopes used in the Kemi mine, are mainly oriented transverse. However, also longitudinal stopes are applied in some sections of the deposit. The transverse stopes are designed to be 25 meters in height and 30 to 40 meters in length. Regarding the width, the values vary between 12 and 20 meters. Which value is used, depends mainly on the current rock mass conditions as well as on the type of the planned stope. The following table summarizes the different widths values, which are applied on various levels. (Mindat.org, 2020)

Depth [m]	Primary [m]	Secondary [m]
300 - 375	12	15
450 - 500	17	18
525 - 550	15	20
Weak ore	12	12
Longitudinal	20-30	/

Figure 17: Stope width – Kemi mine (Rikberg, 2019)

Rock mass data

The main rock types, which occur near the deposit are Granite, Mylonite, Talc-Carbonate and Metaperidotite. The values which are obtained from tests, are listed in the table below.

Rock Type	UCS [Mpa]	GSI	Location
Granite	150	62	FW
Mylonite	28	40	FW
Talc-Carbonate	41 - 56	59	FW
Ore	41 - 56	62	/
Metaperidotite	120	60	HW

Figure 18: Rock types in Kemi mine (Gustafson, 2014) (Rikberg, 2019)

The most problematic rock type in regards to stability is the Mylonite. This zone is located in several sections of the footwall. The talc-carbonate zone, likewise indicates difficulties in stability. The following figure displays the distribution of these zones.

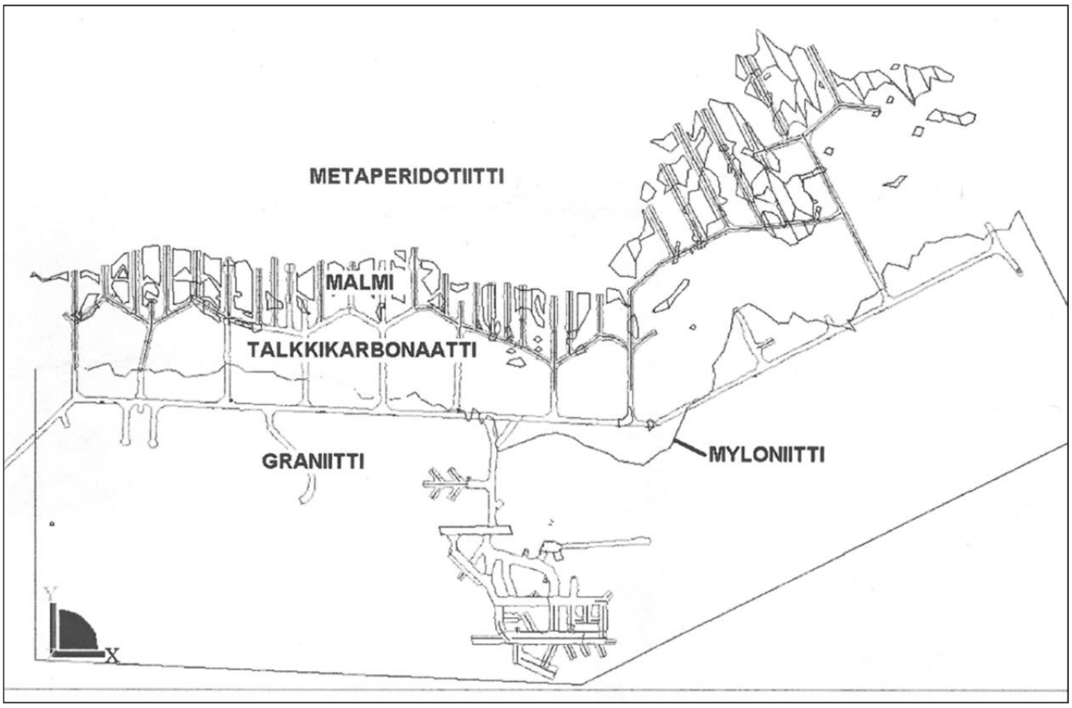


Figure 19: Horizontal view – Production area (Gustafson, 2014)

In the process of geological mapping, visible joints including their characteristics were evaluated. No recognized primary joint formation or foliation with known spacing is present.

3.2.5 Zinc-Lead deposit – Sweden (Lundin Mining – Zinkgruvan mine)

Zinkgruvan, a small village located in south central Sweden, is known for its mining activities since 1857. The deposit located in this area, is referred to as the Zinkgruvan deposit and comprises several orebodies. An important geological characteristic is that, due to a major vertical fault, the entire Zn-Pb-Ag stratiform is split into two sections. While the western part, in which the Burkland orebody is located, is known as the Knalla mine, the eastern part was named Nygruvan mine. Another interesting feature is that a copper stockwork mineralization, is situated in the direct hanging wall of the Burkland orebody. (Jansson, 2017) (Daffern, 2017)

Deposit characteristics

The deposit primarily strikes east-west, measures a length of 5 km and extends to a depth of 1500 meters. Thereby the Burkland orebody is situated at a depth, ranging from 200 to 1500 meters. Regarding the thickness and the dipping angle, the orebody measures a width of 3 to 40 meters and a dip ranging from nearly vertical to sub horizontal, with extensive folding. The average thickness is estimated to be 7 meters. The following figure displays an overview of the region. (Malmström, 2008)

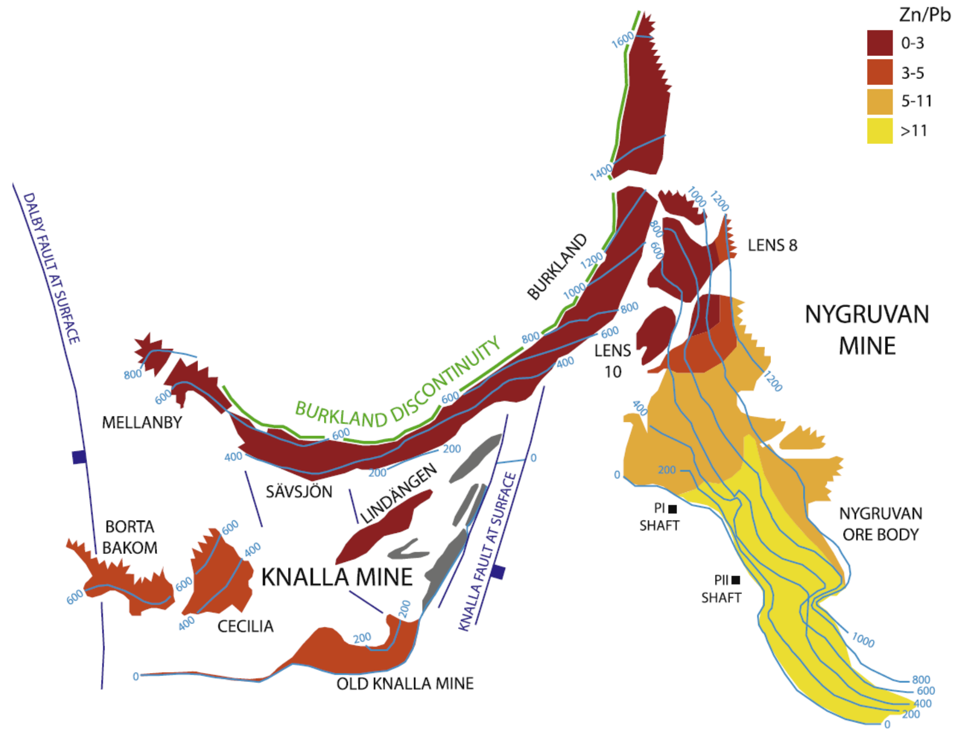


Figure 20: Horizontal view of the Zingruvan deposit (Jansson, 2017)

Concerning the dimensions of the copper stockworks, the strike measures a length of 100 to 180 meters, while the thickness varies from 5 to 60 meters. The average value of the thickness is estimated to be 20 meters. Overall, the copper stockwork is most distinctive at depths between 700 and 1100 meters. As in the case of the Burkland orebody, the dip changes significantly with the depth. For instance, while the Cu-orebody measures a dip of 80° between the 600- and 700-meter level, it flattens out to 45° near the 1000-meter level. The following picture shows an 3D overview, including both sections of the mine, as well as the copper stockworks. (Malmström, 2008) (Jansson, 2017)

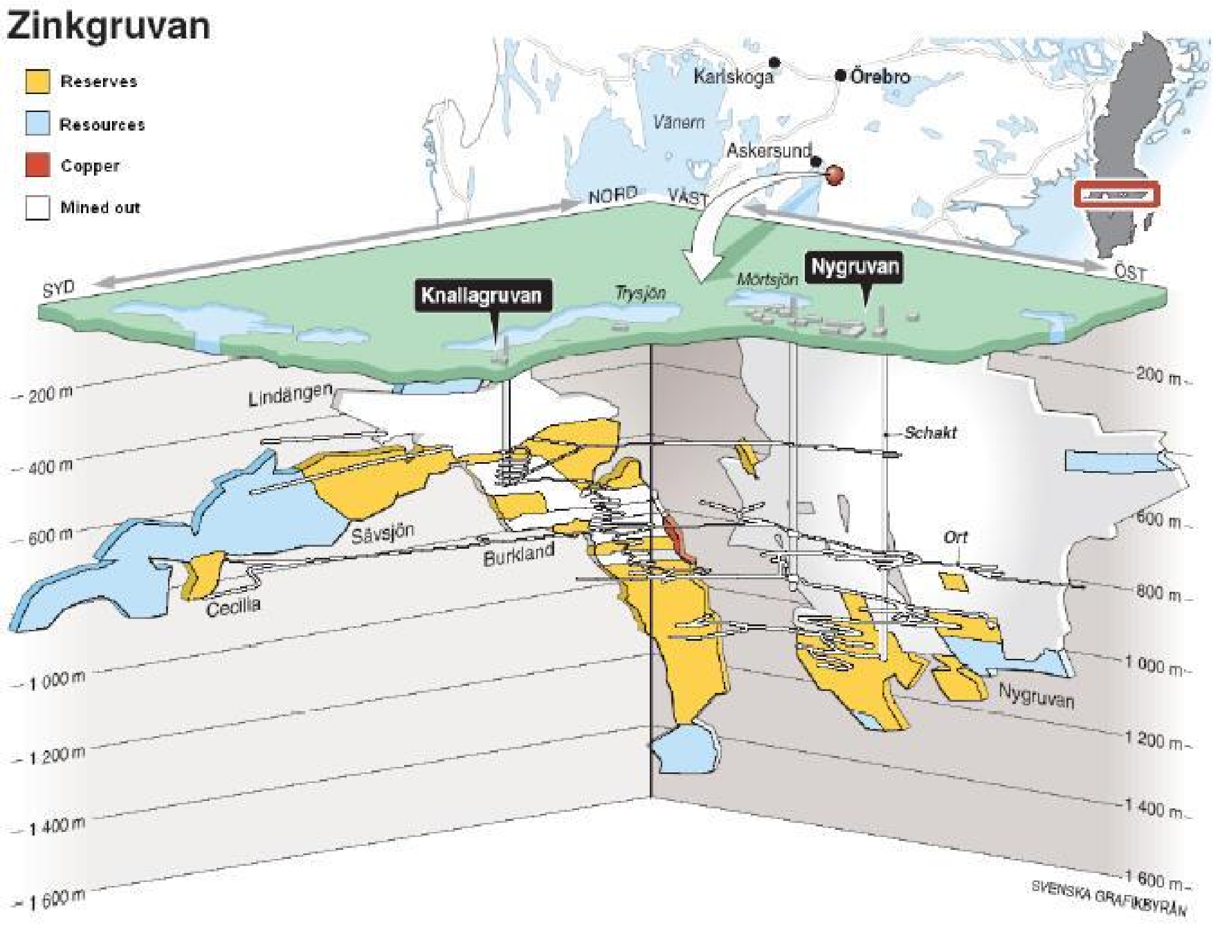


Figure 21: Overview – Knalla and Nygruvan section (Malmström, 2008)

Stope dimensions

The main mining method used for the Burkland deposit is called long hole transverse bench & fill stoping. Within this process transverse stopes, with a sequence order known as "primary – secondary", are utilized. The applied backfill material is paste fill. As there are multiple sources concerning applied stope dimensions, the following list provides an overview.

Stope - Dimensions			
P - Width [m]	S - Width [m]	Height [m]	Source
20	25	35	(Van Koppen, 2008)
30	20	20	(Daffern, 2017)
20	25	38	(Malmström, 2008)

Figure 22: Transverse Stope dimensions - Burkland deposit

Observing this data, it is apparent that between 2008 and 2017, a shift to smaller stope dimensions was performed. In fact, this would not be the first time as according to (Sjöberg, 2005), cave-ins from the hanging led to a reduction of the stope size in previous years. The applied transverse stope length varies between 15 and 25 meters. (Daffern, 2017)

In regards to longitudinal stoping, a "switching" point might be around an ore thickness of 11 meters, as according to (Daffern, 2017) a stable width of 11 meters can be achieved. The minimum mining thickness varies between 3 and 5 meters. (Malmström, 2008)

What should be noted, the diagrams presented in this chapter 3.4 only displays the most recent data.

Rock mass data

Rock stress data is available from a depth of 965 meters. The measurements were conducted in the zinc ore zone and resulted in rather high stress values. Thereby the orientation of the major principal stress, was directed perpendicular to the strike of the ore. The values are as followed:

$\sigma_H = 65$ MPa $\quad\quad \sigma_h = 45$ MPa $\quad$ (Sjöberg, 2005)

In addition to this data, UCS and E values were measured from various rock masses. The GSI ratings are based on estimations and previous experience.

The geological structure of the Knalla section, including the rock properties are displayed in the following table. (Van Koppen, 2008)

Rock Type	UCS [Mpa]	E [GPa]	Poisson ratio	GSI	Location
Biotite Leptite	175 +- 75	60	0,25	50-60	FW (Zn-Pb)
Leptite and Skarn-leptite	175 +- 75	70	0,25	50-65	FW (Zn-Pb)
Zinc-lead ore	225	75	0,25	60-70	Zn-Pb Ore
Leptite and Skarn-leptite	175 +- 75	70	0,25	50-65	HW (Zn-Pb)
Limestone/marble	100	55	0,25	60-65	FW (Cu)
Copper ore	165	65	0,25	55-65	Cu Ore
Quartz-feldspar leptite	300	71	0,33	70-85	HW (Cu)

Figure 23: Rock mass properties – Burkland orbody (Sjöberg, 2005)

Also noteworthy, after several stress measurements, the Knalla fault (which is located in the footwall of the Burkland orebody) indicates no apparent stress rotation. This signifies, that the fault is interlinked and able to transfer shear stresses. (Sjöberg, 2005)

3.2.6 Copper-Zinc-Sulphur deposit – Finland (Pyhäsalmi mine)

The Pyhäsalmi mine is located in central Finland and started its production in 1962. In the beginning of the mining operation an open pit was used to extract the ore. After several years of exploration, the data revealed that the orebody extends further, up to a depth of 1400 meters. In 1967 the open pit reached its final depth of 125 meters and a transition to underground mining took place. The selected mining method was transverse sublevel stoping as well as longitudinal stoping. In this context, longitudinal stoping was the main extraction method until a depth of 1050 meters was reached. At this level the thickness of the orebody broadened and transverse stoping became more efficient. (Hustrulid, Bullock, 2001)

Deposit characteristics

The orebody measures a strike length of 650 meters, a thickness of up to 100 meters and an average dipping angle of 70°. (Hustrulid, Bullock, 2001)

Stope dimensions

Regarding the used stope dimensions, two sources comprising different data, could be found. The first source is Hustrulid and Bullock from 2001, which stated the following.

Longitudinal stopes which were used in the upper parts (<1050 m) feature a height of 40m and a maximum width of 20m. Below this level, mainly transverse stopes measuring a height of 50m are applied. The average stope size concerning transverse stopes, is estimated to be between 40.000 and 50.000 tons. The mining sequence applied in the deeper parts (>1050 m) is performed as followed. Primary stopes (height of 50 m) are extracted as a whole using a bottom level, one sublevel and an upper level. Secondary stopes are mined in two stages, each bench measuring a height of 25m. The upcoming figure, provides an overview of the sequence. (Hustrulid, Bullock, 2001)

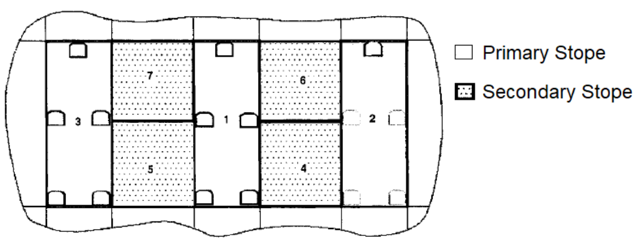


Figure 24: Mining Sequence - Pyhäsalmi mine (Hustrulid, Bullock, 2001)

The second source regarding stope dimensions is the annual information from December 2019. Since this data is more up to date, these values will be used in the diagrams of chapter 3.4.

Primary stopes with single draw points measure a height of 20 to 30 meters, a length of 30 to 70 meters and a width between 15 and 20 meters. Secondary stopes are for the most part identical and differ only in the width, ranging from 20 to 25 meters. The overall stope size varies between 30.000 tons and 150.000 tons. It is further stated that each stope has to be designed individually, due to the difficulties in rock mechanics. (First Quantum, 2020)

Unfortunately, there is no information on the actual depth concerning the used stope dimensions. However, the timing of these reports suggests that the "newer" stope dimensions are potentially used in deeper regions.

Rock mass data

The UCS of the ore, which mostly consist of pyrite, ranges between 92 and 202 MPa and seems to be quite competent. The hanging wall and footwall mainly comprise felsic volcanic rocks and likewise provide good stability featuring a UCS value of 180 MPa. However, a weak talc-schist layer, measuring an UCS of only 62 MPa, occurs in the lower part of the deposit near the footwall and in the upper part in the orebody itself. (Hustrulid, Bullock, 2001)

The rock mass quality for the host rock, which was calculated with the Q-System resulted in good to very good conditions. Furthermore, the massive pyrite showed even better results, varying between very good and extremely good Q-Values. The surroundings of the talc-schist zone, seems to be the most problematic area, showing values between fair and poor. All values are summarized in the table below. (Hustrulid, Bullock, 2001)

Area	UCS [MPa]	Q - Calculated	GSI - Converted
Ore	92 - 202	10 - 400	64 - 97
HW	180	10 - 100	64 - 85
Talc-shist zone	62	1 - 10	44 - 64

Figure 25: Rock mechanical related data - Pyhäsalmi mine (Hustrulid, Bullock, 2001)

3.3 Canadian mines

In this chapter some individual Canadian mines as well as general studies that are related to Canadian sublevel stoping operations, are now being discussed. The topics are structured similarly to the previous chapter.

3.3.1 Gold deposit – Ontario, Canada (Williams mine)

The Williams Mine, established in 1985, is still in operation to this day and is one of Canadas largest gold mines. The method used to extract the deposit is transverse and longitudinal longhole stoping, with a primary-secondary stope sequence.

Deposit characteristics

The deposit strikes east-west, dips to the north and can generally be classified as steeple dipping, showing an angle between 60° and 70°. Regarding the thickness of the orebody, the measured values ranges between 4 and 45 meters. The orebody continues to a depth of 1400 meters. (Hustrulid, Bullock, 2001) The following figure provides an overview concerning the mine structure.

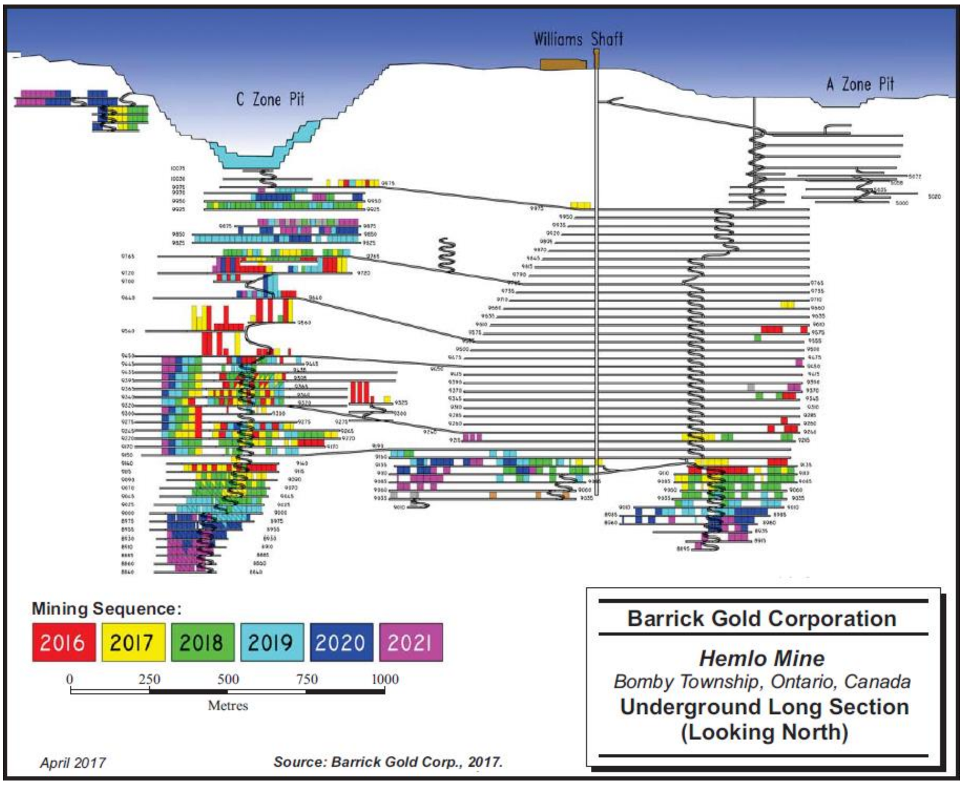


Figure 26: Cross section, looking west – Williams / Hemlo mine (Cox, 2017)

It should be noted that three major discontinuities, interfere with the deposit. The first one strikes east-west and proceeds parallel to the orebody. The others are joint sets, one running sub-vertical (striking north-south) and one running sub-horizontal (striking east-west).

Stope dimensions

In regards to the applied dimensions, two different sources are highlighted. The first reference is once again (Hustrulid & Bullock, 2001) which stated the following. Concerning the applied stope dimensions, a width of 20 meters and a height of 25 meters is used. This includes primary as well as secondary stopes. The interval for the main levels is set to be 105 meters. This adds up to a total number of 4 stopes in the vertical or rather 3 sublevels in one panel. There are no specifications for longitudinal stopes.

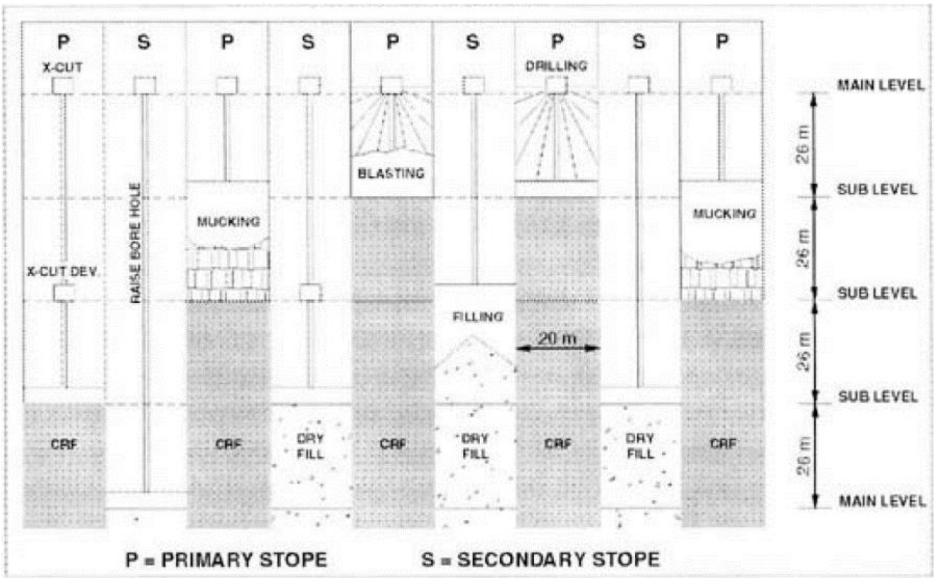


Figure 27: Stoping method – Williams mine (Hustrulid, Bullock, 2001)

The second reference is a technical report from 2017 which stated the following. Longitudinal stopes measure a width of 3 to 8 meters (ore thickness) and length of 15 to 20 meters (in strike). The used height varies between 20 and 25 meters. Transverse stopes measure a width of 15 to 20 meters (in strike), a length of 8 to 30 meters (ore thickness) and a height of 25 to 30 meters. (Cox, 2017)

Comparing these values, it can be stated that the stope dimensions stayed roughly the same over the years.

Rock mass

The drill cores for the laboratory tests, to investigate the intact rock strength, originated from exploration holes. The majority of these tests, indicated a high strength in the footwall as well as the hanging wall. The values regarding classification (Q, RMR) as well as the rock mass strength parameters (m, s) are based on RQD values and underground mapping. All properties regarding the rock mass can be seen in the table below. (Hustrulid, Bullock, 2001)

Location	Rock type	Q (NGI)	RMR (CSIR)	m (H&B 80)	s (H&B 80)	UCS [MPa]
FW	Quartz-muscovite schist	3,9	56,3	2,52	0,0077	84
Ore	Baritic feldspathic	3,4	55,1	2,82	0,0068	100
HW	Muscovite schist	1,3	46,4	1,47	0,0026	85
HW	Banded sediments	3,5	55,4	3,05	0,007	115

Figure 28: Rock mass properties – Williams mine (Hustrulid, Bullock, 2001)

3.3.2 Zinc, Lead, Copper, Silver Deposit – Canada (Brunswick Mine)

The Brunswick mine, which is located in Canada and geologically classified as volcanogenic massive sulfide deposit, was the producer of Lead, Zinc, Copper, Silver and Bismuth. It was known to be the largest underground zinc mine in the world. The mining method that has been used for a long time was cut and fill, but due to efficiency reasons a transition to sublevel open stoping was made.

Deposit characteristics

The deposit measures a strike length of 1200 meters and a width of up to 200 meters. The orebody dips with an angle of 75° and extends to depth of 1200 meters. (Esmaieli, 2010)

Stope dimensions

Due to major mining difficulties, which can be attributed to the bad quality of the hanging wall as well as to the seismic activities, changes regarding stope design were made in 1996. The stope size was reduced from 75.000t to 39.000t. Additionally the primary-secondary stope sequence was replaced with pillarless mining sequences. The reason behind this was to avoid trapped stresses in competent secondary pillars. Furthermore, a report showed that some secondary stopes designed with a width-height ration of 0,43:1 failed. However, secondary stopes in the same area, designed with a ratio of 0,67:1 0,75:1 and 1,2:1 have proven to be stable. (Simser & Andrieux, 2000) (Simser et al., 2002)

Rock mass

While the conducted tests on the sulfides indicated high stiffness and strength, the hanging wall showed much more worse results. All values can be seen in the following table.

Location	E [GPa]	USC [MPa]	Density [g/cm³]
Ore	70	210	4,3
FW	10	50 - 70	2,9
HW	10	30 - 40	2,6

Figure 29: Rock parameters – Brunswick Mine (Hustrulid, Bullock, 2001)

3.3.3 General Studies

In a study created by (Potvin, 2000), the rock mass quality of ore and hanging wall were analyzed in various mines. All 34 Canadian mines involved in this study, used open stoping as their main mining method. The classification that was applied, is known as the modified NGI (Norwegian Geotechnical Institute) Q' system, which will be explained in a later section in more detail.

The results of this study can be observed in the following two figures. What should be noted, the first diagram displays the rock mass quality distribution of the orebody's and the second diagram the distribution of the hanging walls.

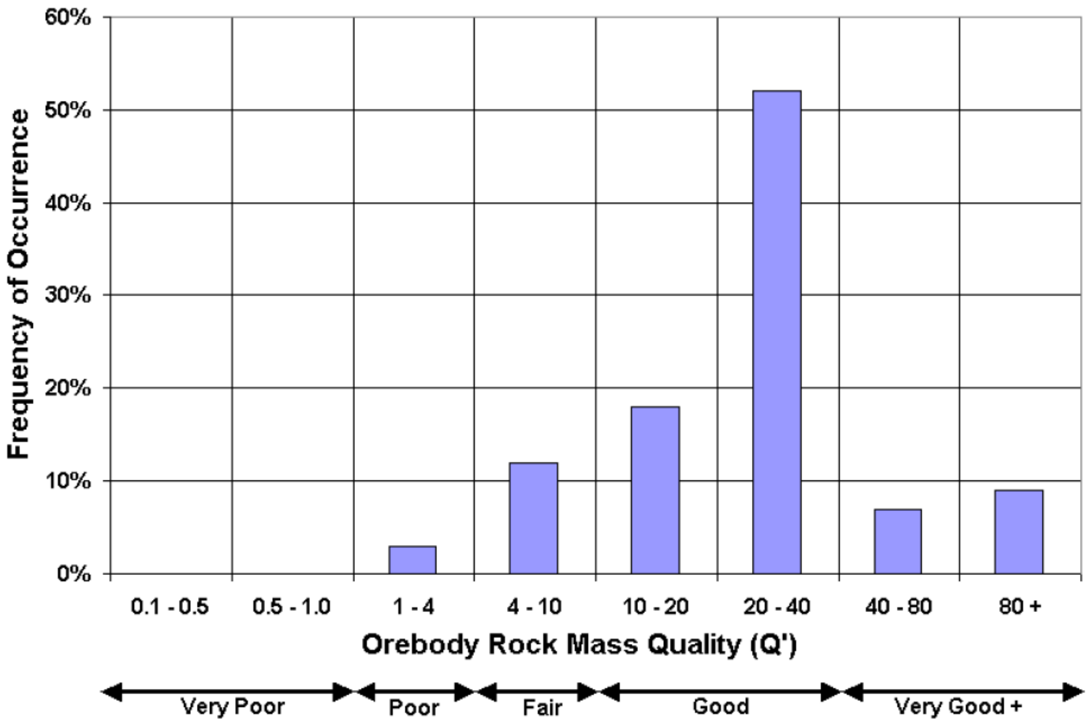


Figure 30: Rock mass quality – Orebody (Potvin, 2000)

Observing this chart, it is apparent that over 85% of the investigated mines show a Q' value greater than 10, which would rate the majority of orebody's "Good" to "Very Good" in terms of rock mass quality. This indicates that in reference to Canadian mines, sublevel stoping is mostly utilized in good to very good conditions and only rarely in poor environment.

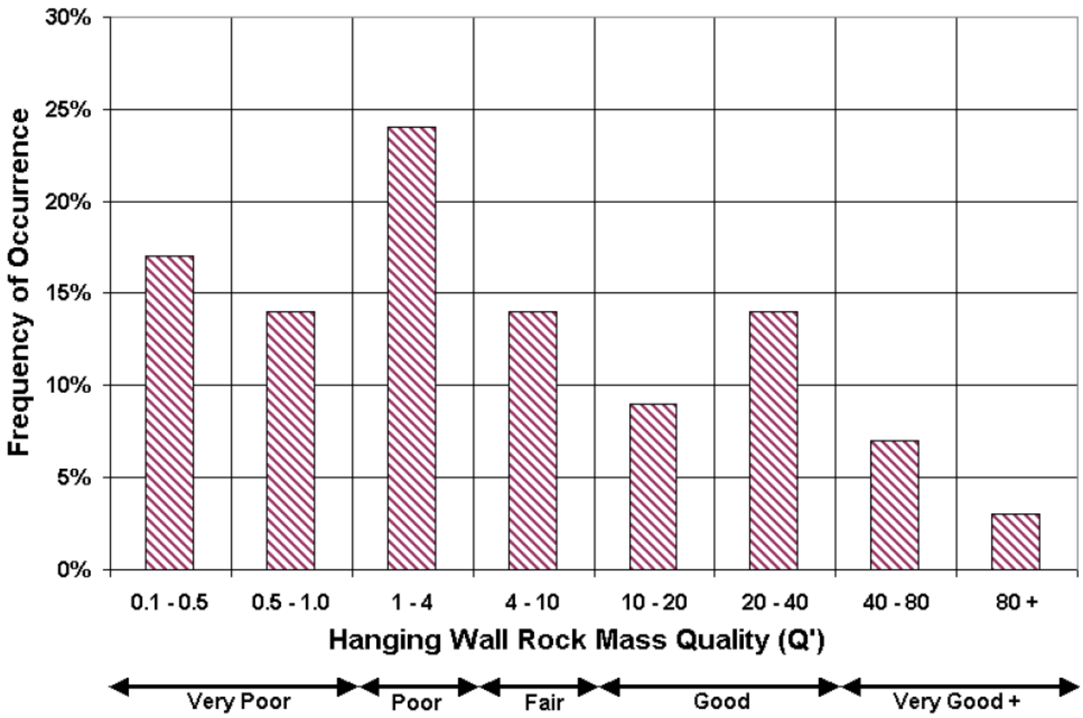


Figure 31: Rock mass quality – Hanging Wall (Potvin, 2000)

Analyzing this second chart, it becomes evident that the calculated rock mass quality for the hanging wall, is on average lower than it is for the orebody. Approximately 70% of the 34 mines indicate a Q'-value smaller than 10, which would classify the majority "Fair" to "Very Poor". (Potvin, 2000)

In order to use the data of both diagrams for a comparison, the Q'-values have to be converted into RMR and GSI values. The equations, which display a correlation between these systems are defined as follows:

Eq. 1) $RMR_b = 7{,}5 * \ln Q' + 48{,}1$ (Russo & Hormazabal, 2019)

Eq. 2) $GSI = RMR_b – 5$ (Ceballos et al., 2014)

It should be noted that the results of these conversions should be interpreted with care, as the general correlation between rock mass classification systems often vary in terms of result accuracy. The following diagram displays the relation, as well as the variance of RMR and the Q' values.

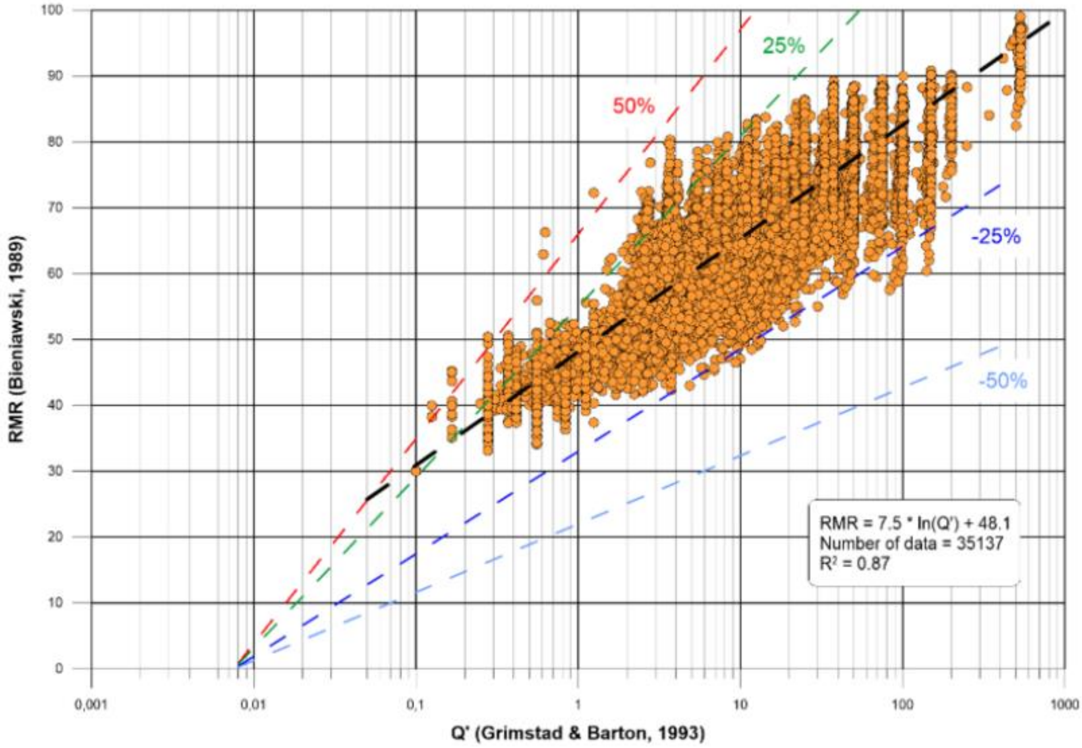


Figure 32: Correlation between RMR and Q' (Russo & Hormazabal, 2019)

As both diagrams (Figure 30 and Figure 31) only indicate value-ranges instead of specific values for each mine, the border values of the Q'-categories will be converted with the above- mentioned formulas. This data will later on be used in chapter 3.4.3.

Q' - Categorization	RMR - Conversion	GSI - Conversion	Ore [%]	HW [%]
0.1 to 1 (Very poor)	31 to 48	26 to 43	0	30
1 to 4 (Poor)	48 to 59	43 to 54	3	24
4 to 10 (Fair)	59 to 65	54 to 60	12	14
10 to 20 (good)	65 to 71	60 to 66	18	9
20 to 40 (good)	71 to 76	66 to 71	52	13
> 40 (very good)	> 76	> 71	15	10

Figure 33: Converted RMR and GSI value ranges

The next diagram, which also originated from this study, confronts the transverse mining width, with the rock mass quality of the orebody. It should be noted that this "width" is related to the thickness of the orebody and therefore to the transverse stope length.

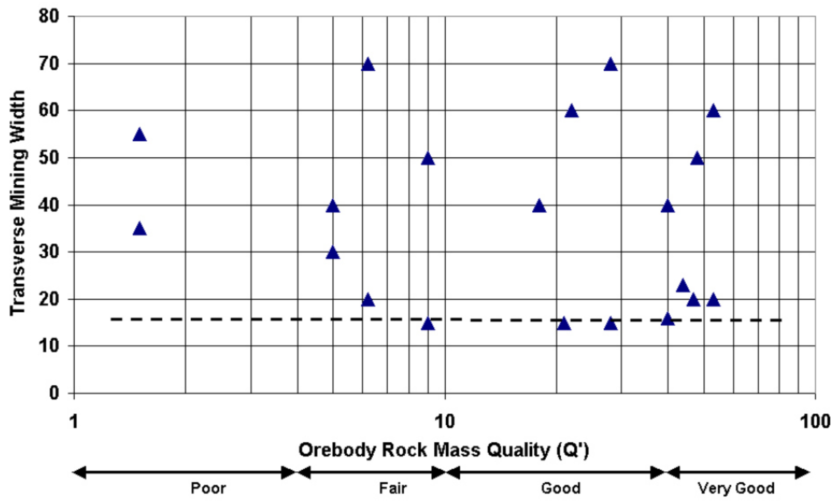


Figure 34: Transverse stope width of Canadian mines (Potvin, 2000)

What can be observed is that transverse mining is not applied, if the mining width is less than 15 meters. However, the results also show that in spite of a "fair" or "poor" rock mass quality, transverse mining is utilized up to a width of 70 meters.

The next chart (also from the same study) shows data related to longitudinal stoping.

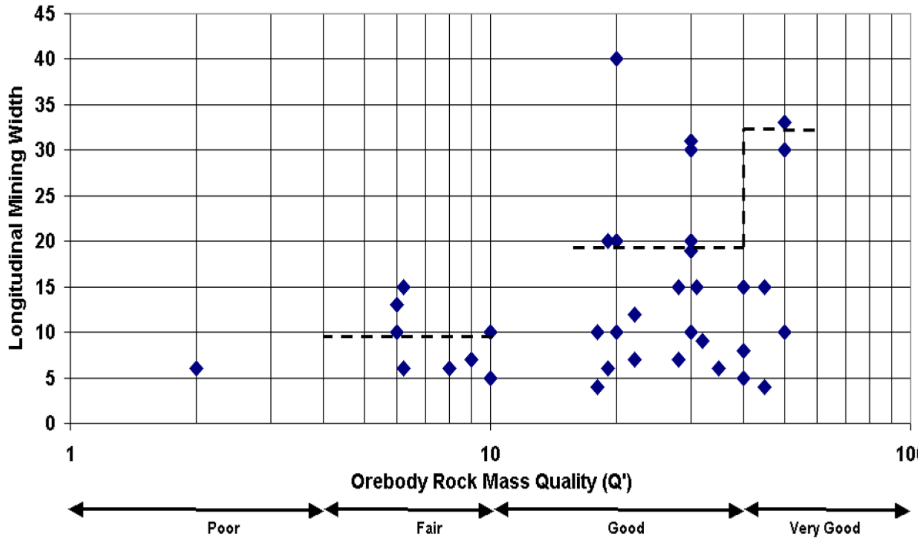


Figure 35: Longitudinal stope width of Canadian mines (Potvin, 2000)

Analyzing Figure 35 it is notable that 75% of all longitudinal stoping operations, showing a Q' value smaller than 10, apply a stope width of 10 meters or less. Furthermore 100% of the same rock mass category use stope width of 15 meters or less. If the Q' value is determined to be greater than 10, widths up to a maximum of 40 meters are applied. In many cases however, mines report instabilities when the width exceeds 15 meters, despite of a better rock mass quality. (Potvin, 2000)

3.4 Comparison and analysis of the collected data

In this chapter, the obtained data of the previous sections will be summarized and illustrated through diagrams. Using this approach, it is possible to compare various aspects of different sublevel stoping mines. The overall purpose is to present a broad insight into the field of application, while at the same time highlight differences, commonalities and possibly trends concerning the implementation of this method.

The first parameters which will be analyzed are related to the deposit geometry of the individual operations. Following on from this geometrical design parameters concerning stope- dimensioning will be compared. Subsequently, the collected data regarding rock mass quality and other rock mass related parameters (UCS, E-module) is presented. Last but not least a conclusion concerning the researched data will be drawn.

3.4.1 Comparison - Deposit geometry

The geometrical properties of all researched orebodies will be displayed and discussed in this section. The first parameter to be addressed in this process, concerns the dipping angle.

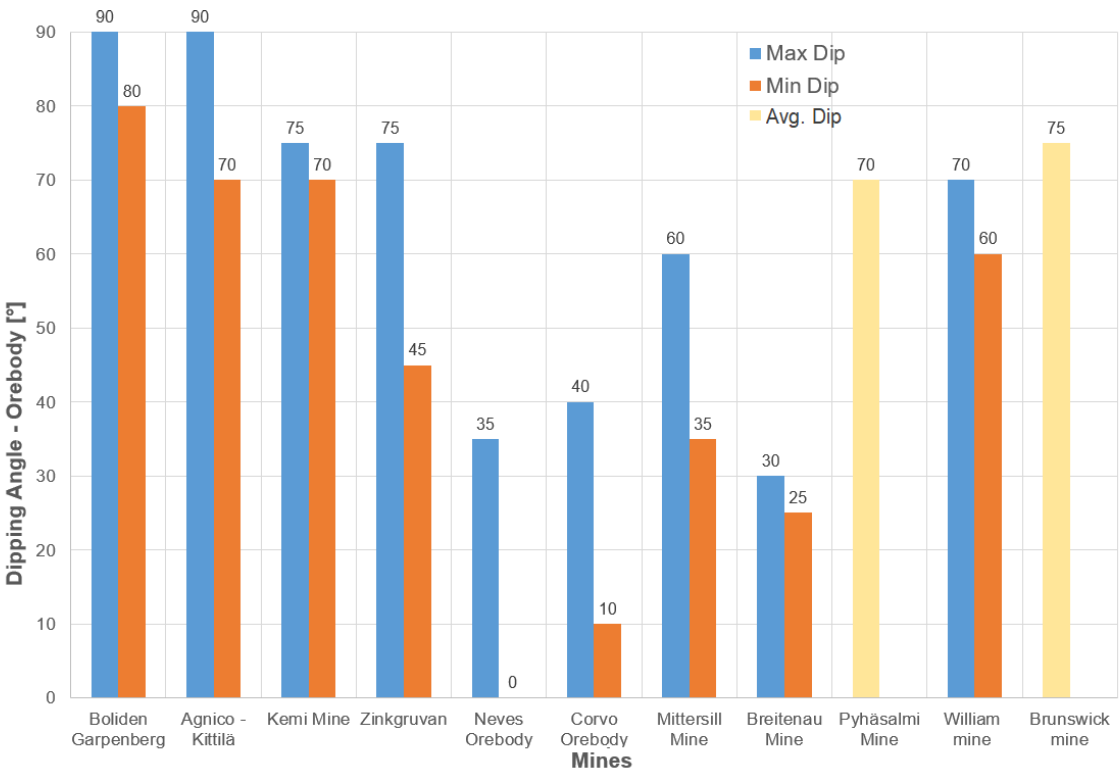


Figure 36: Dipping angle of researched mines

Observing figure 36, it becomes apparent that 8 out of 11 orebodies in which stoping is applied, feature a dipping angle of 60° or larger. This would categorize the majority of orebody's as steep or very steep. The outliers in this regard are the Breitenau mine as well as the Neves-Corvo mine. One reason why stoping nevertheless was a valid option for both deposits, is the large thickness of the orebodies. According to (Newall, 2017), bench-and-fill stoping can be applied profitable at a shallow angle, if the vertical thickness of the Neves or Corvo orebody exceeds 20 meters. What is also evident from this chart, the dipping angle is seldomly constant and can vary to a large scale. Some problems which can occur in that context, will be outlined in chapter 4.3.6, where the stoping procedure is discussed in more detail.

Another geometrical parameter, which is quite interesting for the area of application, is the thickness of the individual orebodies.

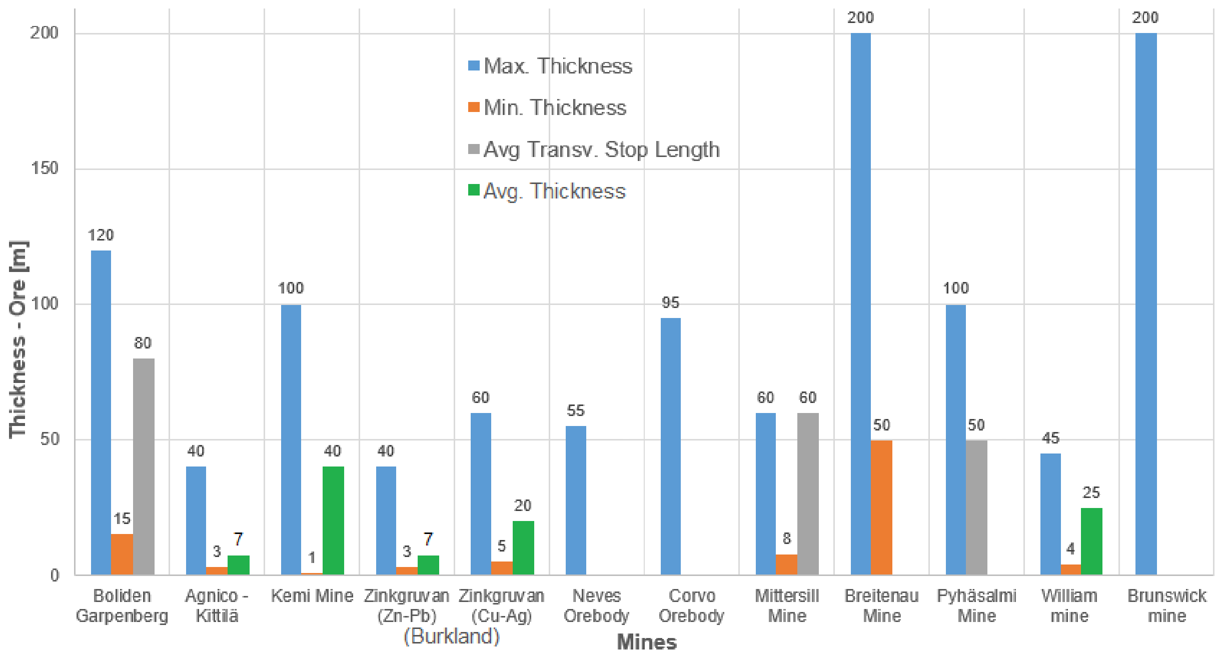


Figure 37: Thickness of researched deposits

Examining this chart, it appears that the general variation between minimum and maximum thickness, tends to be very high. For this reason, an average value would be quite useful. Unfortunately, not every mine provided information concerning the average thickness of the orebodies. To nevertheless get a rough indication, the average transverse stope length (colored in gray), is used for approximation purposes. For a more accurate estimation of the average thickness, the dipping angle of the orebodies has to be taken into consideration. However, since most of the deposits are rather steep, this will be neglected.

In regards to the Breitenau mine it is important to note that the applied stopes are oriented to the strike of the orebody. Consequently, there is no correlation with the thickness.

To summarize the area of application for the sublevel stoping method (related to the deposits geometry), a brief discussion concerning the presented data follows.

Discussion

According to the data presented in figure 37, the thickness in which transverse sublevel stoping is utilized varies between 8 (Zinkgruvan Zn-Pb) and 200 (Brunswick mine) meters. Further information regarding the applicability of transverse stoping, can be derived from figure 34. This diagram (which comprises information of 20 Canadian mines) indicates that transverse stoping is applied in orebodies, which are characterized by a thickness of 15 to 70 meters. (Potvin et al., 2000)

What is also notable, three mines displayed in figure 37, apply longitudinal stoping in case the deposit narrows down to a specific width. A critical value in this regard was determined at around 11 meters for the Zinkgruvan mine and at approximately 7 meters for the Kittilä and Williams mine. The application limit of longitudinal stoping (in all three cases) was specified at around 3 meters. Analyzing the longitudinal stoping operations from the Canadian study (figure 35), it is apparent that the limits likewise range from 15 to approximately 4 meters (for the large majority). A critical threshold for sublevel stoping, can therefore be considered at approximately 3 to 4 meters, in most cases.

Another interesting parameter is the average thickness of the orebodies. The investigated values indicate that an average thickness of 7 meters (Kittilä and Zinkgruvan Zn-Pb deposit) is quite sufficient to apply sublevel stoping. However, in both cases it is necessary to alternate between longitudinal and transversal sections regularly.

3.4.2 Comparison - Stope dimensions

To provide an overview concerning the applied dimensions of the individual stope designs, all gathered information of chapter 3.2 and 3.3 is summarized in this section. The first diagram will highlight the geometrical stope parameters, that are chosen to be used in the respective mines.

General stope dimensions

To avoid confusion concerning dimensional directions, the width of all referred transverse stopes, is directed to the strike of each orebody. What should be noted, the Breitenau mine is the only operation researched, which utilizes multiple longitudinal stopes adjacent to each other. For this reason, this mining operation will be included in this figure.

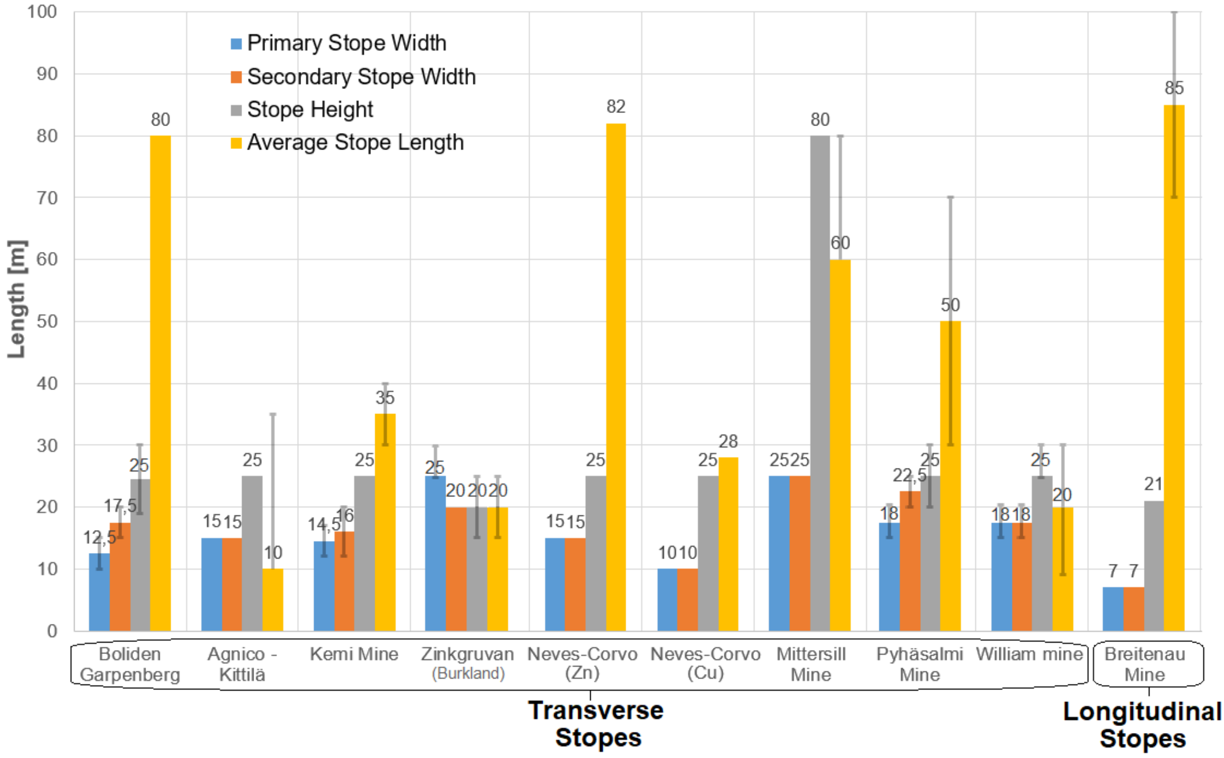


Figure 38: Applied Stope dimensions

Analyzing this chart, it is apparent that the applied stope widths, range from a minimum of 7 to a maximum of 30 meters. A value area that stand out particularly in this regard, lies between 15 and 20 meters, as 7 out of 10 mines utilize dimensions within that range. Another observation concerning the width indicates that (in 9 out of 10 cases), the dimensions used for secondary stopes, are either equal or larger to the dimensions used for primary stopes. In general, there could be several reasons for this, including stability requirements or economic purposes. A more detailed elaboration on these subjects is provided in chapter 4, where the individual elements are discussed.

Returning to figure 38, it is apparent that in regards to the stope height a value of ~25 meters is commonly applied. Rather striking is the height (of 80 meters) utilized in the Mittersill mine. This however, can be attributed to the used extraction method. In comparison the other mines, multiple lifts (up to 5 sublevels) are used simultaneously to extract the ore. This can cause the stope to reach a height of 80 meters at certain sections. (Gaul,2008)

The next diagram highlights the applied longitudinal stope dimensions of the investigated mines. To avoid confusion concerning directions, the blue colored stope width is related to the thickness of the orebody.

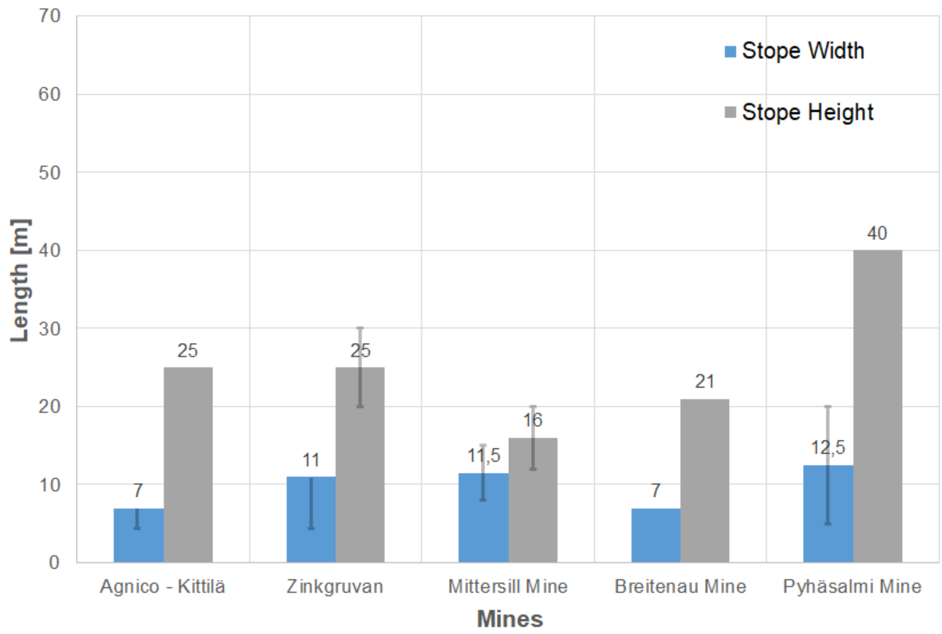


Figure 39: Longitudinal Stope dimensions

Examining these values, it can be concluded that the average width lies around 10 meters, while the overall maximum caps at 20 meters. The heights of the stopes differ on a larger scale. However, an average value seems to be around 25 meters, which resembles a typical height of transverse stopes. Comparing these width values to the data obtain from the Canadian study (Figure 35), it can be observed that 75% (27 out of 36 mines) apply a width smaller than 15 meters. Furthermore, if the Q'-value was determined to be smaller than 10, 75% of the longitudinal stoping operations applied (similarly to the European operations), a stope width of 10 meters or less.

Width to Height ratio

The next parameter which will be highlighted is the width to height ratio used for secondary stopes. This factor is particularly of interest, as many empirical design methods utilize this parameter (in combination with other factors) for stability estimations. More detailed information on the W/H-ration can be found in chapter 4.3.2. The following diagram illustrates the applied values of the researched operations.

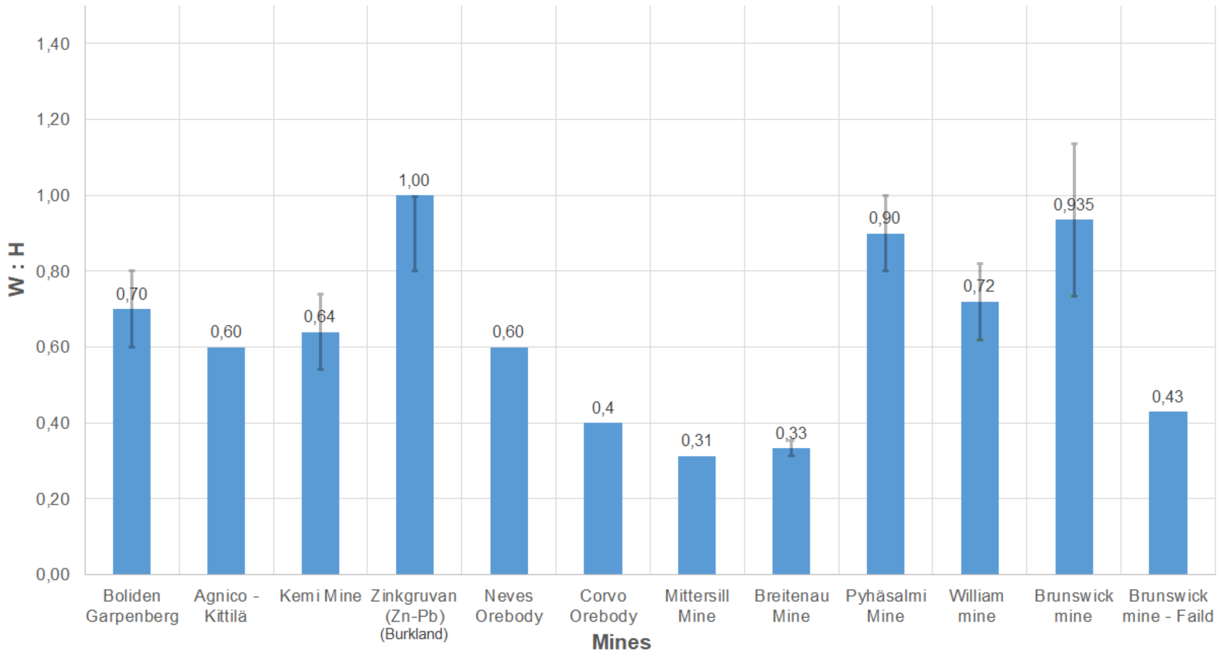


Figure 40: Width to Height ratio - Secondary stopes / pillars

What can be deduced from this chart, around 75% of all displayed mines apply a W/H-ratio of 0,6 or higher for their secondary stopes / pillars. Furthermore, in 5 of these cases the used ratio can be limited to a span of 0.6 to 0.8. The outliers, featuring values of 0,4 or less, are both Austrian mines as well as the Corvo mine. The Brunswick mine tested various W/H-ratios, in which the higher values turned out to be much more stable, regarding their geological environment. (Joughin, 2002)

Stope size

Last but not least the stope sizes (cross-section and volume) will be derived from the researched data. The intention is to identify commonly applied values as well as potential difference between European and Canadian stopes sizes. It should be noted that the calculated values of the cross section (H x W), are based on the displayed dimensions of the secondary stopes, illustrated in figure 38.

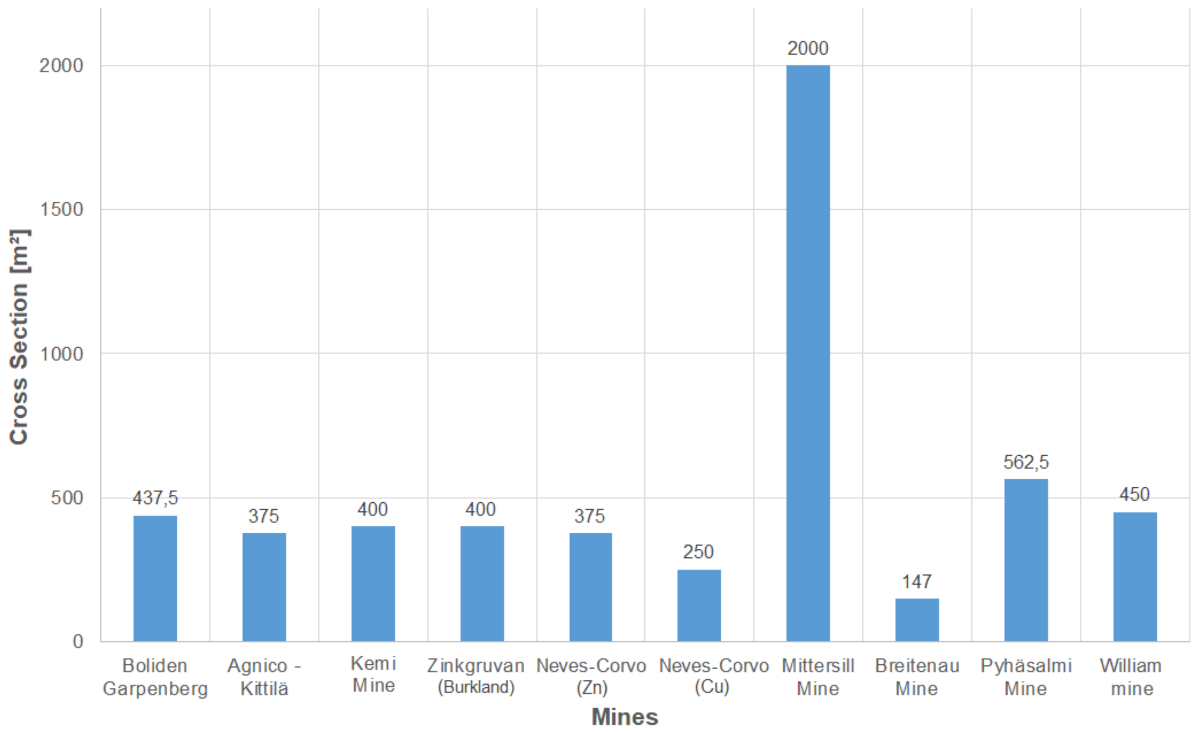


Figure 41: Stope - Cross Section

What becomes apparent is that cross-sections between 375m² and 450m² are applied quite frequently for secondary stopes / pillars. To be more precise, 6 out of 10 mines utilized cross sections within that range. In order to further obtain a value for the stope volume, the cross-section as well as the average stope length will be used. The following diagram presents the resulted data.

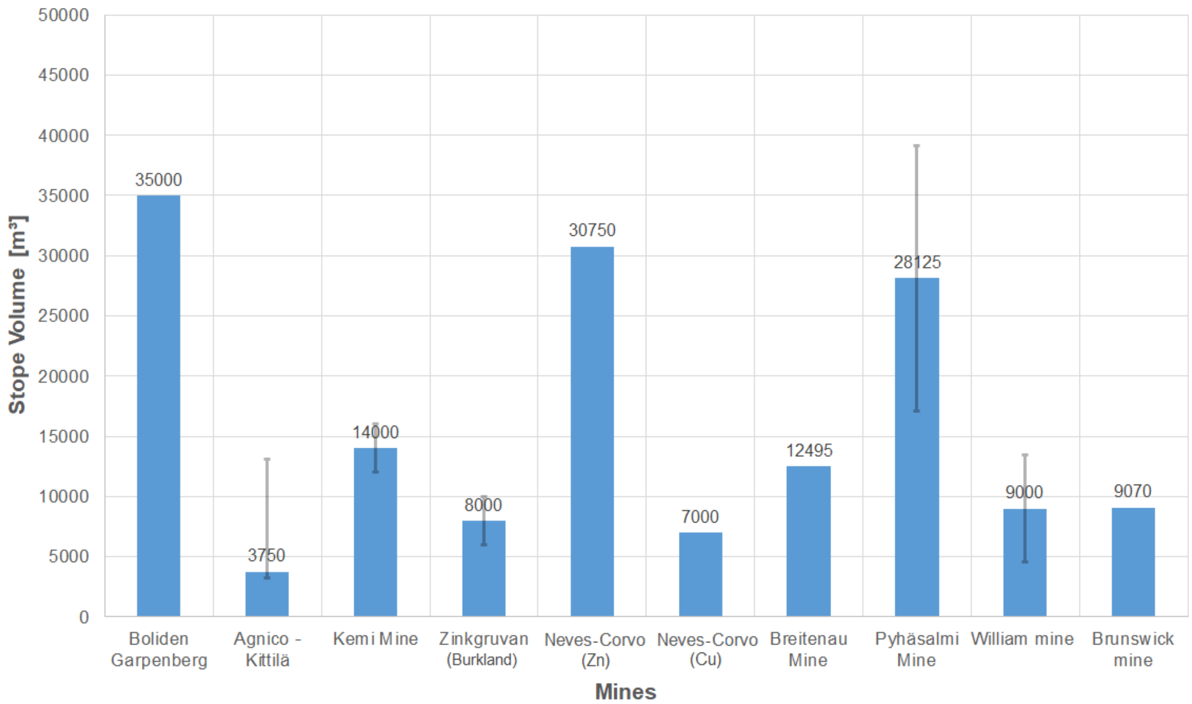


Figure 42: Stope – Volume

A typical stope size which is used in Canada nowadays, lies within the range of 20.000 to 100.000 tons. (Potvin, 2000) This would be equivalent to a stope volume ranging from 5.000 to 35.000 m³, assuming an ore density between 3.0 and 4.0 [t/m³]. Analyzing the data in the diagram (Figure 42), it is apparent that the European as well as both Canadian mines fit into that range.

To get an overall reference value the average volume of the 8 European mines will be calculated. It should be noted that the Mittersill mine is excluded from this calculation, as the deviation is too extensive, due to the modified mining method.

Average stope volume of the 8 European stopes (Figure 42): ~17.400 m³

This reference value indicates that on a general basis, European as well as Canadian stoping mines apply similar sized stopes. However, it is important to note that this generalized value is only based on the data of 8 European stoping operations. Furthermore, there is also a time difference between the mines analyzed by (Potvin, 2000) and this research. A general trend however indicates, that since the 80's and 90's, stopes utilized in Canadian mines tend to become smaller. One of the reasons leading to this transition is the fact that uncontrolled failure, occurred much more often in high stopes than in shorter ones. (Potvin, 2000) An example for such a redesign can be seen in the Brunswick mine, which reduced their stope size from 75.000 to 39.000 tons. This would indicate a volume of 17.000 m³ or 9.000 m³, considering that the ore density is 4,3 t/m³. Further examples for the reduction / adaption of the stope size can be seen in the Zinkgruvan as well as in the Pyhäsalmi mine.

3.4.3 Comparison - Rock mass parameters

In this section the quality of the rock mass, as well as specific rock criterions will be examined. The first parameter to be highlighted in this regard is the GSI factor. Subsequently the researched values of the UCS and the E-Module will be presented.

GSI - Values

All investigated values concerning ore and hanging wall, will be displayed in the following diagram.

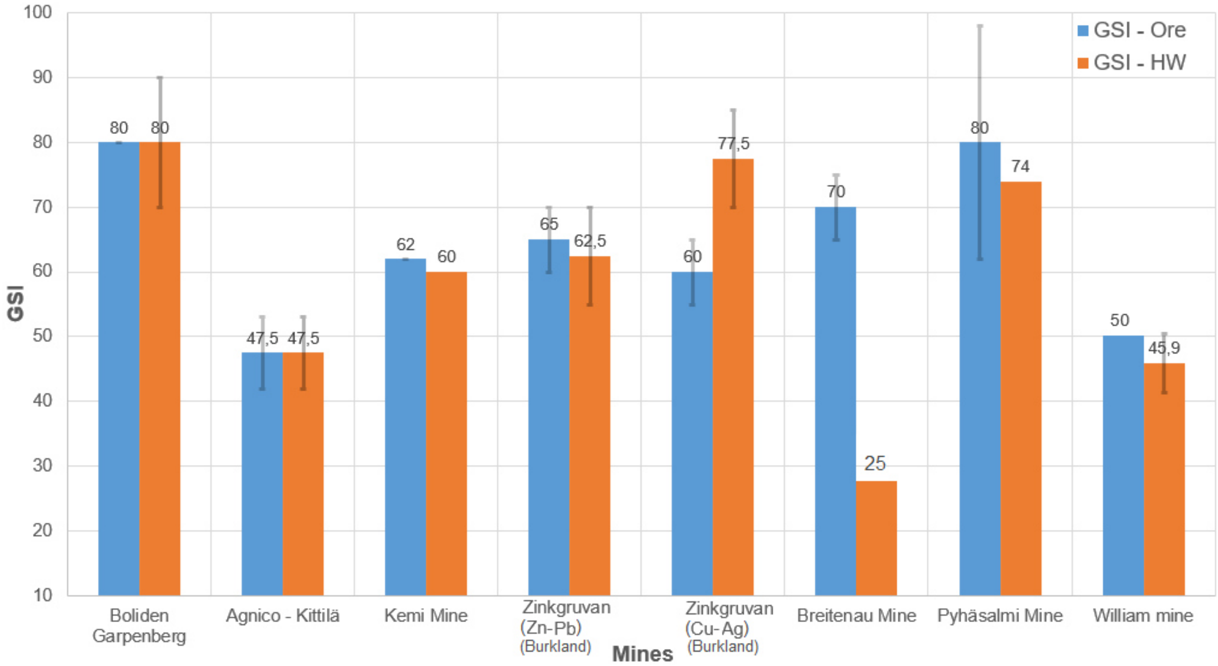


Figure 43: GSI of Ore and HW - Researched mines

Examining this chart, it can be observed that the overall GSI difference between ore and hanging wall, seems to be quite low. The most significant variation can be registered within the Zinkgruvan Copper stockwork, which is directly located in the hanging wall of the Zinc-Lead mineralization of the Burkland orebody. This stockwork zone is cut off laterally by the Knalla fault and visualized in 3.1.5. More specific information of the geological circumstances on this area can be found in the technical report of (Daffern, 2017). Another oscillation can be identified in the ore of the Pyhäsalmi mine, as the min / max values indicate rather high peaks. The reason for this is a weak talc-schist layer which occurs in the upper part of the orebody.

Observing further it is evident that 6 out of 7 European deposits indicate a value greater than 60. This would categorize the majority of analyzed rock masses to “fair” and “good” in regards to the GSI system. The overall mean values of these European deposits add up to 66 (ore) and 67 (HW). These average values will later be used as a reference. The two lowest and somewhat similar values (around 50) appeared in the European Kittilä and the Canadian Williams mine.

The next diagram to be analyzed contains rock mass data of 34 Canadian mines. All displayed GSI values are converted Q’ values from (Potvin, 2000) and should be interpreted with care. The original data as well as the conversion process can be found in section 3.3.3 (Figure 33). It should be noted that the Y-axis indicates the calculated GSI value and the X-axis the percentage as well as the rounded number of mines, featuring this value.

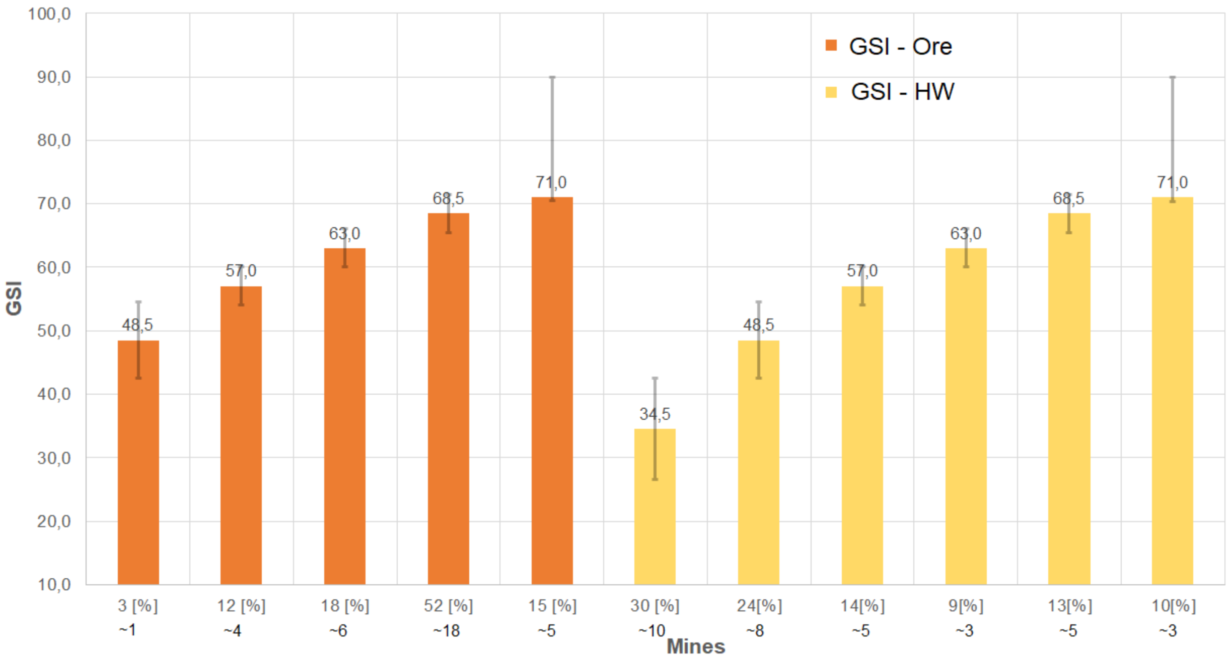


Figure 44: GSI distribution of Ore and HW from 34 Canadian mines

What becomes apparent is that similarly to the European deposits, approximately 80% of the analyzed mines, indicate an ore-GSI of 60 or higher. Looking at the lowest ore values, it is evident that the area of 45 to 55 is likewise similar to the European deposits. One noticeable difference however, are the GSI values measured in the hanging walls, as a clearly larger percentage indicates a “poor” to “very poor” rock mass quality.

Since one objective of this paper is to describe and discuss the area of application concerning sublevel stoping, an average GSI value will now be calculated. What must be noted, this value should not be considered as an indicator to determine whether sublevel stoping can or should be applied, but rather as a statistical value that more accurately describes the area of application.

Using the displayed GSI value in combination with the mine number (Figure 44), an overall mean value regarding ore and hanging wall, can be calculated. In that context, the average GSI value of the ore resulted in 66 while the hanging wall resulted in 52. Comparing these results to the values of European mines, it is apparent that the average Ore-GSI obtained from Canadian mines is likewise 66, while the value of the hanging wall seems to be clearly lower (HW Canada: 52, HW Europe: 67).

UCS - Values

The next figure provides an overview of the investigated UCS values. As with the previous topics, the data of ore and hanging wall will be presented.

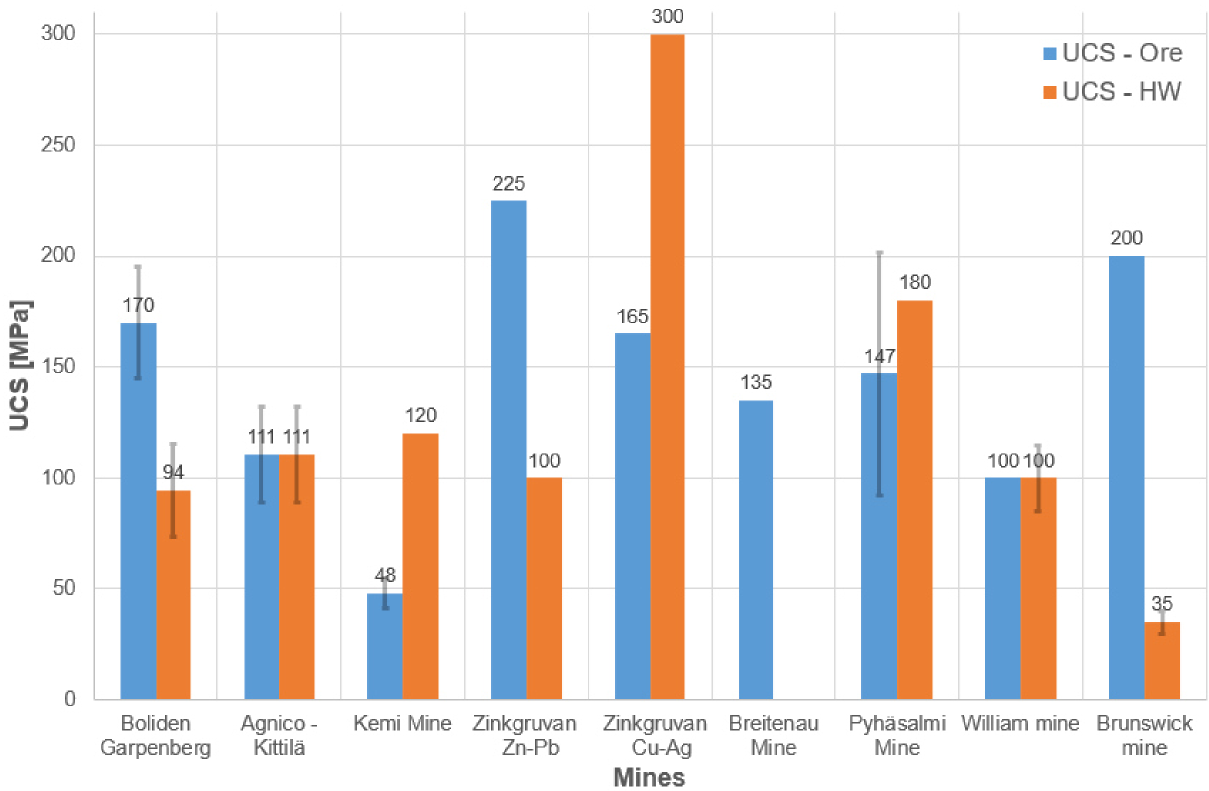


Figure 45: UCS of Ore and HW

Since the uniaxial compressive strength is not included in the GSI system, this data adds a further perspective on the rock mass environment of stoping operations. Examining this chart, it appears that the general difference between ore and hanging wall seems to be much more significant, in comparison to the GSI values. The largest difference in this regard could be observed in the Canadian Brunswick mine, as the ore is characterized by one of the highest values, while the hanging wall indicate the lowest measured value. The minimum UCS concerning the ore, originated from the Kemi mine. The geological reason for this is the soft talc-carbonate based material, which is encountered in various areas of the Kemi deposit. Analyzing the data in general, 8 out of 9 deposits indicate an ore and hanging wall UCS of 100 MPa or higher. Furthermore, six of these deposits can be further be specified to an ore UCS of 135 MPa or higher. The hanging walls measure lower values on average, as only 2 deposits exceed 135 MPa.

E-Module

The last parameter which will be highlighted is the E-Module of ore and hanging wall. The following diagram illustrates the researched data.

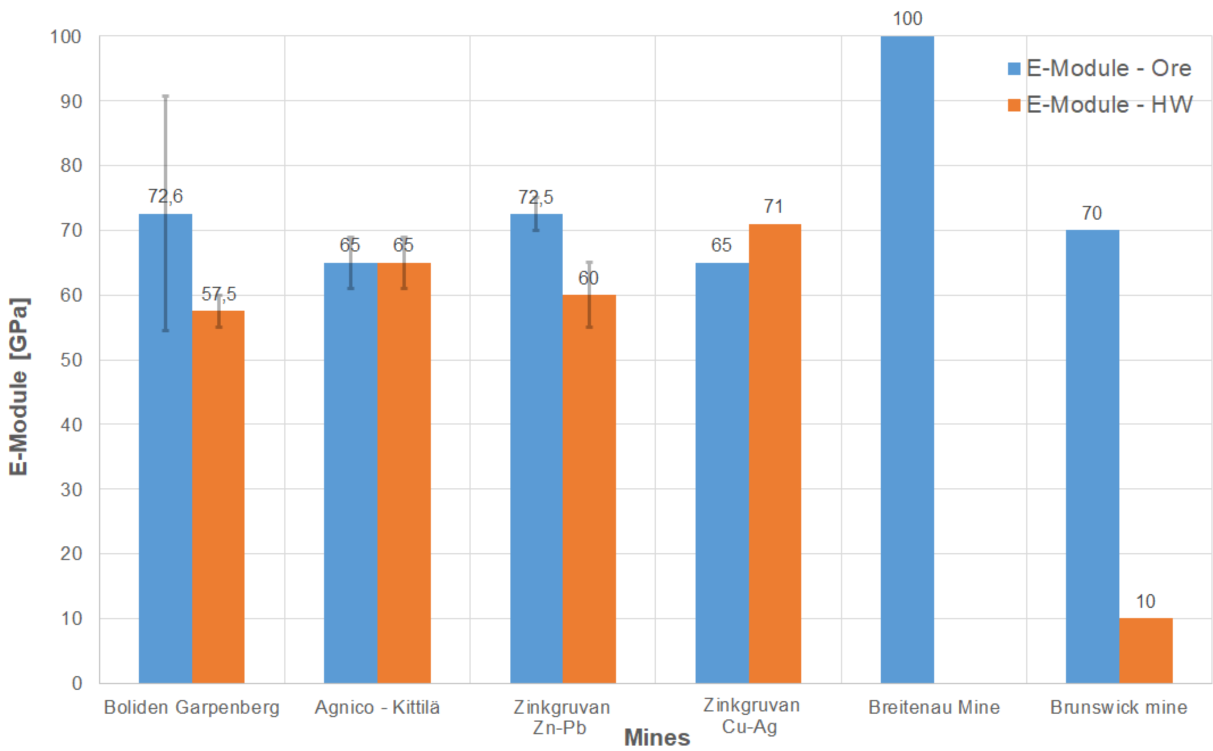


Figure 46: E-Module of Ore and HW

It can be observed that in contrast to the UCS values, the E-Module indicates a much smaller deviation, between ore and hanging wall. The only exception is the Brunswick mine which demonstrates an equally large difference in this chart. Particularly noticeable is the low value measured in the hanging wall. The reasons for this are the chloritic and sericitic metasediments, which are much lighter, softer and weaker in the hanging wall than in the older footwall. These circumstance’s led to major stability problems during excavations. Unfortunately, this chart incorporates less data than the other figures, which limits the statistical significance of possible trends. However, an overall tendency indicates that values between 55 and 75 GPa are commonly seen in deposits where sublevel stoping is utilized.

3.5 Conclusion – Field of Application

The aim of this comparison was to provide a detailed overview concerning the field of application by examining a wide variety of parameters. Through this approach it was possible to identify commonalties and trends in various topics of sublevel stoping mines.

The first point that was addressed in this regard, concerns the geometrical structure of the deposits. In total, 12 orebodies, 10 of which are located in Europe and 2 that are located in Canada, were analyzed. In that context, the following points could be concluded:

- The average orebody thickness, in which sublevel stoping is commonly utilized, lies between 7 and 80 meters.
- The overall application limit concerning the orebody thickness, can be considered between 3 and 4 meters, in most cases.
- The main area of application of longitudinal stoping was determined between 15 and 4 meters.
- Two thirds of investigated orebodies, indicate a maximum dipping angle greater than 70° and an average dipping angle of greater than 60°.

The second topic that was highlighted concerns the geometrical aspect of utilized stopes. Through the comparison of 9 individual European stope designs and 2 individual Canadian designs, the following point could be concluded:

- 70% of all investigated mines applied a transverse stope width (strike direction) of 15 to 20 meters for secondary stopes as well as a stope height of 25 meters for primary and secondary stopes.
- 75% of all investigated mines applied a W/H-ratio of 0,6 or higher for secondary stopes / pillars.
- 6 out of 10 secondary stope designs utilize cross-sections (WxH) between 375m² and 450m².

Last but not least the rock mechanical aspect of this comparison will be summarized. In that context, the quality of the rock mass, UCS and Young's modulus of 7 European and 2 Canadian mines were analyzed. The overall investigation resulted in the following:

- 6 out of 7 European sublevel stoping operations indicate an ore GSI of 60 or higher. The same percentage (~80%) could also be determined in the 34 Canadian mines, researched by (Potvin et al., 2000).
- The lowest rock mass quality value at which sublevel stoping is applied, lies between 45 and 55 (GSI of ore).
- 8 out of 9 deposits indicate an ore and hanging wall UCS of 100 MPa or higher.
- 6 out of 6 deposits measure an E-Module greater than 65 GPa, in regards to the ore.
- On average, the hanging wall is characterized by worse rock mass conditions than the ore. This was observed throughout all three parameters.

To complete this research, one final thought in regards to the available data itself. It is very unfortunate that technical information related to the presented topics is published very rarely and if so, usually with a lack of detail. In order to increase the significance of these application trends, much more data is required.

4 State-of-the-Art Research

In this chapter the overall layout as well as the design aspects, of a sublevel stoping operation, will be illustrated and discussed. To do so, this chapter is divided into four subparts.

The first section deals with the identification (purpose and visualization) of all elements that are essential for the structure and the extraction process of a sublevel stoping operation. Following on from this, three popular rock mass classification systems will be outlined briefly, as they form a central part in many design methods which are discussed later on. In the third part of this chapter the main elements of this mining method are analyzed in more detail. Here the design methods as well as important design parameters and aspects will be the center of discussion. Last but not least, a final conclusion is drawn, summarizing the most essential aspects.

4.1 Identification of Elements

To get an overview concerning the overall mine structure, a simplified example of a sublevel stoping operation, containing all elements, is illustrated first. Following on from this, a definition involving the purpose as well as a visualization of all individual elements, will be made.

The order in which the elements are presented, is as followed:

- Stope
- Pillar (Secondary Stope)
- Pillars located in the footwall
- Barrier Pillar
- Sill Pillar
- Mining Panel
- Stope Sequence
- Secondary Development
- Primary Development
- Backfill

4.1.1 Sublevel stoping layout - Overview

Using the outlines of the orebody that was illustrated in the previous section (Chapter 2.2.3), an example layout of a sublevel stoping operation can be visualized. The following figure shows a possible structure for the extraction of a steeply dipping orebody.

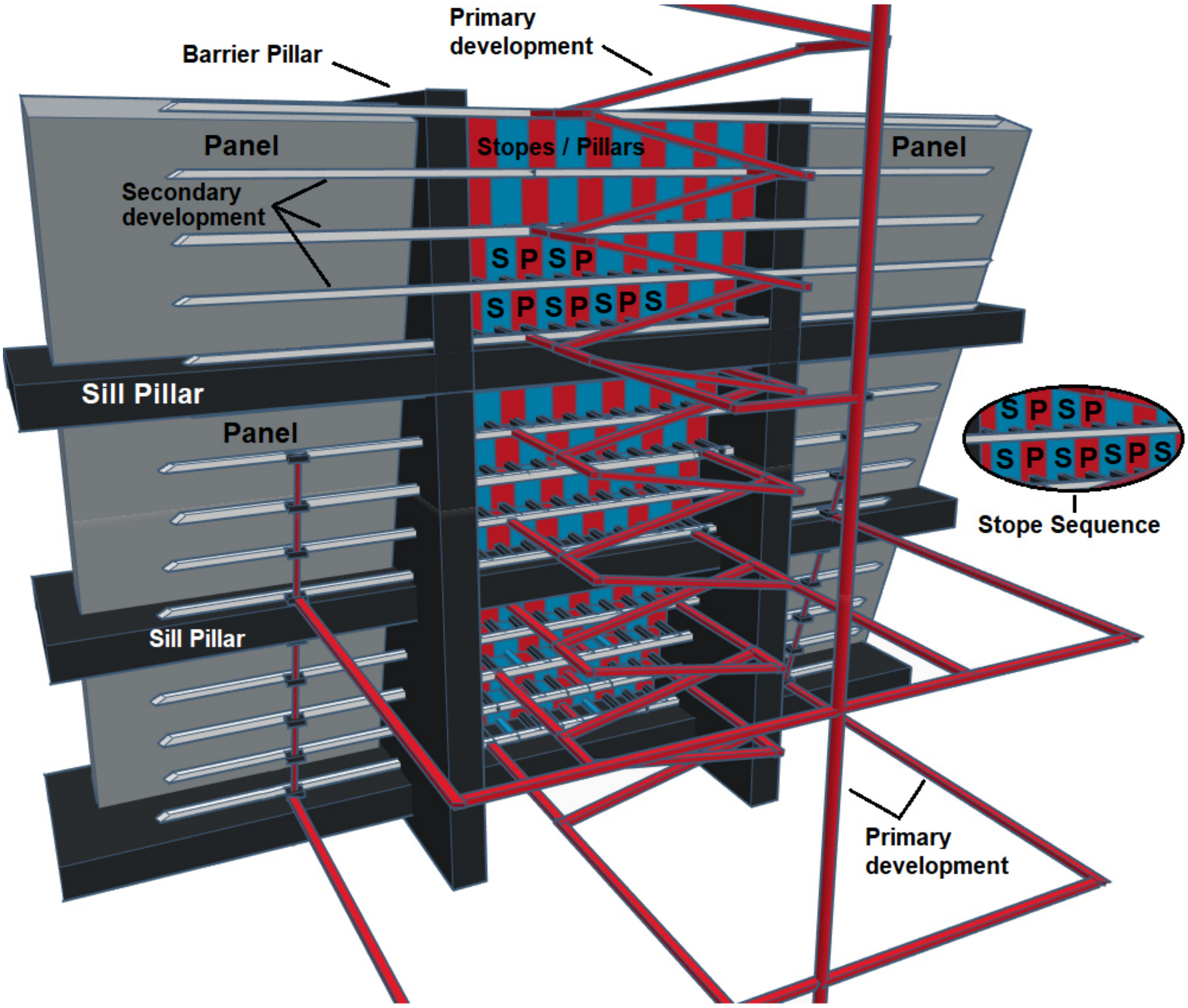


Figure 47: Sublevel stoping – General layout

4.1.2 Stope

The first element to be highlighted, which is probably the most significant concerning this mining method, is the stope itself.

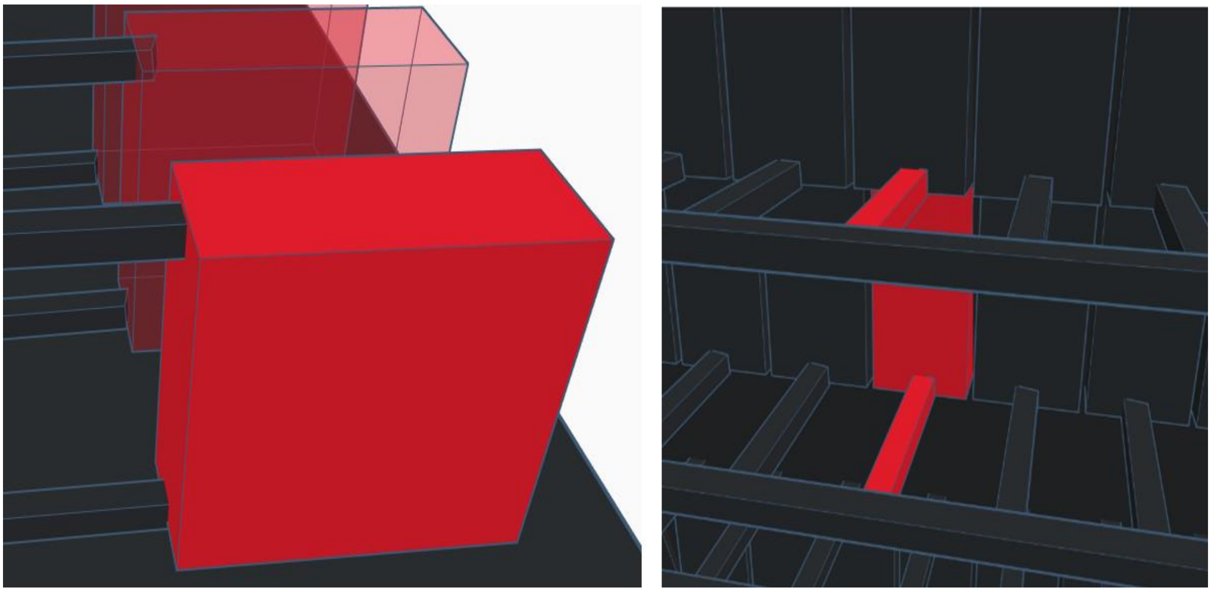


Figure 48: Element - Stope

Definition and purpose

A stope is a large underground excavation zone, typically rectangular in shape, with defined geometrical dimensions. Or to be more precise, it is a specific volumetric part of the orebody, which is going to be excavated, leaving a large cavity for a certain amount of time, until it is backfilled eventually.

The main purpose of a stope is to extract a defined part of the deposit, in an efficient and safe way. Thereby several factors like quality control in regards to dilution, stability in regards to safety and stope size in regards to economics, play an important role. A major dependency all these factors have in common is their relation to rock mechanics. These complex interrelations are the reason why there are many different possibilities to design a stope. In the next chapter, an overview to some of the main design methods is presented.

4.1.3 Stope-Pillar (Secondary Stope / Pillar)

The next subpart to be highlighted is the stope-pillar element.

Figure 49: Element – Stope-Pillar

Definition and purpose

Stope-Pillars or Secondary stopes belong to the most complex elements in a sublevel stoping operation, as they are not only essential support elements, but also future excavation zones. Their rock mass usually consists of ore, which has been left behind intentionally, to temporarily support the mine structure. The overall purpose of this element is to support and stabilize the surrounding rock mass, until it is extracted in a specific stoping sequence. These temporary pillars are therefore also labeled as secondary or tertiary stopes.

4.1.4 Barrier Pillars

This next element to be defined concerns the regional safety and stability of the mine.

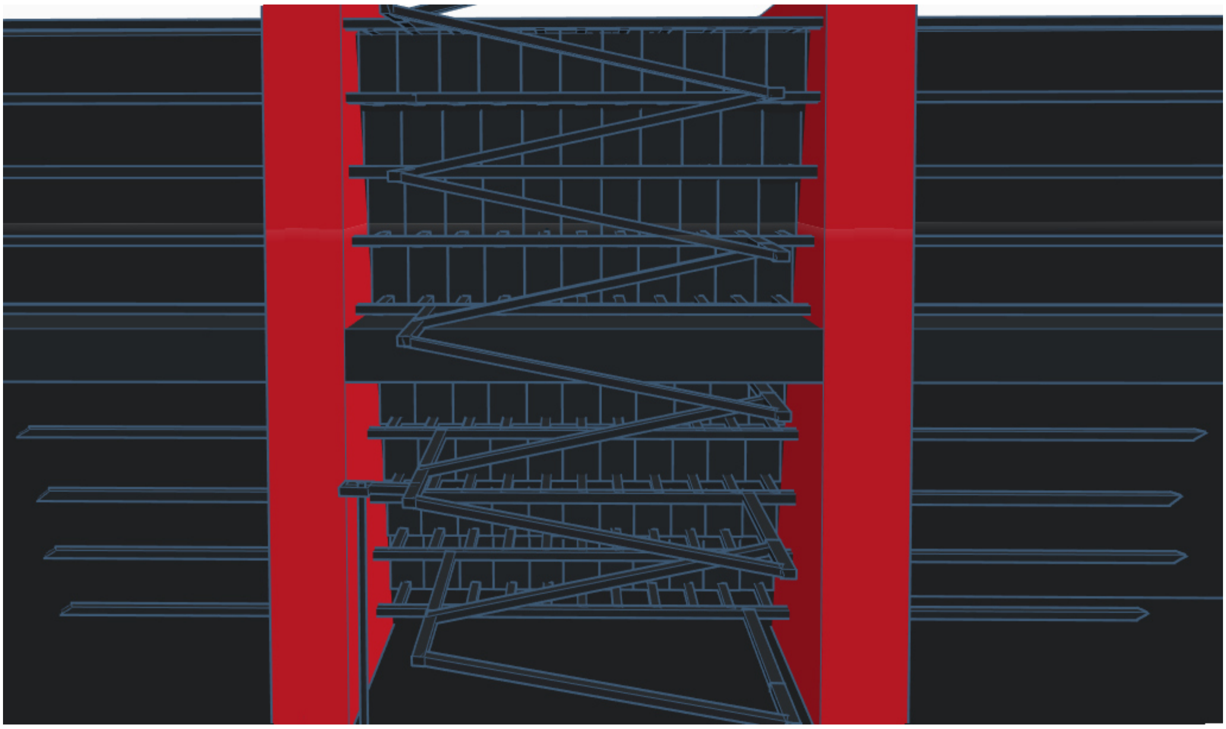


Figure 50: Element – Barrier Pillar

Definition and purpose

Barrier pillars are large blocks of omitted material, which separates big mining areas from one another. Their main function is to provide regional safety and stability by prevent chain reactions, caused by accidental or natural circumstances, from spreading further. These events include water inrushes, gas and fire propagation as well as multi pillar collapses. Although the main function is compartmentalization, there are also other important effects like the reduction of surface subsidence and the control of tensile zones. Barrier pillars have to be designed in such a manner that they are theoretically considered indestructible, so that each area can be treated as an isolated single mining domain. (Brady & Brown, 2005) An important aspect in designing these pillars is the width to height ratio, but more on this topic in a later section.

4.1.5 Sill Pillar

As well as barrier pillars, sill pillars are likewise important concerning the structural stability and the safety of a sublevel stoping operations. The following figure illustrates the application of these horizontal support elements.

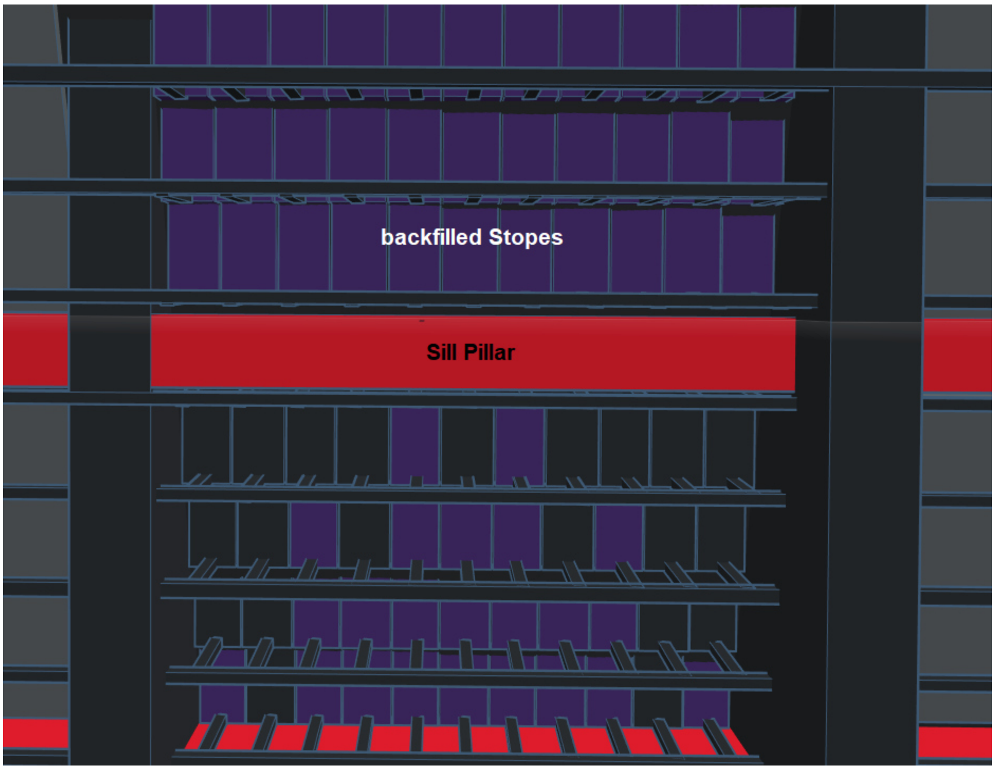


Figure 51: Element – Sill Pillar

Definition and purpose

Sill pillars are ore parts with a defined thickness, which are left behind intentionally to separate two mining sections (panels) vertically. The overall formation process of sill pillars depends on the applied mining-sequence. As the stope creation within a panel advances up-dip, the stope roof is progressively turned into a sill pillar. (Sjöberg, 1993) The reason why sill pillars are utilized is to increase the overall productivity, by enabling simultaneous mining in areas separated from each other. In other words, they are necessary for maintaining the prevailing stability by acting as a horizontal support element. Further studies show that a strategic placement of sill pillars could also lead to a significant reduction of hanging wall overbreaks. Since sill pillars hold a considerable amount of material, an important aspect is the recovery process of these elements. This is usually considered at the end of a mine's life. (Chen et al., 2021)

4.1.6 Pillars located in the footwall

A further pillar element, which is very relevant for the primary infrastructure of a sublevel stoping operations, is highlighted in red and located in the footwall of the orebody.

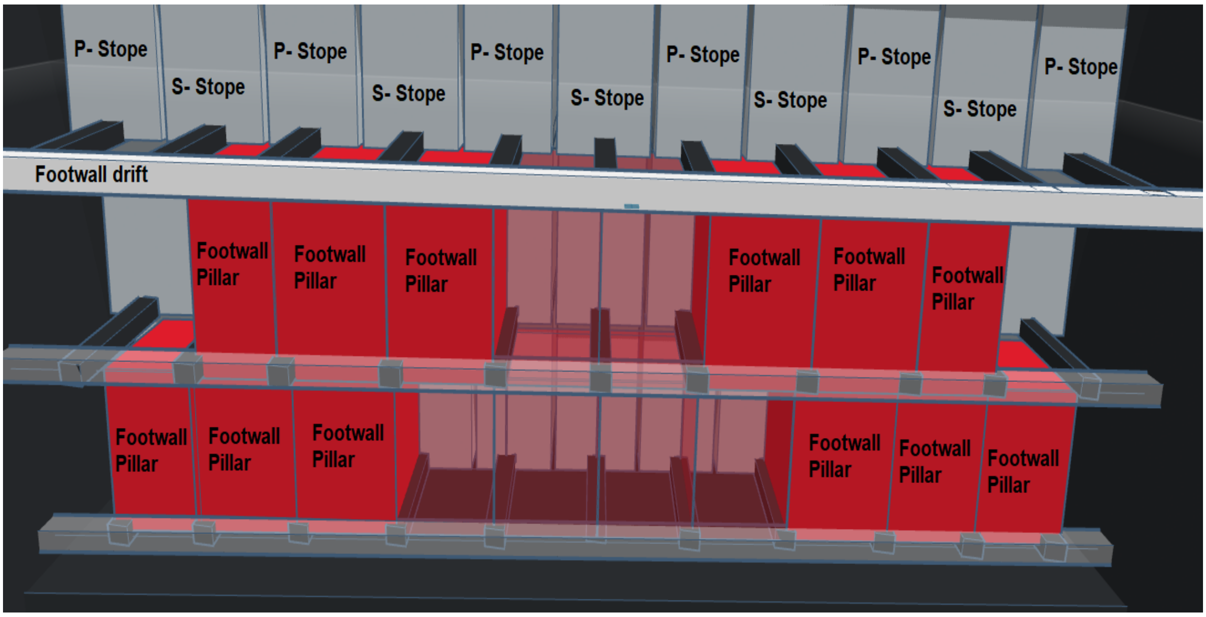


Figure 52: Element – Pillars located in the footwall

Definition and purpose

In contrast to stope-pillars (secondary stopes), these pillars unusually consist of country rock. For this reason, they are for the most part planned to be permanent and not temporary. The general purpose of pillars located in-between crosscuts, footwall drifts and stopes, is to protect the mine infrastructure from stoping activities. As these pillars are usually subject to multiple stress and relive patterns, depending on the used stoping sequence, their condition has to be monitored properly. A more detailed discussion concerning the rock mechanical relation between stopes and adjacent pillars, is provided in the upcoming sections.

4.1.7 Mining Panel

The next element to be discussed (highlighted in red) is the mining panel.

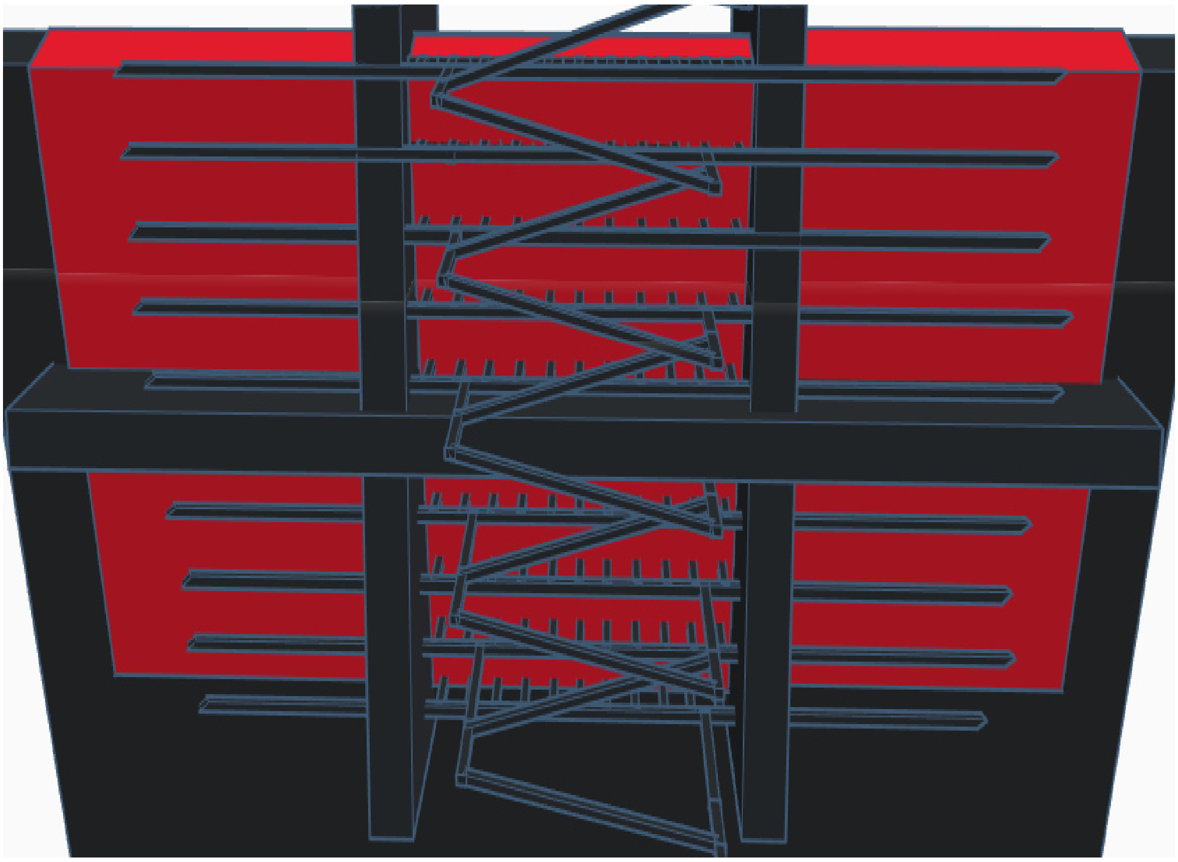


Figure 53: Element – Mining Panel

Definition and purpose

Large orebodies are usually divided into smaller subsections which are commonly called panels / mining panels. These areas are defined by certain boundaries and extend parallel to the strike of the orebody. The pillar-elements which form the border of these sections, are called barrier and sill pillars and are described in the previous sections. By dividing the orebody into individual areas which are independent from one another, stress redistributions caused by excavation processes can be limited to certain regions. This enables safer mining at multiple locations and also enhances production and quality control.

4.1.8 Stope Sequence

In contrast to the elements outlined so far, the stope sequence is not a direct subpart of the mine infrastructure, but rather an important excavation related procedure.

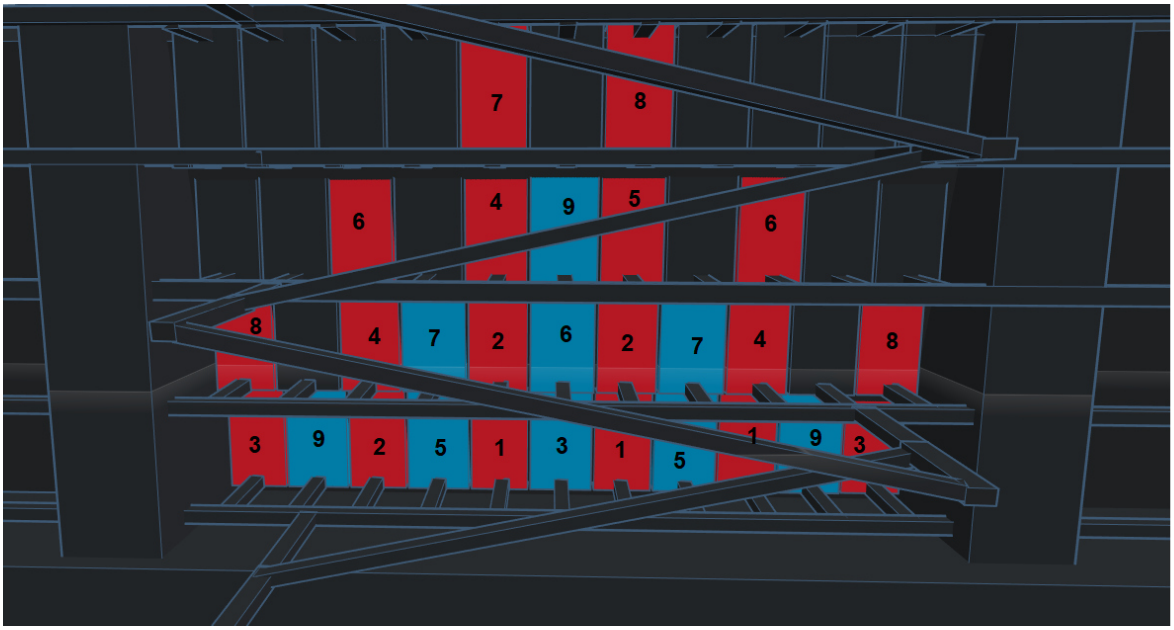


Figure 54: Element – Stope Sequence

Definition and purpose

In general, stope sequences are templates which point out possible excavation orders for stopes. One of the main reasons why stope sequencing has become such an important tool, is the resulting ability to control the behavior of the rock mass. Despite of a constant stope design, different stope sequences lead to different stress situations in the rock mass and the mine structure. By utilizing an appropriate sequence for certain situations, stress redistributions can be managed more effectively. (Shnorhokian, 2015)

Another important aspect, aside from the rock mechanical view point, is the quality management of the ore. To generate a constant ore production with a stable grade, it is of advantage to run as many working areas (stopes) as possible. Depending on the used stoping sequence more or less stopes can be extracted simultaneously. This, however, again has an impact on the stress environment. A list of frequently applied stoping sequences, can be found in chapter “4.3.5”.

4.1.9 Secondary development

The next element to be discussed concerns the infrastructure that is related to the extraction preparation and is commonly known as the secondary development.

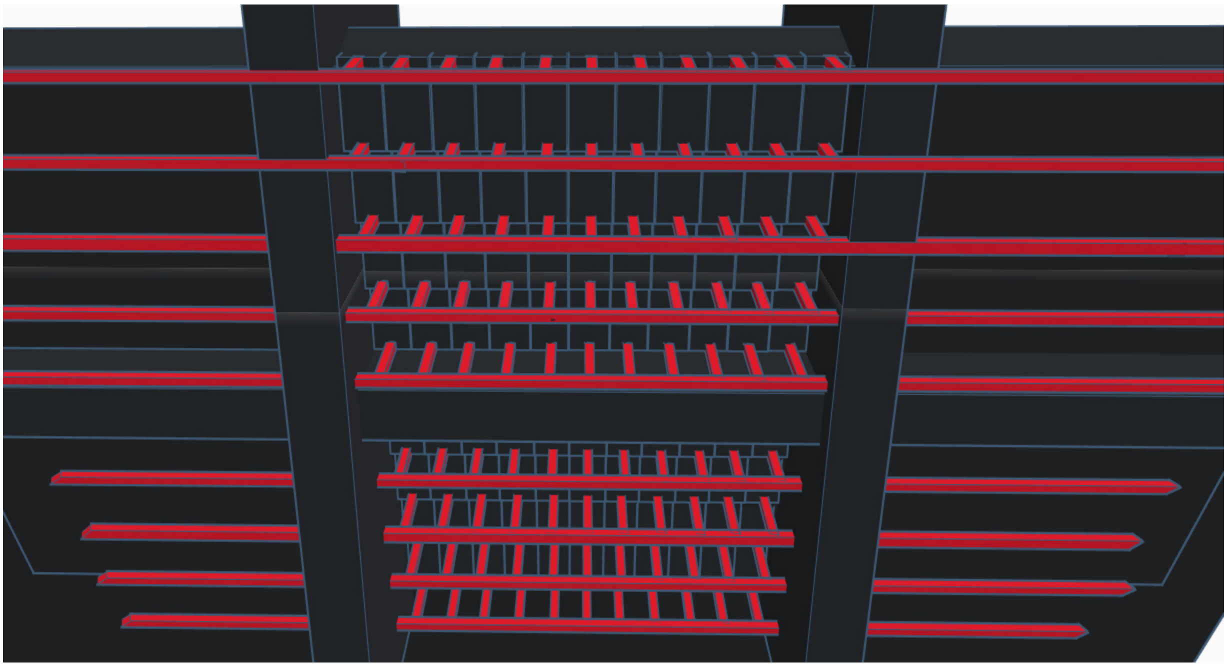


Figure 55: Element – Secondary development

Definition and purpose

The secondary development of a sublevel stoping operation, is characterized by a certain logistic and extraction infrastructure, involving numerous sub-parts. These sub-elements, which form the basis of the production infrastructure can be divided into sublevels, footwall drifts, cross-cuts, stope access drifts and further extraction related establishments. Besides the difference in purpose, it distinguishes itself from the primary infrastructure mainly by its overall lifetime.

The extraction related developments applied within a stope, are dependent on the used stoping layout and can be quite different from mine to mine. However, there are fundamental aspects which are quite similar in every sublevel stoping operation. These aspects will be highlighted in chapter “4.3.6”.

4.1.10 Primary development

The next element to be defined is the primary development of a stoping operation.

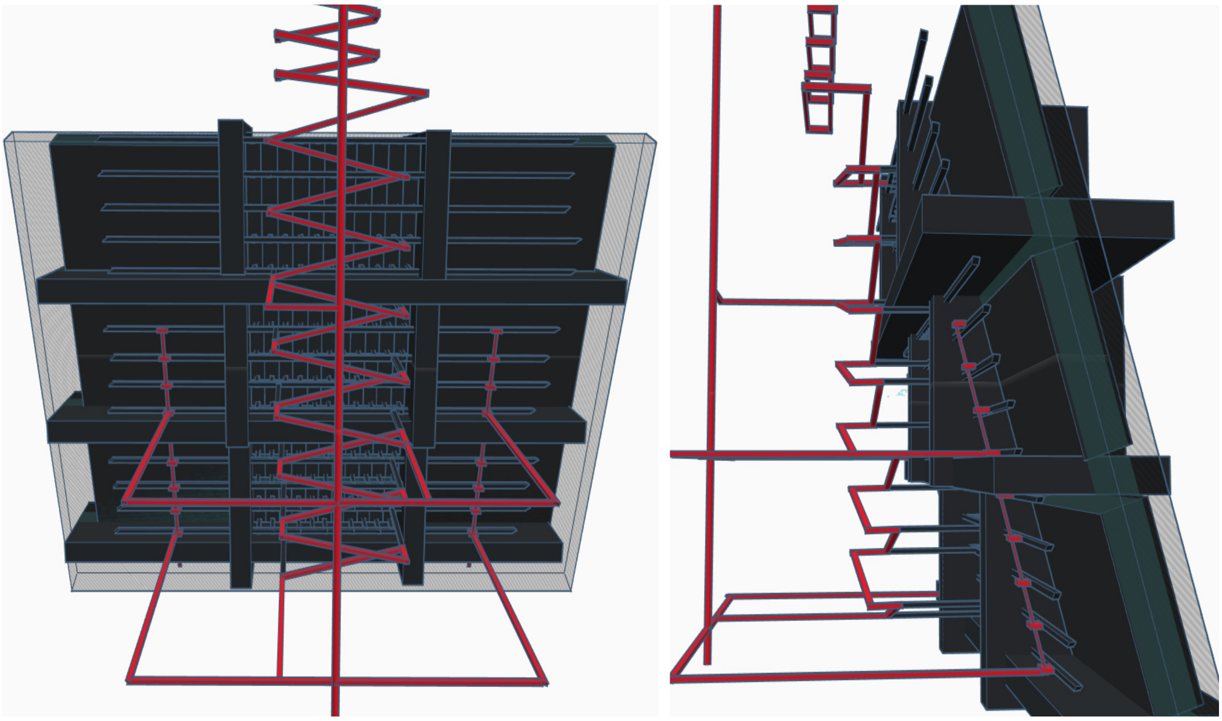


Figure 56: Element – Primary development

Definition and purpose

The primary development of a mine is defined as the direct access-structure, connecting the ground-surface to the orebody. In terms of rock mechanics this implies that this infrastructure should have a very long life-time and must therefore be designed with a high safety factor. Furthermore, it is essential to protect the primary development from mining-induced stress changes as well as seismic activities.

There are two main possibility's for establishing such an access. The first option is the utilization of a ramp and the second option is the construction of a shaft. This does not mean however that either one or the other must be taken. In many cases, a combination of both methods is used to access the deposit. Both access methods offer numerous advantages and disadvantages and the selection process, depends on many parameters. A more detailed discussion on ramps and shafts is provided in the chapter 4.3.7.

4.1.11 Backfill

Last but not least the element “Backfill” will be highlighted.

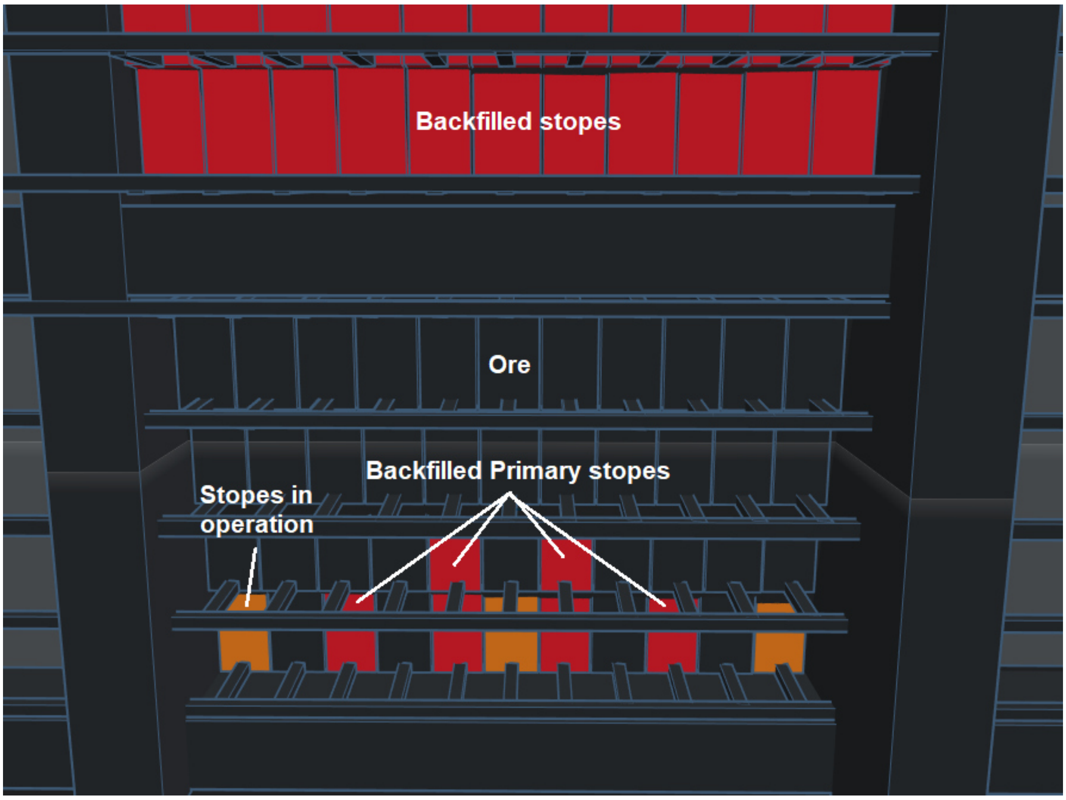


Figure 57: Element - Backfill

Definition and purpose

Backfill is material that is utilized to fill the void of an extraction-based cavity, with the purpose to support and stabilize the surrounding rock mass. Depending on the type as well as the consistency of the used material, the effect differs. What should be noted is that the interaction between rock mass and backfill is very complex, as not only the deformation of the rock mass has to be considered, but also other factors such as the load capacity of the used material. The second major purpose of backfill, is the disposal of accumulating waste rock. In regards to sublevel stoping this is particularly important for secondary stopes.

The most popular types which are in use today, are waste rock, paste fill and hydraulic fill. Each type offers various advantages and disadvantages in regards to transportation, fill rate, strength, fill density and many other factors. A more detailed discussion, about how backfilled is handled in stoping operations, follows in chapter 4.3.

4.2 Rock mass classification

Since a large number of design methods concerning stopes and pillars are based on the quality of the rock mass, this chapter provides an overview of the different classification systems.

4.2.1 Overview

One of the most important aspects in designing stable or purposely non-stable cavities, are estimations concerning the quality of the rock mass. Using classification systems, it is possible to determine whether particular mining methods, stope or tunnel designs can or should be applied. However, since the quality of the rock mass is characterized by numerous geological, rock specific and stress related parameters, the development of a classification system is rather difficult. For this reason, different classification systems emerged over time, varying in the calculation process and the valuation of the individual parameters. The three systems which are going to be highlighted in this chapter are, the RMR system by (Bieniawski, 1973), the Q system by (Barton et al, 1974) and the GSI system by (Hoek, 1994).

4.2.2 RMR – System

The Rock mass rating (RMR) was developed in 1973, with the initial intend to support tunnel projects by estimating the quality / strength of the rock mass. Further on, this system was not only applied to tunnels, but also to other underground rock engineering projects, in which the quality of the rock mass is of major importance. After ongoing development and adjustments, the current RMR system involves 6 input parameters, which are used for the valuation process. A list containing all parameters can be seen in the following.

Input parameters of the RMR system:

1) Strength of intact rock material
2) RQD
3) Discontinuity spacing
4) Discontinuity condition
5) Groundwater
6) Discontinuity orientation

The following figure highlights the original system developed by (Bieniawski, 1989).

C. ROCK MASS CLASSES DETERMINED FROM TOTAL RATINGS					
Rating	100 ← 81	80 ← 61	60 ← 41	40 ← 21	< 21
Class number	I	II	III	IV	V
Description	Very good rock	Good rock	Fair rock	Poor rock	Very poor rock

A. CLASSIFICATION PARAMETERS AND THEIR RATINGS

Parameter		Ranges of values						
Strength of intact rock material	Point-load strength index (MPa)	>10	4 - 10	2 - 4	1 - 2	For this low range, uniaxial compressive test is preferred		
	Uniaxial compressive strength (MPa)	>250	100 - 250	50 - 100	25 - 50	5 - 25	1 - 5	<1
	Rating	15	12	7	4	2	1	0
Drill core quality RQD (%)		90 - 100	75 - 90	50 - 75	25 - 50	<25		
	Rating	20	17	13	8	3		
Spacing of discontinuities		>2m	0.6 - 2m	200 - 600mm	60 - 200mm	<60mm		
	Rating	20	15	10	8	5		
Condition of discontinuities		Very rough surfaces Not continuous No separation Unweathered wall rock	Slightly rough surfaces Separation <1mm Slightly weathered wall rock	Slightly rough surfaces Separation <1mm Highly weathered wall rock	Slickensided surfaces or Gouge <5mm thick or Separation 1 - 5mm Continuous	Soft gouge >5mm thick or Separation >5mm Continuous		
	Rating	30	25	20	10	0		
Groundwater	Inflow per 10m tunnel length (l/min)	None	<10	10 - 25	25 - 125	>125		
	ratio (joint water pressure)/(major principal stress)	0	<0.1	0.1 - 0.2	0.2 - 0.5	>0.5		
	General conditions	Completely dry	Damp	Wet	Dripping	Flowing		
	Rating	15	10	7	4	0		

B. RATING ADJUSTMENT FOR DISCONTINUITY ORIENTATIONS						
Strike and dip orientations		Very favourable	Favourable	Fair	Unfavourable	Very Unfavourable
Ratings	Tunnels & mines	0	-2	-5	-10	-12
	Foundations	0	-2	-7	-15	-25
	Slopes	0	-5	-25	-50	

Figure 58: RMR System (Bieniawski, 1989)

What should be noted is that the value of the sixth parameter (Discontinuity orientation) depends on the construction to be built. That’s why the RMR is often calculated from the first five parameters only and then frequently referred to as the Basic RMR (RMR_b). (Ceballos, 2014)

In regards to the application of this system, each parameter should be measured and analyzed at several structural regions. These regions are characteristics by the same rock type or discontinuity structure and are separated from each other by faults. Depending on the value and quality, the individual parameters are then evaluated / graded according to a specific system, developed by Bieniawski. Last but not least the RMR-Value itself is calculated from the sum of all graded parameters. The result is an index ranging from 1 to 100, in which 100 represents the best possible rock mass quality and 1 the worst.

4.2.3 Q – System

The Q-system is a method to evaluate the current rock mass conditions by analyzing, confronting and weighting the positive as well as the negative aspects of various parameters of the rock mass. The initial data on which this system is based on, comprises 212 tunneling projects for road, rail, storage and hydropower, which were for the most part (60%) located in Scandinavia, but also in other regions of Europe and USA. In the process of data gathering, detailed research concerning jointing of the rock, type of support and apparent stability were carried out. Overall 50 different rock types were represented in this initial study. In the course of time the database was enlarged to more than 1000 use cases and a diagram based on this data, could be extended continuously. This chart is today known as the Q-support chart. Using this diagram, it is possible to estimate the needed support, if the Q-value of the rock mass, as well as the dimensions of the specific cavity are known. (Barton,1988) (Palmstorm, 2006)

The equation for calculating the Q-value, consist of three important sub-parts. These three parts in turn, comprises six parameters and can be characterized as follows:

RQD / Jn … Indication for the relative block size

Jr / Ja … Describes the Inter-block shear strength

Jw / SRF … Indicator for Active stresses

RQD … Rock quality designation
Jn … Joint set number
Jr … Joint roughness
Ja … Joint alternation
Jw … Joint water reduction factor
SRF … Stress reduction factor

Figure 59: Parameter description – Q System (Barton, Grimstad, 1993)

To get an overview regarding this categorization, the next figure displays the formula as well was the classification system.

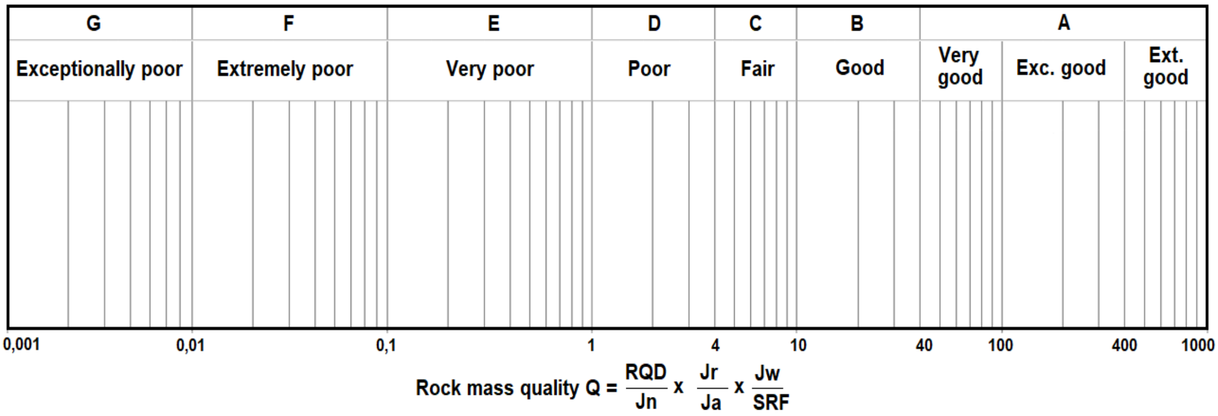


Figure 60: Classification of rock mass – Q System (Barton, Grimstad, 1993)

4.2.4 GSI – System

The GSI (Geological Strength Index) was developed by (Hoek, 1994) and has undergone several modifications since then. In contrast to the Q-system, one of the main goals was to develop a rock mass categorization method that is very straightforward and can be applied rather quickly. Another objective was to create an index that comprises various rock mass properties, which can be reliably used as input parameter for numerical analysis. The overall function of this system is not to estimate and design reinforcement for tunnels or excavations, but simply to assess properties of the rock mass. The two main elements on which the GSI is based on, is on the one hand the overall structure of the rock and the other hand the surface conditions of potential discontinuities. More precisely, the position as well as the number of discontinues form the foundation of this categorization method. The assessment is consequently based on the visual inspection of the rock mass. All these features turn the GSI into a “easy to use” geologically based index. (P. Marinos, V. Marinos, E. Hoek, 2007)

4.2.5 General Application of Q- and RMR-System

Since the RMR system as well as the Q-system form the basis for various empirical design methods, an important point to be addressed concerns the general area of application of the individual systems. In this context the main usage concerning the RMR system is predesigning of tunnels, underground excavations, and other mining structures (Aksoy, 2008). Related to the RMR, (Bieniawski, 1989) also provided various guidelines concerning the support of rock tunnels with a span of up to 10 meters. Using the RMR value it is also possible to determine various rock mass properties like UCS or Deformation modulus, as many empirical relations have been developed over the years. (Abbas, 2015) The main application of the Q-System is to determine the needed support, which is necessary for an underground excavation with certain dimensions to be stable. This is done by applying the Q-support chart, which has been continuously advanced until today. As the total number of cases used to develop the Q-System is quite significant, credible support recommendations are provided for a broad spectrum of tunnel sizes, excavation types, depths, and rock qualities. (Barton,1988)

When it comes to limitations the RMR system shows inaccuracies when estimating the support requirements. Overall it is known to be much more conservative than the Q-system, which leads to overdesigning in certain situations. A further problem is that the majority of historical cases are based on competent rock, which automatically makes it somewhat unreliable in weak rock (Singh and Geol, 1999). Last but not least the RMR system is not intended for deciding on which mining method would be most suitable. (Abbas, 2015) The Q-System on the other hand has the disadvantage that the determination of the parameters is not quite simple. Especially the Stress Reduction Factor is difficult to estimate as a wide range of in-situ stresses are covered with a single value. For this reason, the Q-System is not appropriate for detailed designing. (Kaiser, 1986) Last but not least, according to (Palmstrom & Broch, 2006) this system is generally not appropriate for soft rock, but rather for advances in which drilling and blasting is applied.

4.3 Elements of a Sublevel stoping operation

In this chapter a broader analysis of certain elements which are defining for this mining method will be made. This includes the main structural elements like stope and pillars (Stope-pillar, Barrier pillar, Sill pillar), the mine infrastructure (Primary and secondary development) as well as functional aspects like backfilling and stope sequencing. The overall aim of this chapter is to provide a state-of-the-art overview, concerning the most important design aspects of the individual elements. This involves various design methods, critical parameters and all effects that are associated with the design process.

4.3.1 Stope

The first element to be discussed in more detail is the stope. The following section provides an overview of all design method-types which can be used to design and estimate the stability of stopes.

4.3.1.1 Stope – Overview

The main objective in terms of stope planning is the creation of a stable cavity. In this process the generated void should be as large as possible (for efficiency reasons), but also as stable as possible (to prevent outbreaks and dilution). From a rock mechanical standpoint this seems to be difficult, as both aspects conflict each other. However, there are several types of design methods which can be applied to meet these requirements:

- Empirical stope design
- Semi-empirical methods
- Analytical methods
- Numerical Modelling

Each of these design categories is characterized by certain features and limitations regarding its application. These will be outlined in the following section.

4.3.1.2 Stope – Design methods

As can be perceived from the list above, there are many different design approaches to plan and develop a stable stope. The most frequent used type to verity stope stability, is based on the empirical methodology. For this reason, this will be the first point to be addressed.

Empirical stope design

The widespread application of empirical methods can be attributed several reasons. On the one hand the utilization of these methods is, compared to the more complex and time-consuming numerical approach, rather simple. On the other hand, due to the reliance on experience in combination with rock mass quality systems, empirical methods provide a solid solution concerning the inhomogeneity of the rock mass.

The most popular rock mass classification systems, which are implemented in empirical stope design methods are the Q-System by Barton and the RMR by Bienawski. (Stewart, 2005) More information on these classification systems can be found in the previous chapter. In the course of years, numerous empirical methods and graphs have been developed. The following list offers an overview of the most popular methods which are frequently used for stability analyzes and stope planning:

- Mathews Stability Graph (Mathews, 1981)
- Modified Stability Chart (Potvin, 1988)
- Extended Mathews Stability Graph (Trueman & Mawdesley, 2003)
- Laubscher Design Graph (Laubscher, 1994), (Bartlett, 1998)
- Shape Factor (Milne, 1996)
- ELOS Stability Chart (Clark and Pakalnis, 1997)
- Mathematical Dilution Model (Pakalnis, 1986)
- Critical Span Graph (Lang, 1994)

As indicated in the list above, there many different types of stability charts that can be applied to design a stope. Therefore, the purpose regarding utilization is essential for the decision process. Another important matter concerns the data on which the graph is based on. Stability charts are only useful if the conditions in which they are applied are quite similar to the conditions they were developed in. This indicates that in order to make a proper choice, the database should be analyzed carefully. (Suorineni, 2010)

Empirical methods offer a fast and straightforward solution to problems that have a very complex root. However, a simple diagram cannot represent the nature and behavior of a highly complex constantly changing joint structure known as the rock mass, accurately. A major limitation for example, is that the stability graph method does not consider relaxation zones around stopes, which verifiably can cause instabilities. (Sepehri et al., 2017) Furthermore, they do not rely on a detailed understanding of failure mechanisms. Due to these shortcomings empirical methods should, according to (Cepuritis & Villaescusa, 2012), only be used for preliminary designs.

In order to get a general overview concerning the application as well as the differences to some of these methods, two stability graphs will now be illustrated.

Mathews stability graph

Since the Mathews stability graph is widely used and by now has many different versions, a brief overview of this method will be provided. The original "Mathews stability chart" was developed by (Mathews et al., 1981). The purpose of this graph was to predict a stable stope span for mines exceeding 1000 meters in depth. Thereby the original chart is based on 50 historical cases (Mathews et al., 1981). In the course of the years, the database was extended by Potvin, who collected 176 new cases between 1986 and 1987 (Stewart, 2005). This new chart, is known as the "Modified Stability Chart" and commonly used today. Another important expansion took place in 2003 by Trueman and Mawdesley. The number of historical cases was increased one more time from 176 to 485 and the "Extended Mathews Stability Graph" was developed. However, a noteworthy fact, due to acknowledged uncertainties concerning the parameters used in the calculation of the stability number, 100 cases provided by Potvin have been replaced. Furthermore, there is also a difference in the calculation process of the stability number, which describes the Y-axis of the stability chart. While the "Extended Mathews Stability Graph" uses the original calculation factors for the stability number "N", the "Modified Stability Chart" uses slightly different A, B, C factors to calculate the modified stability number "N'". Both versions are very promising and in use today. (Stewart, 2005)

The following figure illustrates the "Extended Mathews Stability Graph".

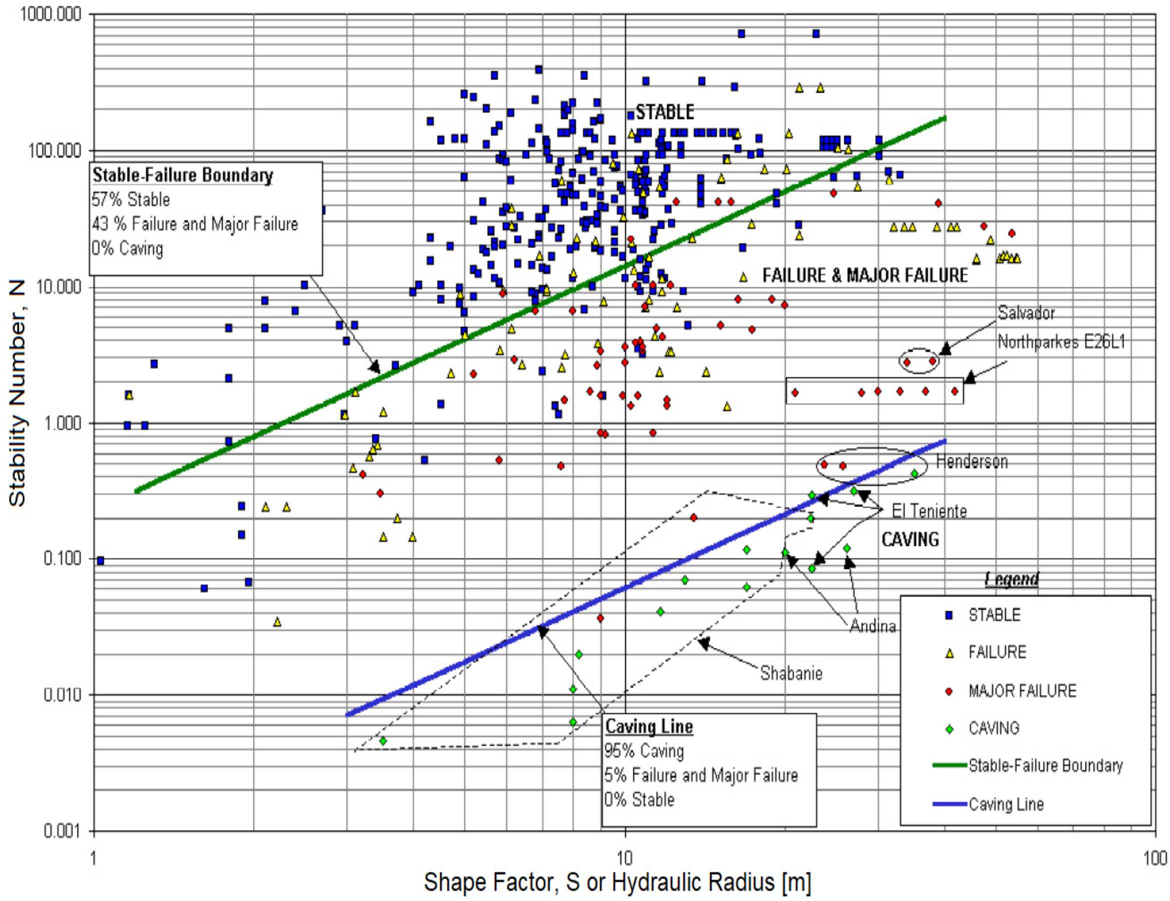


Figure 61: Extended Mathews Stability Graph (Trueman & Mawdesley, 2003)

To empirically determine whether a planned stope is stable, two parameters must be known. The first one is the hydraulic radius or shape factor and displayed on the X-Axis. The second one is the stability number (N), which describes the Y-Axis. Depending on the values of both parameters the stope can be incorporated and its behavior assessed.

<u>The stability number is calculated as followed:</u> $N = Q' \times A \times B \times C$

A… Rock stress factor – Ratio of UCS of intact rock to induced compressive stress

B… joint orientation adjustment factor – Orientation of critical structure to stope surface

C…Gravity adjustment factor – Effect on stability of gravity on a horizontal surface is 8 times that of a vertical surface

Concerning the hydraulic radius and shape factors, a more detailed description is provided in the upcoming section. The next stability graph which is going to be highlighted, is the critical span graph developed by Lang in 1994.

Critical Span Graph

The original span graph contains 172 cases of entry-type excavations and was extended twice in the course of time. The first update happened in 2002 by Wang who extended the database to a total of 292 cases. A year later a final extension took place by Kumar to 399 cases. The following figure illustrates the originated graph.

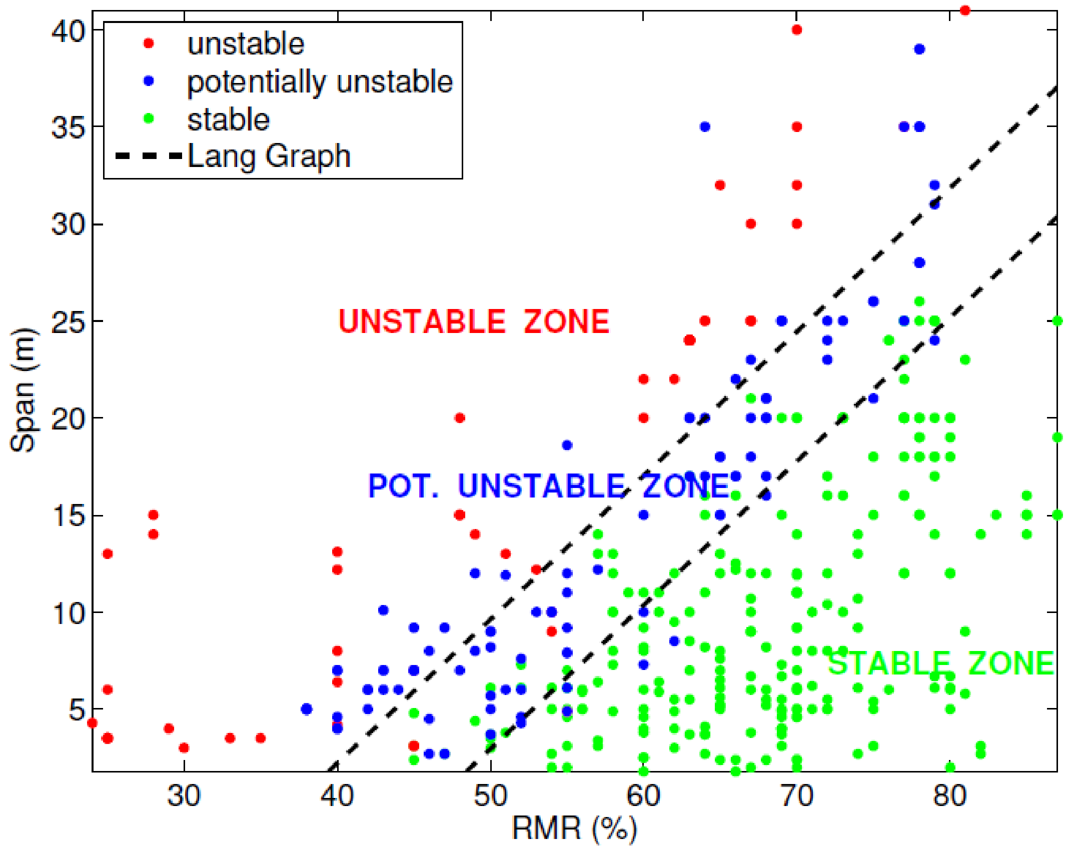


Figure 62: Critical Span Graph (Lang, 1994)

Observing this figure, it is apparent that the graph is characterized by two dotted lines which subdivides the overall area into three different zones (Unstable, potentially unstable and stable). The two main parameters which are utilized in this stability graph is the RMR_{76} (X-Axis) and the Span (Y-Axis). A definition of the RMR can be found in chapter 4.2.2. The span on the other hand is defined as followed: “The diameter of the largest circle that can be drawn within the boundaries of the exposed rock, in plan view”. (Garcia-Gonzalo, 2016)

Although the principle of both charts is quite similar, there are nevertheless certain differences. To further highlight these, a brief discussion concerning the area of application will be made.

Discussion

The area of application concerning the extended Mathews stability graph and the critical span graph overlaps, but is certainly not the same. An obvious difference is that the Mathews chart, utilizes a much more complex process for determining the stability of a stope. In fact, not only basic rock mass characteristics are involved in the calculation, but also specialized factors dealing with the appearance of stope and rock mass structures (factors B and C). A further difference is the inclusion of depth related stresses, which are described by the factor A in the stability number. All these modifiable parameters turn the Mathews graph into a well usable empirical design tool for stopes. The critical span graph on the other hand, has widely been accepted as a simple tool to estimate the maximum span, that can potential be used in a design, based on the known RMR_{76}. (Garcia-Gonzalo, 2016)

Semi- Empirical methods

As well as empirical methods, semi-empirical methods are nowadays commonly applied when it comes to stope planning. They combine the obtained results from empirical methods with mechanistic models and assumptions. (Zhao, et al., 2019) An overall objective of semi-empirical methods such as back analysis, is to provide functions which are related to the past performance of stopes and can also be used as a reliable indicator for future performance. (Cepuritis & Villaescusa, 2012) Although such a method cannot be applied for a preliminary stope design, it does provide valuable information on the correlation between the stope stability and individual parameters of specific stability charts. Furthermore, it is an important tool to review the reliability of stress related damage and failure criteria, of numerical models. (Cepuritis & Villaescusa, 2006)

According to (Suorineni, 2010) there are important limitation which have to be considered when using semi-empirical approaches. The first point is that such methods do not address the problematic of progressive rock mass failure, which can or cannot occur over time. In this context, these methods do not lead to any detailed understanding why a failure happened or what caused a failure. The second issue is that the exact geometry of the planned stope as well as the associated mining process is disregarded in this methodology.

Analytical methods

Besides empirical and semi-empirical methods, analytical approaches have proven to be very valuable, when it comes to stope planning. These methods are particularly useful to obtain an initial evaluation of certain situations and problems quickly and in a straightforward manner. A very frequent application for example, is the identification of extreme stresses or deformations. (Mathews et al., 1981) In that context, analytical design methods are usually based on stress and deformation analyzes around underground excavations / openings.

A very popular analytical approach to analyze stresses was developed by (Kirsch, 1898). Using these equations, it is possible to calculate the stresses as well as the displacements around a circular tunnel. Other popular methods which can be applied to evaluate the stability of excavations, is the Beam and Plate Theory (Obert & Duval, 1967) and the Voissoir Beam Theory (Brady & Brown, 1993).

According to (Pakalnis, 1986) analytical methods are mostly applied to obtain additional comparative designs and to create parametric studies. Furthermore, since these methods interpret the rock mass as a homogenous, linear elastic material, results concerning larger structures have to be interpreted with care. A further issue analytical methods entail is that they only consider one failure mechanism. This is one of the main reasons why they should be utilized in combination with empirical methods. (Stewart, 2005)

Numerical methods

The core aspect of numerical modeling is to make use of mathematical models, that describe specific physical processes and scenarios, by using / solving various equations. The purpose as well as the difficulty hereby is to reflect a given scenario, like the probable behavior of rock masses, in a highly realistic way. To produce authentic results, large amounts of data as well as a certain degree of computing power must be provided. This leads to the fact that this methodology is more complex and time-consuming than others design approaches.

An important aspect in numerical modeling is the mathematical solution which used for the calculation and simulation process. For example, continuum-based software, which relay on partial-differential equations, is widely spread and often used in the fields of geomechanics and rock mechanics. In this context methods such as the finite-element, boundary element or finite difference method are commonly applied to assess the stability of open stopes and pillars. (Sepehri et al., 2017) An example for a software package that is commonly used in the fields of underground mining is FLAC2D. It utilizes the finite difference solution to represent the rock mass and its behavior in relation to the added loads, through various models.

One of the biggest issues numerical models entail is the strong dependence on accurate input parameters, when it comes to stability assessments of created cavities. (Mawdesley, 2002) Especially in the beginning of a mining operation it is challenging to identify accurate values for certain parameters. This is why numerical models should not be used singly, but only in combination with empirical and analytical methods, in an early stage of a mining project. (Stewart, 2005). Furthermore, depending on the operation phase as well as the use case, it is reasonable to apply different models. For example, elastic (linear) approaches can potentially be used in earlier stages, than the more complex inelastic (non-linear) approaches. (Sepehri et al., 2017)

Summary – Design methods

In order to summarize all the above-mentioned design approaches, a figure created by (Cepuritis & Villaescusa, 2012) will now be presented. This graphic illustrates very well the fields of application as well as the timing in which these design methods can be applied.

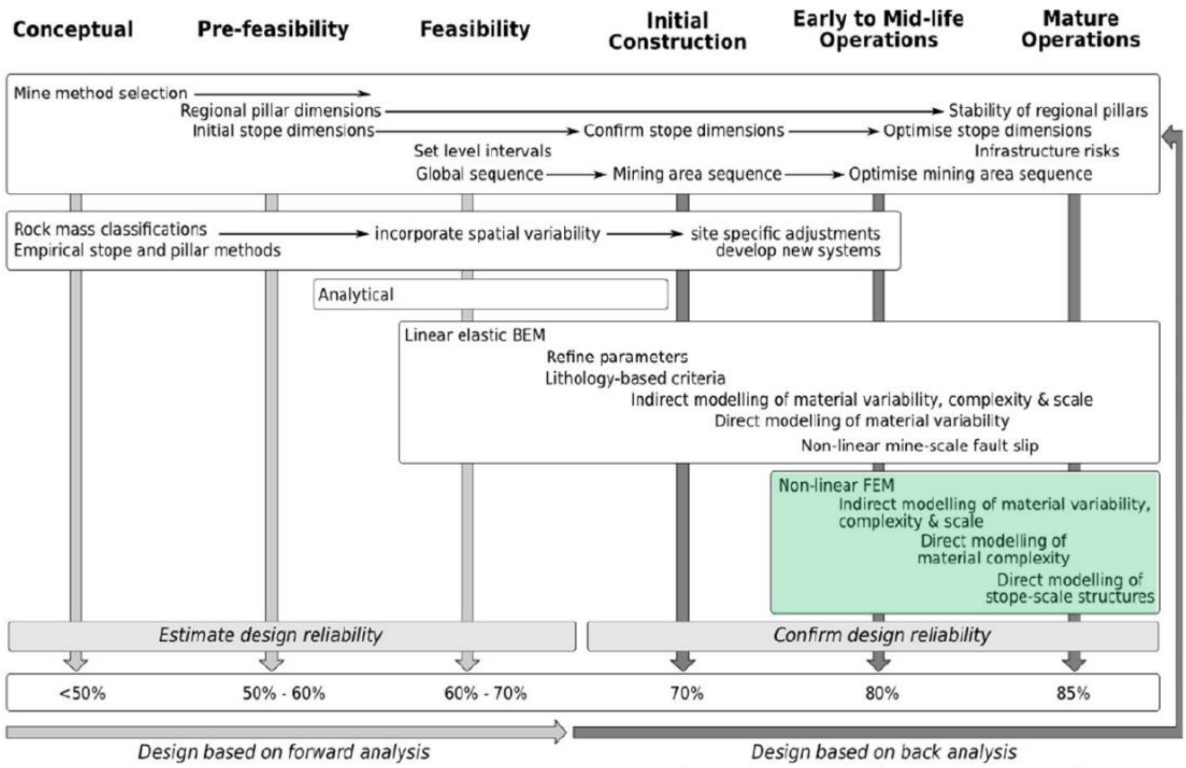


Figure 63: Application of design methods related to the phases of a mining project (Cepuritis & Villaescusa, 2012)

What can be concluded from this figure, is that empirical as well as analytical methods are very useful in the early stages of an operation, as there is only very limited data to create a meaningful numerical model. Another noticeable fact, almost all phases are characterized by the simultaneous application of multiple methods. Through this approach, the benefits of different methods can be combined and design alternatives can be analyzed on a broader scale. Last but not least, this figure presents a good indication regarding the design reliability, related to the operation phase and the used methodology.

In order to provide a deeper insight into the problems and difficulties concerning the design process of a stope, the next chapter will take a closer look at important parameters, which are used within these design methods.

4.3.1.3 Stope Design – Critical parameters

Throughout the planning process of a stope, numerous geometrical and rock mechanical parameters have to be investigated, weighted and determined by applying various methods. To get a clear overview which parameters are essential for the overall stope design process, a summary is made in the following.

<u>Geometrical paraments which have to be determined:</u>

- Shape related parameters
  - Stope / cavity profile
  - Shape factors (Hydraulic radius, Radius factor)
  - Width to height ratio
- Dimensions of the stope
  - Stope Span (Width)
  - Stope Height
  - Stope Length
  - Stope Dip
- Orientation of the stope
  - Transverse
  - Longitudinal

<u>Critical paraments which must be monitored and considered during the design process:</u>

- Deposit geometry
  - Depth
  - Orebody Thickness
  - Strike length
- Rock mass related parameters
  - Rock mass quality
  - Rock parameters
- Stress environment
  - Prevailing virgin stresses
  - Future stress redistributions

4.3.1.3.1 Parameter Description

To highlight the purpose as well as the effects of the above-mentioned geometrical parameters, an elaboration will be made in the following. In that context, the differences in acting stress concerning the applied stope shape (Profile), will be demonstrated first.

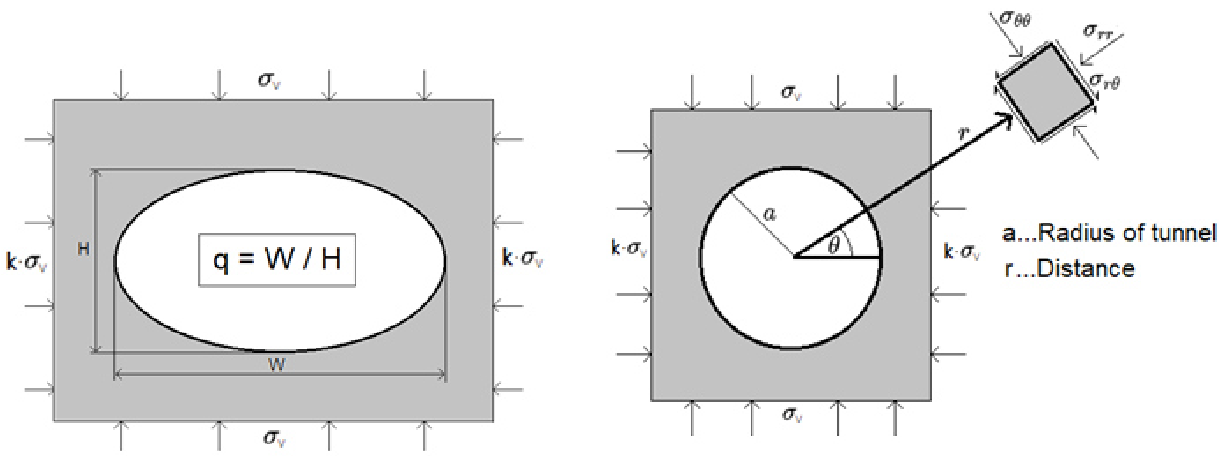
Stress effects of cavity profiles

To point out the difference, concerning the acting stresses caused by the shape of a created cavity, a circular profile will be compared to an elliptical profile. In that context the (Kirsch, 1898) equations will be used to calculate the tangential stresses, in the roof and the sidewalls of a circular shaped tunnel. In order to obtain the acting stresses around an elliptical profile, the (Inglis, 1913) approach will be applied.

Formula to calculate tangential stresses around a circular tunnel:

$$\sigma_{\theta\theta} = \frac{\sigma}{2}\left[\left(1+\mathrm{K}\right)\left(1+\frac{a^2}{r^2}\right)+\left(1-\mathrm{K}\right)\left(1+3\frac{a^4}{r^4}\right)\cos 2\theta\right] \qquad \text{(Kirsch, 1898)}$$

Visualization of the cavity-profiles and the applied formulas:



Tangential stresses:

Side wall ($\theta = 0°$): $\sigma_{\theta\theta} = \sigma\,(1 - k + 2q)$

Roof ($\theta = 90°$): $\sigma_{\theta\theta} = \sigma\,(k - 1 + 2k/q)$

Tangential stresses:

Side wall ($\theta = 0°$): $\sigma_{\theta\theta} = \sigma\,(3 - k)$

Roof ($\theta = 90°$): $\sigma_{\theta\theta} = \sigma\,(3k - 1)$

Figure 64: Tangential stresses in Circular and Elliptical tunnels (Kirsch, 1898) (Inglis, 1913)

The results of the displayed equations, can be seen in the following table. It should be noted that the hydrostatic case (k = 1) will be used for simplicity reasons and the ratio q is set to 2 (W=2, H=1).

	Elliptical	Circular
Side wall	4σ	2σ
Roof	1σ	2σ

Figure 65: Tangential stresses - Elliptical and Circular shaped tunnels

This example shows, that a change to an elliptical shape affects the acting tangential stresses significantly. Strictly speaking they halved at the tip of the roof and doubled in the middle of the side walls in comparison to the circular shape. The inclusion of profiles in stress calculations, is therefore of great importance. In regards to stope planning, shapes are often utilized to generate more suitable stress conditions. An example for this is the stope design applied in the Mittersill mine, where spherical roofs are used. The next parameter to be discussed, which is also related to the stope shape is the so-called shape factor.

Shape factors

Shape factors are indicators for the void exposure of underground openings, such as a created stope or a cavity. They have proven to be very valuable for empirical and semi-empirical methods as they provide a good reflection of the stope size in connection to its shape. One of the most frequent used shape factors is the hydraulic radius. It is mathematically defined by “Area (Stope cross section) divided by its perimeter” and is applied in Mathews stability chart and Laubscher Design Graph. According to (Milne & Pakalnis, 1997) the hydraulic radius is very suitable for two-dimensional rectangular surfaces, but not necessarily for complex geometries. Another example of a shape factor is the radius factor, which according to (Milne et al., 1996) provides a more accurate evaluation of the distance for irregular shaped geometries. It is defined as half the harmonic radius and can be calculated by using the following formula. It should be noted that ‘n’ represents the number of measurements while ‘r_0’ indicates the length to an abutment.

$$RF = \frac{Rh}{2} = \frac{0.5}{\frac{1}{n}\sum_{\theta=1}^{n}\frac{1}{r_\theta}}$$

Another important factor which is related to the shape of a stope is the applied width to height ratio. However, as this parameter is especially interesting for secondary stopes in connection to pillar planning, this topic will be discussed in the upcoming section.

4.3.2 Stope Pillar

The next element to be discussed in more detail is the stope pillar. In this regard, important general information concerning the design process will be covered in the overview. Since the structure of a sublevel stoping operation is characterized by different types of pillars, it is important to note, that all formulas mentioned in the upcoming section (4.3.2.2), are only intended for stope and rib pillars.

4.3.2.1 Pillar – Overview

The most important aspect of pillars is that they have to be designed in such a manner, that the loads they are intended for, are supported properly. To achieve this task, various formulas have been developed in the past, with the purpose to estimate the stability of certain structures.

One of the most popular variables for describing the stability, is the so-called factor of safety (FOS). In regards to pillars, this factor (ratio) results from the strength of the pillar and the load acting on that pillar. This indicates the higher the calculated FOS-value the more reliable the structure.

$$\mathbf{FOS} = \frac{\mathbf{S}_P}{\sigma_P} \qquad \begin{array}{l} \mathbf{S}_P \text{ .... Pillar Strength} \\ \sigma_P \text{ .... Average Pillar Stress} \end{array}$$

The difficulty however, is to obtain accurate values for the variables describing the FOS. Especially the assessment of the pillar strength is quite challenging, due to the natural inhomogeneity of the rock masses. For this reason, numerous approaches regarding the calculation of the pillar strength were developed over the years. To get an overview, the next section provides a broad insight.

4.3.2.2 Pillar – Design methods

Empirical methods in combination with numerical modelling can be seen as the current state of the art, when it comes to the design of pillars. Some of the most prominent empirical formulas and graphs will now be presented. What should be noted, according to (Malan & Napier, 2011) whether numerical modelling nor empirical methods should be used singly for the design process, as only both together can provide a reliable foundation to predict the pillar strength.

One of the most famous formulas for calculating the strength pillars was introduced by Salamon and Munro in 1967. During their research, which is based on 125 coal pillars located in a South African mine, they came up with the following formula.

$$S_P = K \times W^{\alpha} / H^{\beta} \qquad \text{(Salamon \& Munro, 1967)}$$

K ..... adjusted or non-adjusted strength of a unit cube of pillar rock determined statistically or through laboratory test results.

H ..... pillar height

W ..... pillar width

α, β .... empirical constants

The values used for the constants, were as followed:

$$S_P = 7{,}2 \times W^{0{,}46} / H^{0{,}66}$$

What should be noted is that this formula is intended for square pillars only. In order to calculate the strength of rectangular pillars, the width (W) has to be replaced by an effective width (W_e). According to (Wagner, 1980) the variable W_e can be calculated by using the following formula.

$$W_e = 4A / C$$

A... Pillar plan area

C... Pillar plan circumference

In 1972 Hedley and Grant adopted the formula developed by Salamon and Munro and used it in their research to design hard rock pillars. The data they applied originated from Canadian uranium hard rock mines and comprise 28 pillars, located in a depth between 150 and 1040 meters. What should be noted is that the database mostly consists of pillars indicating a W/H - ratio between 1 and 1.5. Therefor this method should not be applied to pillars showing a slenderness outside of this area. (Oke et al., 2017) In regards to the strength, a unit cube hard rock was statistically determined to be 133 MPa. Unfortunately, there was no information on the rock mass quality. (Zvarivadza, 2012) Their completed formula appeared as follows.

$$S_P = 133 \times W^{0{,}5} / H^{0{,}75} \qquad \text{(Hedley \& Grant, 1972)}$$

In addition to their developed formula, Hedley and Grant used the collected data to create a pillar stability graph. The purpose was to visualize the relation between stress, strength and width to height ratio of the pillars.

Another very popular stability graph was developed by Potvin et al. in 1989. The data used for the development of the graph originated from various open stope mines in Canada. They stated that the majority of analyzed pillars within these mines, were characterized by a "good" rock mass quality. What should be noted is that the yielding line indicates the border area, where the first major issues concerning stability occur.

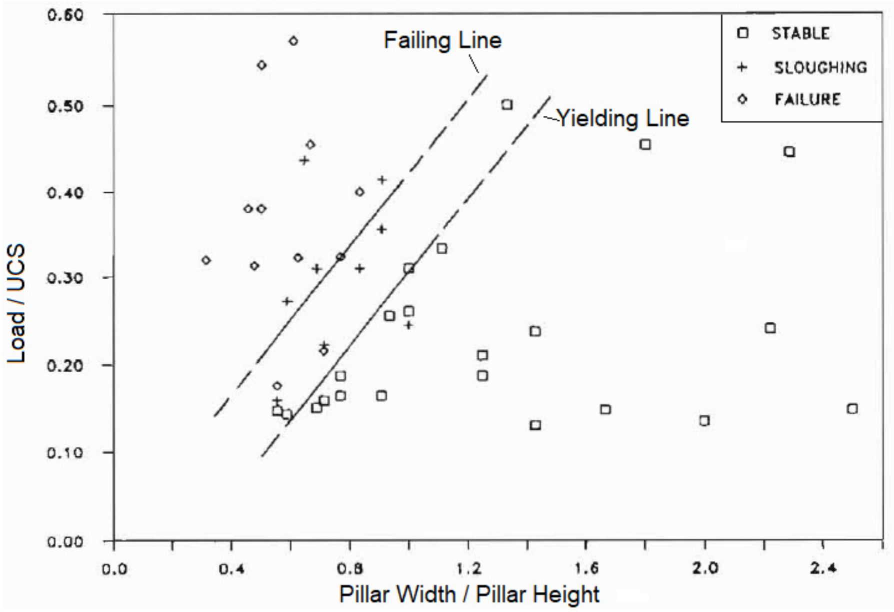


Figure 66: Pillar stability graph (Potvin et al., 1989)

In regards to this graph, there are several recommendations provided by (Potvin et al., 1989). Permanent as well as temporary pillars should be designed below the yielding line, if they are intended to be stable. On some occasions however, it is useful to design temporary pillars to fail. In that case, the designed pillar should be on or above the failing line. Additionally, the backfill procedure in surrounding stopes must be carried out as quickly as possible. The advantage of such a "failing" design is that the recovering process is easier, as the pillar will not get overstressed.

In regards to the estimation of the pillar strength, many more empirical formulas and graphs emerged over the years, covering various locations like mines in Botswana by (Kimmelmann et al., 1984) and mines in Sweden by (Sjoberg, 1992b). One of the most commonly applied charts, when it comes to the design of hard rock pillars, was compiled by (Lunder & Pakalnis, 1997). The graph they developed, is based on a large merged database, consisting of many important studies from the past (178 cases). Furthermore, the formula for calculating the pillar strength, shows a higher degree of complexity as it also includes the confinement. However, the graph does not differentiate between pillar types nor the used type of stress analyses. Furthermore, there is also no differentiation between rock mass qualities and other parameters like the stiffness. According to (Oke et al., 2017) this chart is therefore very conservative when it comes to the design of long pillars and rather “aggressive” when it comes to the design of square pillars. The resulted stability graph by (Lunder & Pakalnis, 1997) can be seen in the following figure.

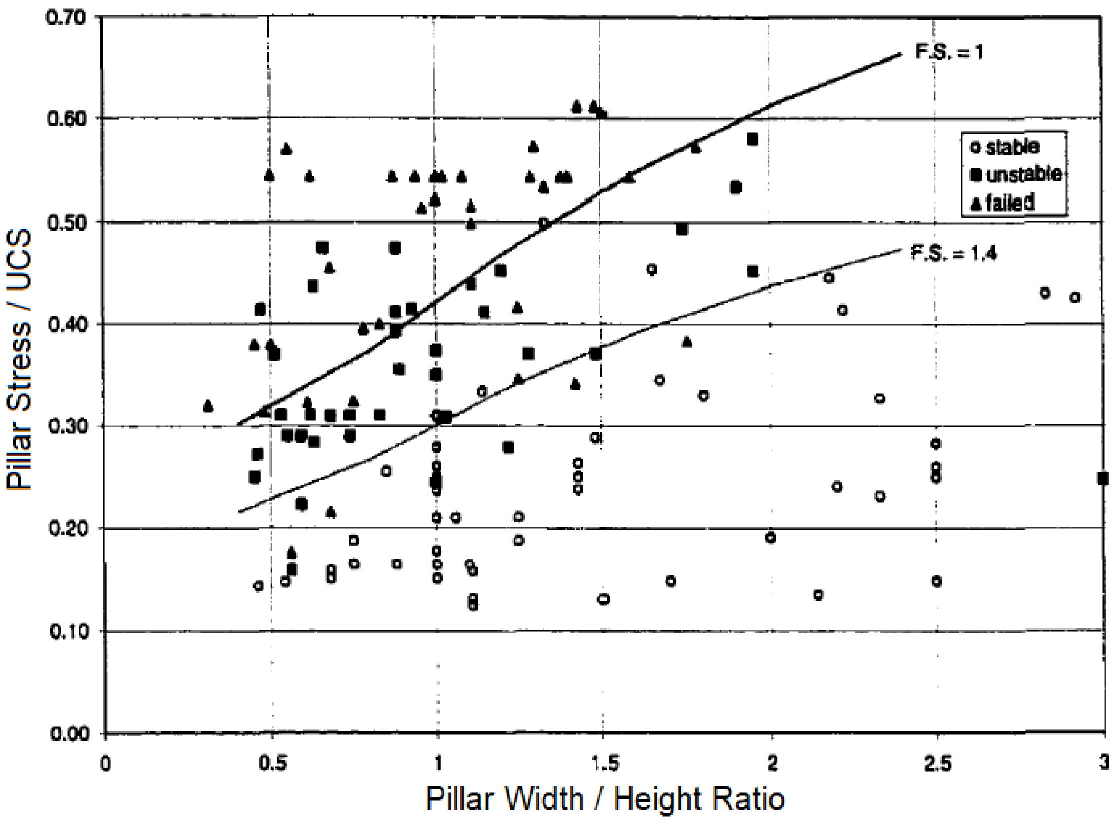


Figure 67: Pillar stability graph (Lunder & Pakalnis, 1997)

Lunder & Pakalnis used two FOS lines, to indicate the stability of the pillars. In that context they stated, pillars showing an FOS > 1,4 are defined as stable, while pillars indicating an FOS < 1,0 fail. The area between 1 and 1,4 is declared as unstable, with a tendency for “partial failure”. (Martin & Maybee, 2000)

Besides empirical methods, there are also various semi-empirical approaches which combine empirical assessments with specific models. A popular example is the semi-empirical pillar design graph developed by (Diederichs et al., 2002), who utilized a theoretical damage model to analyze the pillar stability. A disadvantage this graph entail, is that it does not provide direct information on the width to height ratio, leading to a more difficult pillar shape optimization process.

Last but not least a summary chart created by (Martin & Maybee, 2000), containing the most popular hard rock pillar stability formulas, is presented. It should be noted that the two lines indicated by “Hoek-Brown brittle parameters”, represent the results of a 2D-elastic model which was developed by (Martin & Maybee, 2000). The objective of this model was to utilize the Hoek-Brown brittle parameters and to analyze if the resulted pillar strength curves, are in agreement with the observed empirical failure envelopes.

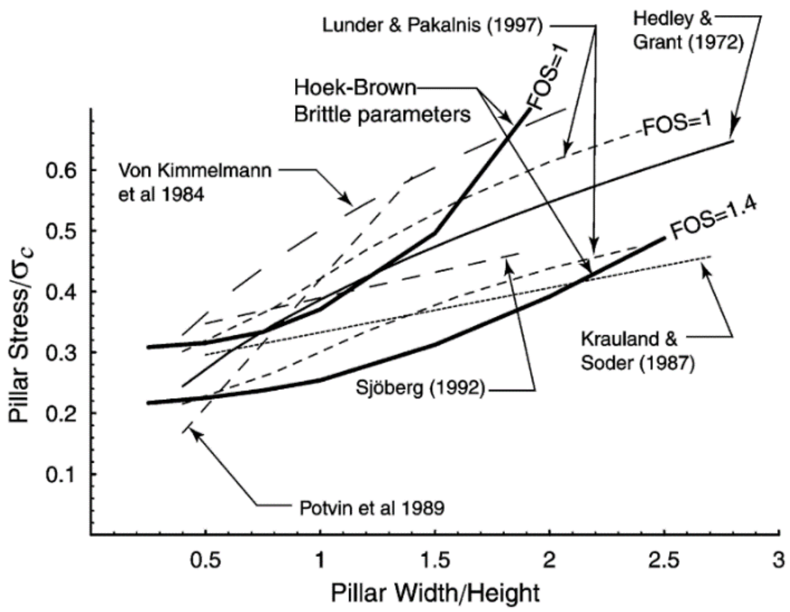


Figure 68: Empirical pillar stability formulas and 2D elastic model using Hoek-Brown brittle parameters (Martin & Maybee, 2000)

Besides the empirical and semi-empirical methodologies mentioned in this chapter, there are much more methods and graphs which are in use today. For this reason, it is important to investigate the underlying databases and check on similarities in regards to the area of application and rock mass quality. If a suitable method is identified, it is further important to calibrate the selected methodology by adding site specific data.

Last but not least, there are several important conclusions that emerged from the development and the analyses of these empirical charts. According to (Martin & Maybee, 2000) almost every investigated ‘failed’ pillar is characterized by a width to height ratio smaller than 2.5. Another important observation concerns the process of failure. The pillar formula developed by Lunder and Pakalnis agrees with other empirical formulas, stating that the failure concerning slender pillars (W/H < 1.0), initiates when 1/3 of the laboratory UCS is reached. In regards to squat pillars (W/H >1.5) the pillar strength was predicted to be slightly higher, showing values between 1/3 to 2/3 of the laboratory UCS. (Martin & Maybee, 2000)

4.3.2.3 Pillar Design – Critical parameters

Similarly, to the stope planning process there are many aspects which need to be considered when designing a stable hard rock pillar. To provide a structured overview, two lists comprising all essential parameters and factors will be presented.

<u>Structural paraments which have to be determined:</u>

- Factor of safety
    - Permanent or temporary pillar
- Shape of the Pillar
    - Rectangular
    - Square
    - Other shape
- Slenderness
    - Width to Height ratio
- Size / dimensions of Pillar
    - Width
    - Height
    - Length

<u>Critical paraments which must be monitored and considered during the design process:</u>

- Parameters and effects related to the Pillar strength
    - Rock mechanical properties of pillar, floor and roof
    - Geological defects (Especially Discontinuities and its inclination)
    - Deformation characteristics (Rock mass stiffness)
    - Shape effect (Slenderness ratio)
    - Confinement effect
    - Scale Effect
- Parameters related to the Pillar load
    - Overburden / Depth
    - Density of adjacent rock and ore
    - Method of stress measurement / calculation
    - Stress redistribution / changes (Consideration of seismic activities and strain and relief patterns through mining activities)

4.3.2.3.1 Parameter description

To highlight the importance of these parameters, various effects and correlations are now explained in more detail. In that context, the impact of the applied width to height ratio as well as the effect of confinement will be discussed first. What should be noted is that all following statements are specifically related to hard rock pillars.

Width to Height ratio and the effect of confinement

Through a series of in-depth studies, which addressed the topic of pillar stability, the width to height ratio was found to be one of the most significant parameters, when it comes to the determination of pillar strength. In that context it was verified that a lower W/H-ratio results in a lower pillar strength and further, to a higher probability of pillar failure. (Esterhuizen et al., 2008) According to (Martin & Maybee, 2000) an important value in this regard is the ratio of 2, as pillar failure is only seldomly observed beyond that value. Another conclusion drawn was that not only the pillar strength is impacted by the W/H-ratio, but also the general variability of the strength. Slender pillars (W/H-ratio < 1.0) for instance, show much more variable strength than wider pillars. However, according to (Esterhuizen, 2007) this could potentially be attributed to the fact that slender pillars are much more susceptible to inclined discontinuities than broader pillars.

An important reason for this behavior could be attributed to an effect, known as confinement. Confinement is a major factor when it comes to the estimation of pillar strength. According to (Potvin et al., 1989) this effect could very well be observed in squat pillars, showing deformations in the open sided pillar walls, but none in the confined central core which is subjected to 3-axial stress conditions. (Potvin et al., 1989) states further, that slender rib pillars on the other hand, are characterized by a non-existing confinement effect, reducing the pillar strength significantly. An interesting question in this context is: At what W/H-ratio does this effect become significant and how can the relation between slenderness ratio and confinement be expressed? According to the research of (Lunder & Pakalnis, 1997) it is possible to estimate the average confinement by expressing the σ_3/σ_1 stress ratio, using the width and height of the pillar. In this regards they came up with the following formula.

$$\frac{\sigma_3}{\sigma_1} = 0.46\left[\log\left(\frac{W}{H} + 0.75\right)\right]^{\frac{1.4}{(W/H)}}$$ **(Lunder & Pakalnis, 1997)**

By using this equation, it was possible to develop a graph, which indicates the progression of the confinement in relation to the W/H-ratio.

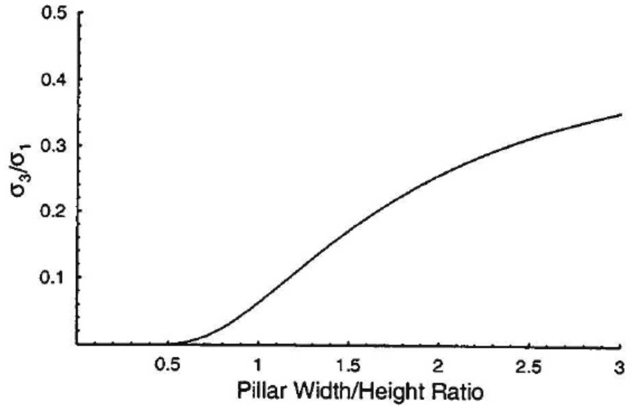


Figure 69: Confinement representation by (Lunder & Pakalnis, 1997)

In that context it was possible to determine that from of a W/H-ratio of 1.0 upwards, the confinement effect increases significantly. Another interesting observation was made by (Maybee, 1999) who stated that the confinement effect, is also dependent on the k-value. He further stated that this factor becomes especially relevant from a W/H-ratio of 1.0 upwards. A diagram which demonstrates this very clearly was created by (Martin & Maybee, 2000) and is shown in the following.

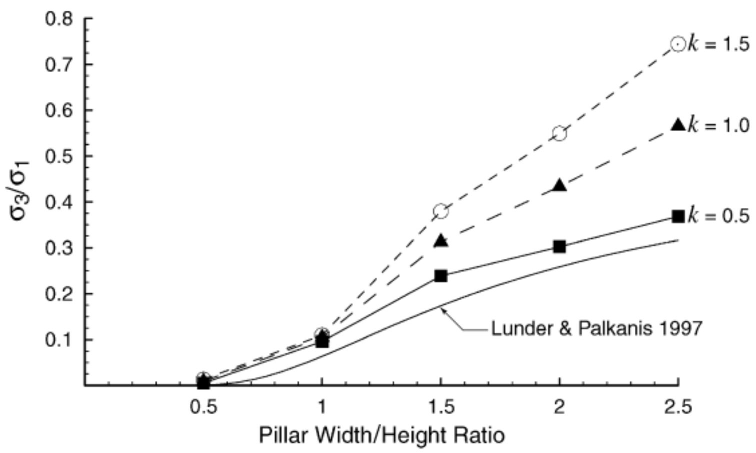


Figure 70: Confinement representation by (Martin & Maybee, 2000) as a function of k

What can be observed is that the confinement effect difference, becomes more relevant with rising W/H-ratio and rising k-value.

As the overall topic concerning “confinement effect” in relation to pillar strength is rather important, there are many more studies which address this topic extensively. The next effect which will be discussed concerns the size aspect of pillars.

Scale Effect

Observing the empirical design graphs from the previous section, it becomes apparent that an efficient way to project the strength of hard rock pillars, is to use the width to height ratio as an indicator. Although this provides a very useful perspective concerning the slenderness, the scale effect is no longer recognized. The scale effect relates to pillars which are characterized by the same slenderness ratio, but differ in the actual size. In that context it was found that pillars showing the same width to height ratio, become weaker with increasing dimensions. An example for this effect was presented by (Oke et al., 2017) who compared the results of several numerical models, which used a slenderness ratio of 0.5. The following graph was the result.

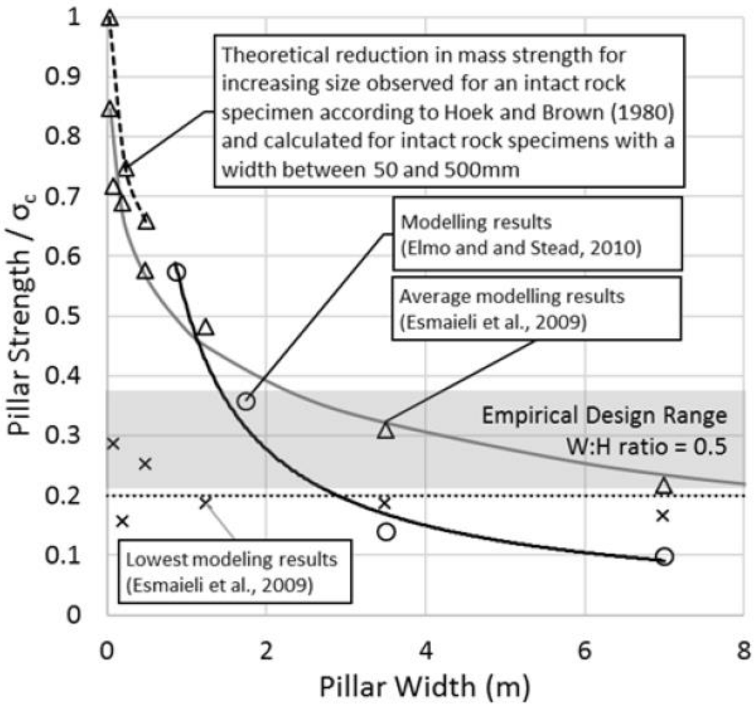


Figure 71: Illustration of scale effect using numerical simulation (Oke et al., 2017)

The results of this numerical simulation indicate, that there is clear tendency of decreasing pillar strength with increasing pillar size. However, what is also noticeable, the curve flattens considerably with increasing dimensions. Furthermore, since this example only refers to one specific slenderness ratio (0.5), there is no information on whether this effect is equally significant for larger ratios.

The next aspect which will be highlighted briefly concerns the load determination.

Pillar load determination

The determination of the pillar load is one of the most important aspects when it comes to the design process of hard rock pillars. Together with the pillar strength, this parameter is decisive for the factor of safety and thus for the actual pillar stability.

In regards to the determination of the pillar load, not only the in-situ stresses are important, but also other influences like mining induced stresses which are often characterized by load and relief patterns. Especially this second point makes an accurate determination of acting stresses rather complicated.

Overall, the methods of load determination differ foremost in accuracy and complexity. In that context one of the most popular and simplistic methods for determining the pillar load is the Tributary Area Theory (TAT). The main idea behind this method is that the load of each pillar, is defined as the vertical rock mass above that pillar and half the area to each adjacent pillar. However, it is important to note that this theory was particularly developed for horizontal tabular deposits characterized by large extensions, in which repetitive similar sized pillars are used. (Lunder, 1983) As sublevel stoping is usually characterized by a rather vertical mining direction as well as specific stoping sequences, the stress situation is somewhat more complicated.

Another method to estimate the pillar stresses is the Coates method developed in 1965. In comparison to the TAT, certain geometrical and rock properties are considered in load determination process. For instance, factors like pillar height, span and depth of mining zone as well as horizontal stresses and modulus of deformation concerning the material are incorporated in the formula. However, this method is only intended for undisturbed long rib pillars located in the center of the mining area. (Coates, 1965) Furthermore, according to (Lunder, 1983) this method appears “impractical”, since the required measurements assume an intact elastic pillar material.

The most accurate way for determining acting stresses in such environments, is the use of numerical analysis. Depending on the problem that needs to be recreated, various numerical solutions like finite difference, boundary element or distinct element can be utilized using 2D or 3D software packages. What should be noted is that each software solution is characterized by specific limitations which must be identified prior application. A more detailed description on numerical modelling is provided in chapter 4.3.1.1.

4.3.3 Barrier Pillar

Similarly, to the previously discussed stope-pillars, barrier pillars will now be examined. In that context some general information will be covered in beginning. Subsequently potential design methods as well as important design parameters and effect are highlighted.

4.3.3.1 Barrier pillar – Overview

Unfortunately, there is only very limited information on the design process of barrier and inter-panel pillars in regards to sublevel stoping. Also, during the research done in chapter 3, only few data was provided by the individual operations. Most information available in regards to the functionality and design process of barrier pillars, refer to room and pillar or longwall mining. This however is no surprise since barrier pillars originally emerged from these mining methods. The lack of information concerning the utilization in stoping mines could indicate that barrier pillars are not applied as frequently in stoping operations as in other methods. In that context, according to (Jager & Ryder, 1999) barrier pillars are not essential for a stoping layout, if the applied in-stope pillars are designed with an 'adequate' factor of safety (~1,6). However, they also stated that these pillars nevertheless are an important form of insurance against unexpected events and are therefore recommended. Especially in depths between 300 and 400 meters, where in-stope pillars have to bear the entire load of the overburden. With that in mind, some important structural differences to other pillar types will now be discussed.

Barrier pillars differ in comparison to stope or rib pillars in a significant structural way. The variable that highlights this better than any other, is the applied width to height ratio. While the value of this factor varies between 0.5 and 2.5 for stope pillars, barrier and inter-panel pillars are usually designed with much larger ratios, sometimes greater than 10. The reason for this has to do with the effect of confinement, which was described in the previous section. According to (Stacey & Page, 1986) this desired effect becomes even more pronounced when the width to height ratio exceeds a value of 5. All this indicates that barrier pillars should be designed with a rather large W/H-ratio, to fulfill the purpose of ensuring regional safety and stability.

The following section provides more details on the design aspects of barrier pillars.

4.3.3.2 Barrier pillar – Design Methods

Since barrier pillars have to meet other requirements than stope pillars, the design formulas mentioned in the last chapter, are not directly intended for this pillar type. According to (Jager & Ryder, 1999) barrier pillars need to be 'squat' to provide the required level of longevity and strength. A formula which is very popular and serves this purpose, was developed by (Salamon, 1982) and is known as the squat pillar formula. This design method will now be highlighted.

Squat pillar formula

The squat pillar formula also known as "Salamon' s extended formula", is the extension of a design method which was originally developed for coal pillars by (Salamon & Munro, 1967). It can be used to design barrier pillars in hard rock mines, assuming that the selected width to height ratio is greater than 5. The developed formula reads as follows.

$$\sigma_s = K\frac{R_0^b}{V^a} \, x \, \frac{b}{\varepsilon} \, x \, \frac{R}{R_0}^{\varepsilon} - 1 \; + 1$$

$$V = W_{eff}^2 \, x \, h$$

$$R = \frac{W_{eff}}{H}$$

K ... design rock mass strength [MPa]
R_o ... critical width to height ratio ($R_o = 5$)
V ... pillar volume [m³]
a ... determined by Madden, 1991 (a = 0,0667)
b ... determined by Madden, 1991 (b = 0,5933)
ε ... rate of strength increase by Madden, 1998 ($\varepsilon = 2.5$)
R ... effective width to height ratio
W_{eff} ... effective width [m]

Figure 72: Squat pillar formula (Salamon, 1982)

What should be noted, the parameter 'K' is assumed to be 35% of the UCS concerning the pillar material.

Further design aspects

Another important topic which is related to the design process of barrier pillars, is the applied span width between them. According to (Ryder et al., 1995) this span can be estimated by using a simple design formula, which they stated as follows.

$$\frac{H}{L} > 4$$

L ... Span
H ... Depth

Although this design method is supported by many theoretical results and is based on various criteria, the overall estimation should be interpreted as rather conservative. (Jager & Ryder, 1999) further stated, that in practice numerous mines applied one half of the depth for the span instead of one quarter, which has also proven to be successful, considering that the applied in-stope pillars were always designed sufficiently.

4.3.3.3 Barrier pillar – Critical parameters

To get an overview which aspects are important concerning the design process of barrier pillars, a summary is made in the following. It should be noted that this section only features parameters and effects that are directly related to barrier pillars. General aspects, like the effect of confinement can be found in the previous chapter.

<u>Structural paraments which have to be determined:</u>

- Factor of safety (according to Hedley 1976, SF ~ 4,5)
- Width to Height ratio
- Size / dimensions of Pillar
  - Width
  - Height
  - Length
- Span between barrier pillars

<u>Critical paraments and aspects which must be monitored during the design process:</u>

- All parameters and general effects which are related to the overall pillar strength are listed and described in chapter 4.3.2.3. These also apply to barrier pillars!
- Overburden / Depth
- Foundation strength
- Number and FOS of applied in-stope pillars between barrier pillars
- Size of tensile zones
- Deformation characteristics (Rock mass stiffness)

4.3.3.3.1 Parameter description

An important issue that is usually linked to the design process of massive support elements like regional / barrier pillars, is the valuation of the foundation strength. For this reason, this topic will be examined first. The second aspect that will be highlighted, concerns the stresses which potentially act on barrier pillars, after an in-stope pillar failure.

Foundation strength / failure

In addition to the pillar strength, the bearing capacity of the foundation can be considered as equal importance, when it comes to the design process of barrier pillars. Even if a pillar seems to be designed as "indestructible", indicating a W/H ratio greater than 10, it can only support as much load as the strength of the floor or roof allows. In that context, according to (Stacey & Page, 1986) barrier pillars are more likely to fail due to a foundation failure than to pillar failure, if the pillar is characterized by a W/H-ratio greater than 7. What should be noted, this statement assumes that both structures consist of the same material. The risk of foundation failure should be especially taken into consideration in case different (weaker) materials are present. (Stacey & Page, 1986) This fact was also confirmed by (Jager & Ryder, 1999) who stated that the foundation strength is especially important, when the width to height ratio exceeds a value of 10.

In this regard one of the most famous formulas to determine the foundation strength was developed by Terzaghi in 1943 and is illustrated in the following figure.

$$q_u = cN_c + qN_q + \gamma B_p N_\gamma$$

$$N_\gamma = 1.5\ \ N_q - 1\ \ \tan\phi$$

$$N_c = \ \ N_q - 1\ \ \cot\phi$$

$$N_q = e^{\pi \tan\phi} \tan^2(\frac{\pi}{4} + \frac{\phi}{2})$$

q_u ... foundation strength [Mpa]

c ... cohesion of foundation rock

B_p ... foundation depth

ϕ ... internal friction angle of foundation rock

N_c, N_q, N_γ ... bearing capacity factors

Figure 73: Formula for foundation strength (Terzaghi, 1943)

A further noteworthy design criterion concerning barrier pillars was developed by (Jager & Ryder, 1999) and is highlighted in the following.

Design APS (Average Pillar Stress) ≤ f_a x UCS of the weakest foundation strata

Related to that criterion they stated that the lowest UCS value should be used, involving hanging wall as well as footwall. Furthermore, friction and other factors like the presence of weak layers are not considered in this design approach. The empirical factor **f_a** can (according to COMRO, 1988) be set to 2.5. However (Hedley et al., 1997) stated that this variable can also reach lower values, up to a minimum of 1.6.

Stress related effects

In order to point out the stress related effects on barrier pillars caused by an in-stope pillar failure, the next figure presents certain scenarios. It should be noted that these examples are based on a flat lying deposit situated at a depth of 900 meters. In regards to the layout, the stopes indicate a height of 21 meters and a width of 7 meters. The pillars separating the stopes measure a width of 7 meters. A further assumption was that the residual strength of all failed pillars is zero. (Wanger et al., 2016)

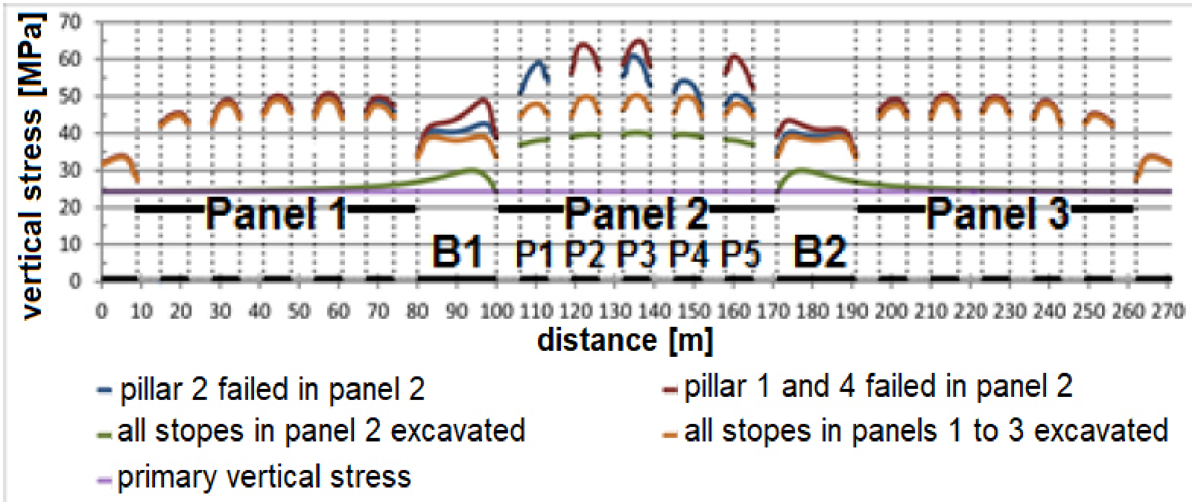


Figure 74: Effect of Barrier pillars (Wagner et al., 2016)

Observing this figure, the violet line represents the primary vertical stresses and consequently provides a useful basis for the comparison of the individual cases. The yellow line on the other hand indicates a non-failure example, representing all acting vertical stresses after a complete excavation in all panels. In other words, a best cases scenario. Observing the blue line, it is apparent that one pillar failure (pillar Nr. 2 in panel Nr. 2) has a significant effect on the neighboring pillars and a moderate effect on all further pillars in that panel. What can be recognized, the inter-panel pillars are only slightly affected by this failure and there is almost no impact on the pillars located in the neighboring panels. A more extreme example is represented by the red line, that indicates two pillar failures in panel Nr. 2. In this scenario the stress increase concerning the inter-panel pillars, seem to be fairly larger, as a peak recognizable. However, it is also apparent that this stress peak decreases significantly across the width of the barrier. What is interesting, the in-stope pillars located in the neighboring panels, are (similarly to the blue case) not significantly affected in terms of stresses. These examples illustrate the importance of inter-panel barriers. (Wanger et al., 2016)

4.3.4 Horizontal pillars (Sill and Crown Pillar)

The last pillar type which is going to be highlighted in this chapter is the horizontal pillar. As with the previous elements, some general information concerning application and structure will be presented first. Subsequently various design methods are examined in more detail. Last but not least, design parameters which are important for the overall design process are listed.

4.3.4.1 Horizontal pillar - Overview

Horizontal pillars are support elements that play an essential role in the extraction of tabular vertical deposits and are therefore embedded in the standard layout of sublevel stoping and cut & fill operations. The main structural difference to “normal” pillars (stope and rib pillars) is the orientation of this pillar type. For this reason, sill and crown pillars are also labeled as horizontal pillars. In that context, according to (Maybee, 1999) the designation of the pillar dimensions is usually adjusted, according to the maximum stresses. As sill pillars, for example, are characterized by high horizontal stresses, the pillar length is usually measured in the horizontal direction, while the width (thickness) is measured vertically. The general thickness of horizontal pillars can vary to a large scale, since this geometrical parameter mostly depends on the predominant rock mass conditions. The primary objective concerning dimensioning is to establish a structure that reliably maintains ground stability, but on the other hand is not over-dimensioned due to economic reasons. Some example values concerning the thickness of sill pillars are researched in chapter 3 and were found to be between 7 and 30 meters.

What should be noted, is that the designation “sill pillar” can sometimes have different meanings. For example, most definitions state that only the portion that is located above the level drive (which is situated in the overall horizontal pillar-structure) is designated as a sill pillar. The portion below the level drive is then labeled as a crown pillar. However, there also papers which refer to the entire horizontal pillar structure as sill pillar. (Hemant et al., 2017)

The following figure illustrates the division of a horizontal pillar.

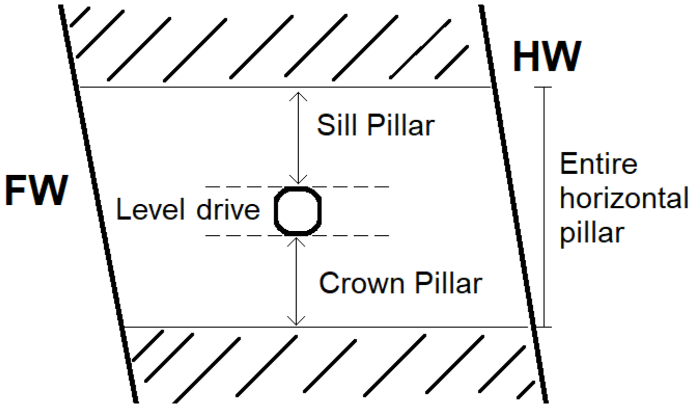


Figure 75: Illustration of a horizontal pillar

Sill and crown pillars are overall less well researched, concerning the determination of strength and stability, in contrast to stope pillars. The main reason for this lies in the lower amount of data that was collected in the past, leading to less empirical research. However, since horizontal pillars have proven to be very valuable in terms of safety and stability much more research is conducted nowadays. (Chen et al., 2021) The following section provides a more detailed insight into various design approaches.

4.3.4.2 Horizontal pillar – Design methods

As mentioned in the previous section most empirical formulas were developed to estimate the stability of vertical oriented pillars. However, there are certain approaches that are also relevant for horizontal pillars. To get an overview some methods will now be discussed.

4.3.4.2.1 Empirical approaches

The first method that will be addressed briefly, is the empirical design graph developed by (Lunder & Pakalnis, 1997). This database, which comprises 178 case files, is primarily used to design rib pillars (see chapter "4.3.2.2"). However, according to (Martin & Maybee, 2000) "many" of the researched cases are rib or sill pillars. These circumstances would support the argument that an application on sill pillars is also reasonable. However, there is no exact clarification concerning how many cases in the database are determined sill pillars. Furthermore, it is also evident that the large majority of cases are rib pillars. The utilization of this method in regards to sill pillars is therefore not entirely clear.

Another approach to estimate the stability of horizontal pillars from an empirical standpoint, was made by (Kersten, 1984). He applied the revised formula from (Hedley & Grant, 1972) and also used the 'effective width' developed by (Wagner, 1980) to calculate the strength of the pillars. A further and final modification of the formula served as a compensation for the higher strength values, which were measured in the laboratory. This modification was performed after (Wagner et al., 1979). The final formula can be seen in the following.

$$\text{Pillar strength} = \left[\frac{(w)}{(wo)}\, 0{,}5 \,/\, \frac{h}{ho}\, 0{,}75\right] \sigma c \;\; [\text{kPa}] \qquad \text{(Kersten, 1984)}$$

w = pillar width
wo = specimen width
h = pillar height
ho = specimen height
σc = uniaxial compressive strength

Figure 76: Calculation of horizontal pillar strength after (Kersten, 1984)

Using this formula in combination with the acting stresses, it was possible to determine the strength as well as the FOS of sill and crown pillars in relation to the combined pillar width. The following two diagrams display the results.

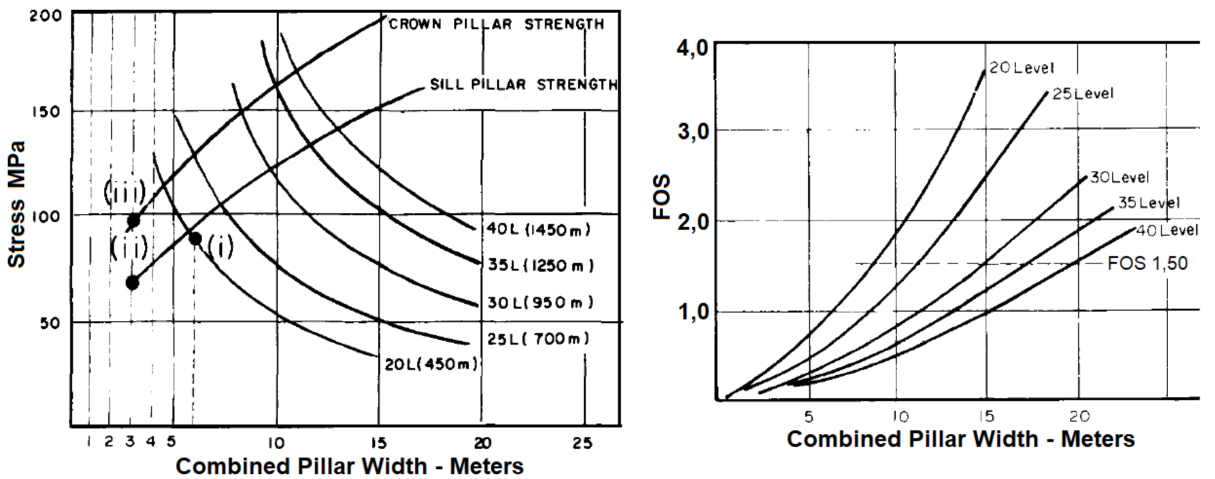


Figure 77: Stresses, pillar strength and FOS contrasted to the combined width (Kersten, 1984)

Through back analyzing (Kersten, 1984) concluded that the results were overall quite suitable and that this approach could potentially be applied to other mines showing similar setting. However, according to (Sjöberg, 1993) the limitation regarding available data, led to mixed results concerning the success of this design methods.

4.3.4.2.2 Numerical approaches

Besides the use of empirical methods, numerical modeling has become the state-of-the-art when it comes to the design process of pillars. This also applies to horizontal orientated sill and crown pillars. The following two examples demonstrate how numerical models can be beneficial for stability analyzes of horizontal pillars.

Approach by (Hemant et al., 2017)

A well-documented example which makes use of numerical modeling, was provided by (Hemant et al., 2017) and discusses the crown pillar stability in relation to the applied thickness. The deposit that was analyzed in this regard is an Indian metal mine, located at a depth between 600 and 1000 meters.

In order to investigate the stability of crown pillars, (Hemant et al., 2017) developed 108 non-linear numerical models, using the Drucker-Prager failure criterion. The results were then analyzed in terms of displacement, stresses as well as failure zones around excavations and pillars. These 108 finite element models covered 3 different orebody modules, 4 different crown and sill pillar thicknesses, 3 variations of orebody RMR/GSI and 3 different UCS values. The software that was used to develop the solid model and the finite element meshes is known as ANSYS. According to (Hemant et al., 2017), one of the main difficulty's was to establish a relation between the factor of safety and the applied rock mechanical input parameters such as the GSI, E, UCS and others. An important aspect the model highlighted, the extend of the yielding zone which emerges around an excavation, is directly influenced by the GSI of the rock mass. This developed multivariate regression model was later on used to create further design charts for crown pillars in different mining conditions.

Approach by (Sjöberg, 1993)

Another ambitious paper, which addresses the task of designing sill pillars and stope roofs in hard rock mines, was written by (Sjöberg, 1993). The approach Sjöberg took is a combination of research, weighing of different design criteria's as well as making use of numerical modelling.

The developed methodology, comprises the following four basic steps:

Step 1 involves a general investigation concerning the stability of stope and sill pillars. Thereby a documentation of the geometry, mining sequence as well as all failure modes should be compiled.

In step 2 all variations in the local geology are researched. The primary objective is to identify whether there is a correlation between the observed failure modes and the local geology.

During the third step, the main focus lies in the development of various tools with the aim to analyze the base cases more closely. This includes simple stress analysis using basic criteria and modelling of the researched failure modes. The main objective hereby is to create a model, which can reproduce the observed failures.

The fourth step deals with the creation of guidelines and the choice of input parameter which are used in the numerical model. (Sjöberg, 1993)

Through this combination it is possible to design sill pillars and stope roofs for various environments. However, compared to other methods this complex methodology also needs much more effort than usual empirical approaches.

4.3.4.3 Horizontal pillar – Critical parameters

To summarize all aspects that are essential in the planning process of horizontal pillars, a list comprising all parameters is made in the following. It should be noted that this section likewise only features parameters that are directly related to horizontal pillars. All general factors in regards to pillars, can be found in chapter "4.3.2".

<u>Structural paraments which have to be determined:</u>

- General structure of horizontal pillars
  - Sill pillar
  - Crown pillar
- Factor of safety
  - Temporary / Permanent pillar
- Width to Height ratio
- Size / dimensions of Pillar
  - Individual thickness of horizontal pillar sections
  - Combined pillar thickness
  - Pillar Length
- Sill and Crown pillar spacing and placement

<u>Critical paraments and aspects which must be monitored during the design process:</u>

- All parameters and general effects which are related to the overall pillar strength are listed and described in chapter 4.3.2.3. These also apply to horizontal pillars!
- Overburden / Depth
- Usually Rectangular shaped
- Horizontal to vertical stress ratio (High horizontal stresses)
- Deformation characteristics (Rock mass stiffness)
- Monitoring of hanging wall outbreaks with increasing depth
- Potential pillar extraction process

4.3.5 Mining / Stope Sequence

In contrast to the previous chapters, the next element to be discussed is a procedure rather than a physical object. However, as the stope sequence is a very defining and important aspect of sublevel stoping, all possibilities concerning the utilization will be highlighted. In that context, general information including researched data (related to the stoping mines in chapter 3), is provided in the overview. Following on from this, the most popular stope patterns will be presented. Last but not least critical parameters and effect, which are important for the decision of a sequence, will be highlighted.

4.3.5.1 Mining / Stope Sequence – Overview

In stoping operations, the excavation process usually takes place in multiple areas (stopes), sometimes simultaneously. To maintain a solid degree of structural stability throughout the entire mine, two core aspects concerning the extraction process have to be defined. These aspects are the mining direction and the stope sequence. Together these two factors are responsible for the occurring change in the structure of the panel layout. Since a definition for the stope sequence has already been provided in chapter 4.1.8, a definition for the mining direction is now added.

Mining direction

The mining direction describes the overall direction in which a panel is going to be excavated. The two main possibilities which exist in this regard are either "bottom-up" or "top-down" mining. Both directions can be applied with the utilization of sublevel stoping. However, "bottom-up" mining is nowadays the more popular option. A fundamental reason why this is the case, has to do with stress management. As the actual mining level proceeds upwards, the induced stress rises. At the same time, however, shallower levels, showing lower pre-mining stresses are reached. This strategy has become especially important for "deep mining" operations. (Potvin, 2000)

Stope Sequence examples

To get an overview which sequences are commonly used, in regards to European and Canadian sublevel stoping operations, the following table presents various examples.

Mining - Stope Sequences		
Mine	**Description**	**Source**
Garpenberg	Use of primary / secondary stopes - Pyramid shape sequence - Six lvl's of stopes, each 25m high.	(Souley, 2018)
Lappberget	The idea behind the sequencing is to maintain the sawtooth pyramid shape as much as possible.	(V. Koppen, 2008)
Agnico - Kittilä	Basis of the stope sequencing is bottom up primary-secondary sequencing.	(Tommila, 2014)
	Mining starts from the middle of the sill level and gradually expands both horizontally and vertically.	
	Secondary stopes can only be mined after all primary stopes around them are mined and backfilled.	
Outokumpu's Kemi	Use of primary and secondary stopes - Mining is done bottom up.	(Mindat.org)
	Primary stopes are extracted one or two levels above the Secondary stopes.	
LM - Zinkgruvan-Burkland	Transverse bench and fill stoping is used with a sequence of primary and secondary stopes.	(Daffern,2017)
	Sill pillars are at -965m, -800m, -650m, and -450m levels; Have been left to separate mining areas.	
	The lower levels of Burkland are mined by a top down mining sequence (Underhand Bench and Fill)	
LM - Neves-Corvo	All OBF (Optimised bench & fill) stopes are mined in a bottom up sequence.	(Newall, 2017)
	Primary stopes on a new level are never more than one level ahead of the secondary stopes below.	
	~10m thick sill pillars are used between up-dip mining panels.	
Pyhäsalmi Mine	Benching with delayed backfill in primary/secondary or sometimes horizontally retreating sequence.	(First Quantum, 2020)
	Some areas are mined by horizontal retreat due to connecting development or location in sill pillars.	
	Primary stopes are mined using sublevel stoping over a height of 50m.	(Hustrulid & Bullock, 2001)
	Secondary stopes are split into two benches, each 25 meters high.	
Mittersill Mine	Mining method is adjusted to circumstances. Sublevel caving is used in deeper massive areas.	(Gaul, 2008)
	Sublevel stoping is used in transvers as well as in longitudinal form. Excavated areas are backfilled.	
Breitenau Mine	Mining method was changed in deeper areas from post pillar mining to a "cut and fill" method.	(Wagner, 2015)
	Stopes are excavated and later backfilled (paste fill); 7m wide pillars are left to separate stopes.	
	After backfilling, the adjacent stope is excavated. Only one stope per panel is mined at any time.	
	Between mining panels of four to six stopes wide barrier pillars are left in place.	
William Mine	Triangular retreat shape with primary and secondary stope arrangement.	(Hustrulid & Bullock, 2001)
	Prim. stopes are mined and filled two vertical lifts before, mining of the Sec. stope in-between starts	
Brunswick Mine	Used a primary / secondary stope pattern with delayed backfill in the beginning.	(Hustrulid & Bullock, 2001)
	Switch later to Pyramidal pillarless open-stope mining with rapid paste backfilling.	
	Used sill pillers had a thickness of 30 meters.	

Figure 78: Stoping Sequences of researched Mines

Analyzing this data, it becomes apparent that one of the main sequences which is applied nowadays, is the primary - secondary stope pattern with subsequent backfill. Thereby, a triangular / pyramidal structure has proven to be very effective. In regards to the implementation of the sequences, primary stopes were always mined 1 to 3 levels ahead of the neighboring secondary stopes, resulting in a sawtooth shaped structure. One reason why this sequence is widely adopted, can be attributed to the rather good flexibility, as it allows the mining of several stopes simultaneously. But more on this topic in the next section. To get a visualization to some of these examples, the following figure displays various sequences.

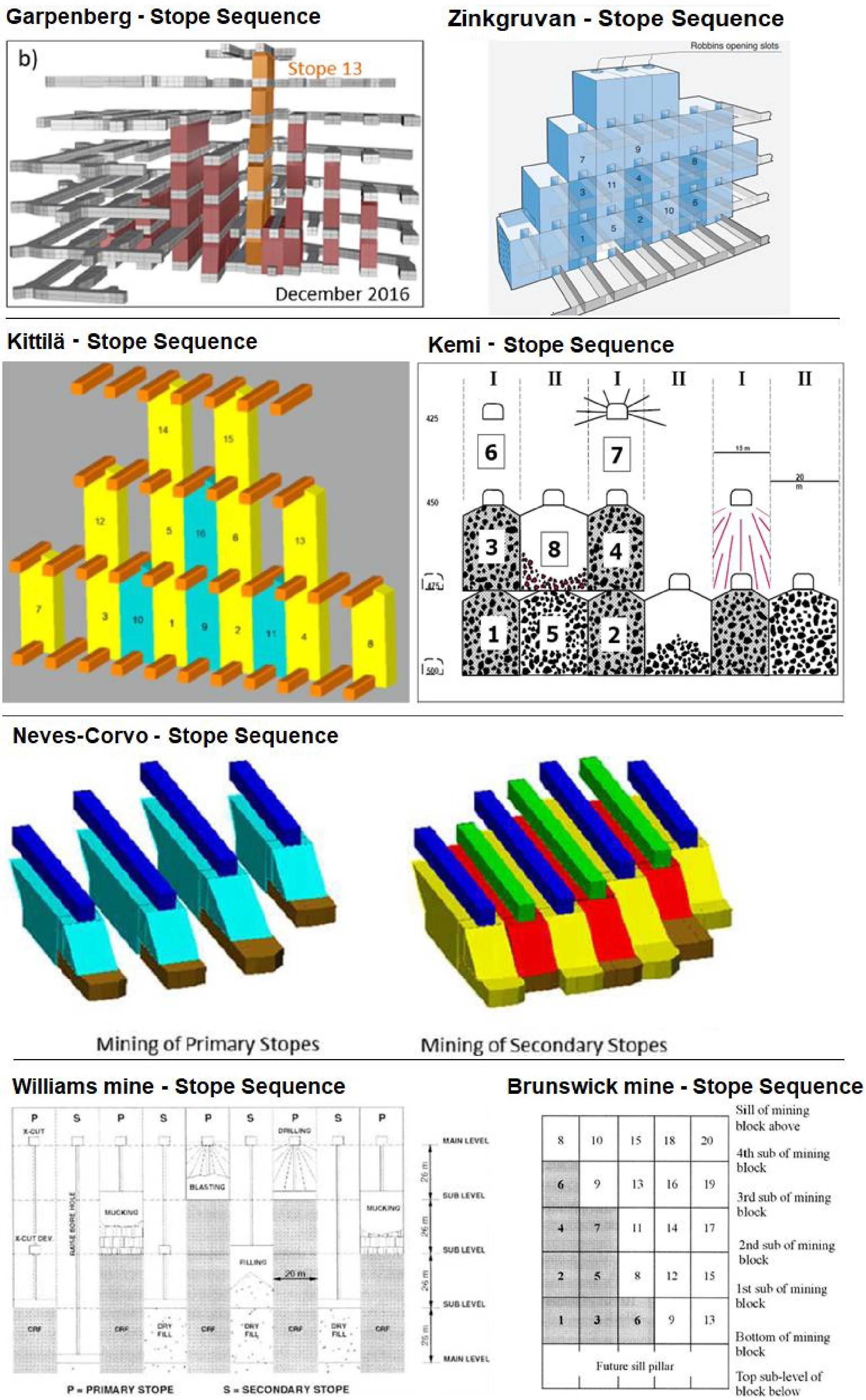


Figure 79: Stope Sequences of researched mines (Santis et al., 2020) (Fernberg et al., 2007) (Tommila, 2014) (Rikberg, 2019) (Newall et al., 2017) (Hustrulid & Bullock, 2001)

4.3.5.2 Mining / Stope Sequence – Description of Sequence patterns

This section provides an overview of the most popular stope sequence pattern, that can be applied in sublevel stoping operations. The first sequence that will be highlighted is the Primary – Secondary pattern.

4.3.5.2.1 Primary - Secondary Stope Pattern

The Primary - Secondary pattern is a commonly used sequence, which subdivides all stopes into two major groups. Thereby one half of the stopes are assigned to be primary, while the other half will be designated as secondary. In general, this sequence is characterized by high flexibility, as it is not exactly determined which stope has to be extracted next. Reviewing the examples from the last section it can be observed that there are significant differences in the excavation order, although the underlying stope sequence is the same. The overall structures which can be realized during the excavation of a panel, can therefore be quite different. A popular approach however, is to create a saw tooth pattern with a more or less triangular structure. The starting point is usually located at the central bottom of the panel. In regards to the utilized backfill, secondary stopes are most often filled with rockfill, which offers a significant potential for cost savings. Primary stopes on the other hand are usually filled with paste fill. The biggest disadvantage this pattern entails, is the constant redistribution of stresses to secondary stopes / pillars, resulting in a high stress concentration in specific areas. The primary - secondary pattern is highlighted in the following figure.

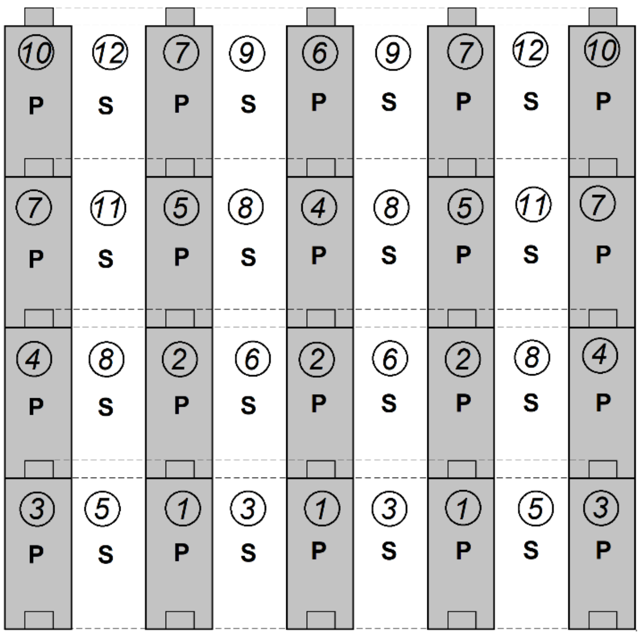


Figure 80: Primary – Secondary Stope Sequence

4.3.5.2.2 Primary - Secondary - Tertiary Patterns

This sequence also known as the “1-4-7 sequence”, includes an additional tertiary stope in the excavation pattern. The numbers 1, 4 and 7 refer to the numbering of primary stopes on one level. The stopes located to the right of primary stopes (highlighted by the numbers 2, 5 and 8) are designated as secondary stopes. Last but not least the stopes numbered with 3, 6 and 9 (located between secondary and primary stopes) are defined as tertiary stopes. What should be noted, through this assignment, secondary stopes are exposed to one fill wall, while tertiary stopes are located between two fill walls. This leads to the necessity that primary as well as secondary stopes have to be filled with consolidated backfill, what in turn increases the general costs. The overall advantage of this pattern lies in the better flexibility and the fact that more stopes can be excavated simultaneously. The result is better quality control and a potentially higher production. However, another problem that this pattern entails is the worse handling of stress redistributions and stress concentration in specific areas.

A related stoping sequence is the “1-5-9 pattern”. Within this sequence the stopes 1, 5 and 9 as well as the stopes 3, 7 and 11 are defined as primary stopes, having pillars on both sides. All stopes which are even numbered are defined as tertiary stopes. The 1, 5, 9 stopes are usually the leading primary stopes and kept ahead of the 3, 7, 11 stopes. The tertiary stopes are usually mined last on the respective levels, indicating that the 3, 7, 11 stopes are already one to two lifts ahead. An illustration of both sequences can be seen in the following figure.

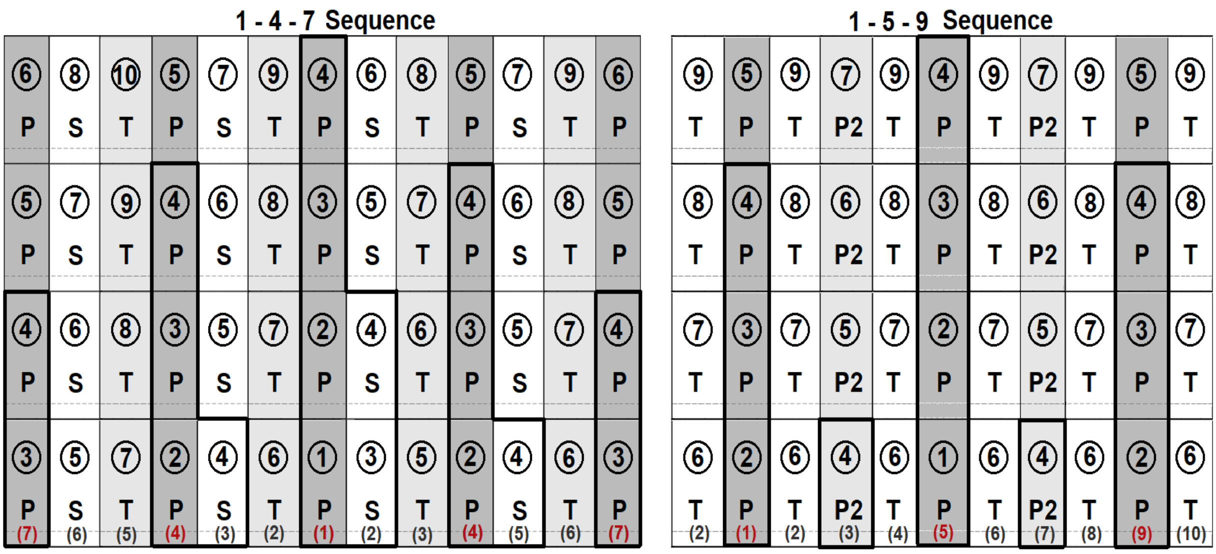


Figure 81: Primary – Secondary – Tertiary Sequences

4.3.5.2.3 Pillarless center out Pattern (Only Primary stopes)

In this sequence every stope in a panel is defined as a primary stope and only one stope is mined at any given time. The structure which is created by applying this sequence, resembles a pyramid. The overall disadvantages this sequence entails is on the one hand, that consolidated backfill has to be used for every stope and on the other hand, that it is not possible to enhance the production by excavating multiple stopes at the same time. In other words, the production as well as the flexibility is quite limited. However, since there are no pillars used in this sequence, the stress problematic which can potentially occur in secondary and tertiary pillars is eliminated. For this reason, this method is primarily applied in difficult rock mass conditions.

4.3.5.3 Mining Sequence – Critical parameters

In this section all parameters and aspects, which have to be considered in the selection process of a stoping sequence, will be listed. To further display the interconnection of all parameters, an example illustrating the implications of certain sequences is presented. Last but not least the issue of stress redistribution will be highlighted briefly.

<u>Critical paraments and aspects which must be considered:</u>

- Economic aspects
  - Annual production
  - Quality control
  - Infrastructural costs
- Rock mechanical aspects
  - Rock mass quality
  - Rock parameters
  - Discontinuities and joint structures
  - Elastic capacity
- Stress environment
  - Pre-mining stresses
  - Active stresses on stope pillars
  - Stress redistributions
  - Seismic activities

Interrelation of parameters (Brunswick mine)

An example which illustrates the interrelation of these parameters quite well, is the problematic situation which occurred in the Brunswick mine. The initial issue were the increasingly worsening rock mass conditions as mining progressed. At some point numerous secondary pillars reached a critical level in regards to stability, which lead to the necessity to change the mining sequence. Up to this stage, a primary - secondary stope sequence was applied. The final conclusion after analyzing the situation was that a pyramidal pillarless mining sequence has to be utilized, as the stresses could not be handled through secondary stopes. This pattern was the only alternative to push the ground stresses towards the abutment, without risking multiple failures. Although this strategy worked very well, the mine had to sacrifice flexibility and productivity in regards to this sequence, since only two stopes can be mined simultaneously. Furthermore, the costs concerning of the infrastructural development changed as well, since more levels have to be operable for this sequence. (Hustrulid & Bullock, 2001)

Stress redistribution (Garpenberg mine)

In this section the stress related impact, of a primary - secondary stope sequence will be highlighted. The example used for this illustration is the Garpenberg (Lapperget) mine, which was investigated by (Van Koppen, 2008) in this regard.

Van Koppen stated that the ongoing stress redistribution, caused by the extraction of stopes, can very well be observed in pillars located in the footwall area (between footwall drift and stopes), as a gradual deterioration is clearly visible. Since these pillars were also mechanically scaled, this deterioration has to be induced by stress changes, for the most part. He further stated that this effect was particularly noticeable at the corners to the footwall drift. The pillars which were most effected in this regard, were located in areas in which the secondary stopes were mined. (Van Koppen, 2008)

These observations were also confirmed by conducted measurements, which will be presented shortly. First however, an illustration concerning the different stages of the used mining sequence.

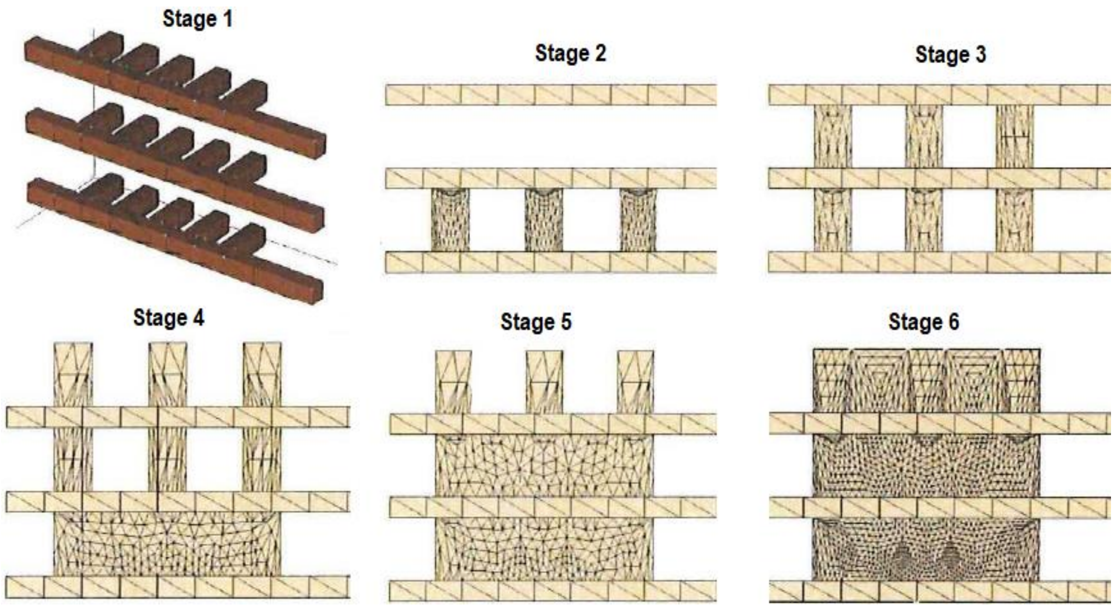


Figure 82: Stages of stress change (Rocscience Inc.,2003)

What can be observed is that primary stopes are extracted from stage 2 to stage 4, while secondary stopes are mined from stage 4 to stage 6. The level difference used between primary and secondary stopes seems to be between two and three levels.

To get an overview regarding the stress changes in these different stages, the results of the performed measurements (in the pillars located between footwall drift and stopes) will now be presented. What should be noted, all measurements were performed on the upper half of the middle row pillars. (Van Koppen, 2008)

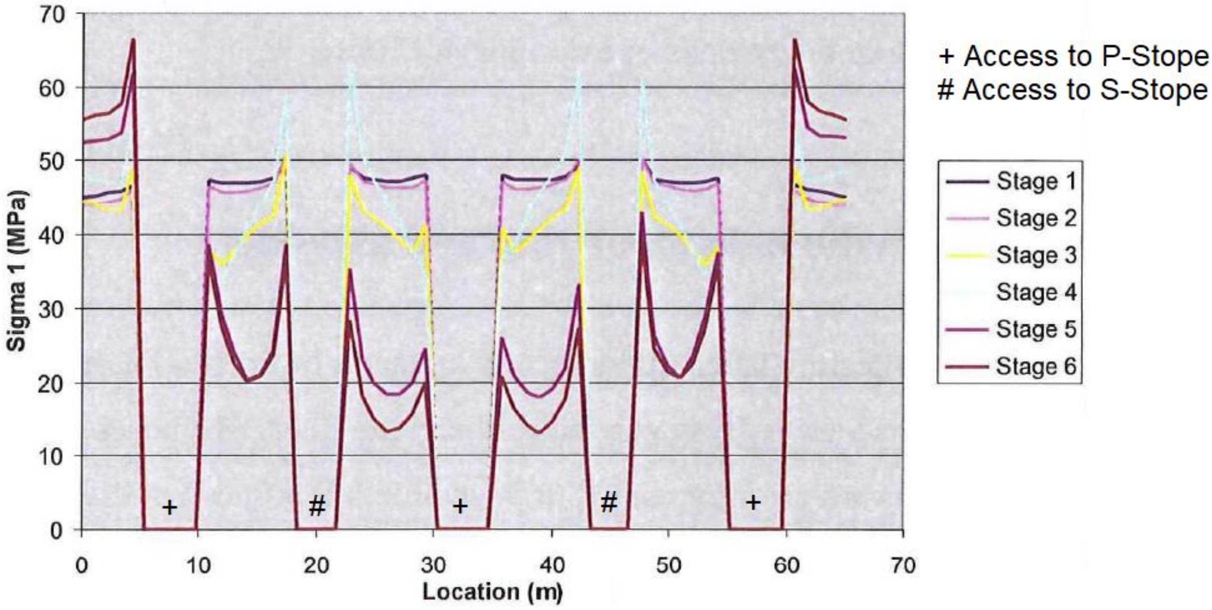


Figure 83: Stress changes - Middle row pillars (Van Koppen, 2008)

Analyzing this diagram, it can be seen that due to the removal of primary stopes (Stage 1 to Stage 4), the stresses are redistributed, resulting in a stress increase near the access drifts to secondary stopes and a stress decrease near the access drifts to primary stopes. Observing further (Stage 4 to stage 6), the effects caused by the removal of secondary stopes becomes visible. The part of the pillars which is adjacent to the cross-cuts leading to the secondary stopes, experience a drastic stress relive in these stages. Depending on the pillar position the stresses near the access drifts to primary stopes either continue to decrease or remain roughly the same.

This example demonstrates the complexity of underground stress management and that certain parts of pillars are loaded and relieved differently depending on their location and the applied sequence. An essential parameter in this regard is the elastic capacity of the rock, which describes the response to a linear elastic deformation. If a certain value is exceeded, irreversible deformation would be the outcome. For this reason, the behavior of the rock mass should be observed carefully when it comes to stress redistributions. (Van Koppen, 2008)

4.3.6 Secondary development

In this chapter the secondary development including the stope preparation will be the main topic of discussion. The first section will highlight the general aspects concerning all major procedures and subparts which are related to the secondary development and the stope preparation process. This will be done in reference to the investigated mines of chapter 3. Subsequently the differences as well as the similarities of the used developments will be analyzed. In this regard, various illustrations concerning the applied design will be presented. Last but not least, critical design parameters which have to be determined and monitored will be listed.

4.3.6.1 Secondary development - Overview

In order to analyze the current state-of-the-art regarding the secondary development, a table (Figure 84) containing real-world data (from mines in chapter 3), has been compiled. This table comprises information about various procedures and subparts concerning stope-access, -preparation and -extraction. Analyzing this data, the following aspects could be identified.

What becomes apparent, all mentioned stopes are characterized by access drifts, which were driven into the ore from previously established footwall drifts. These footwall drifts are essential for logistic and transportation purposes and usually merge into parts of the primary development. In other words, they form the backbone of the secondary development. A further observation indicates that at least two working levels (One over- and one undercut) are utilized to prepare and extract a stope. Depending on the stope orientation, these working drifts are either developed perpendicular to the footwall drift (transverse stoping) or parallel to the footwall drift (longitudinal stoping). The function of these drifts, which are usually created in the horizontal center of the stope, can slightly differ from mine to mine. In most cases however the overcut is used for drilling, blasting and backfilling, while the undercut is primarily used for mucking purposes. To avoid unnecessary drift development, it is possible to reuse the overcut in later stages as an undercut for the stope above. This is done in the Neves-Corvo and Breitenau mine, for instance.

After the development and installation of the needed drifts and support, the next step lies in the creation of an initial opening. In transverse stoping this opening is usually created at the “end” (near the HW) of a stope. This space is necessary for the fragmented material which is generated by the first blast. Overall, there are several possibilities to accomplish this task. For example, the slot can be created via box-hole-raise from the bottom drift, via drop raise or raise boring from an overcut or via other techniques like the Alimak raise method. After the initial slot has been established, it is possible to enlarge it to the full width of the stope if needed. Following on from this, vertical boreholes (often arranged in ring pattern) are drilled and blasted from the specified level (overcut, undercut or main level). The applied direction in this regard is usually on retreat.

The following table provides the collected information, concerning used secondary developments and related stope preparation processes, of the individual mines.

Mine	Extraction / Stope preperation	Source
Garpenberg	Ore is mined in layers between two drifts (perpendicular to the footwall drift) vertically 25m apart.	Högnäs, 2018
	Access is created to the stopes on the bottom; Box hole raises are used to create a free face;	
	After box hole is created, rings are drilled, charged and blasted; Mucking is done in bottom drifts;	V. Koppen, 2008
	Longhole drilling and charging equipment is capable of dealing with downholes and uppers.	
Agnico - Kittilä	From the footwall drifts, overcuts and undercuts are driven into the ore, perpendicular to strike.	Tommila, 2014
	Overcut is used for drilling, charging and backfilling while undercut is used for mucking.	
	Draw points are systematically cable bolted which should decrease dilution from FW undercut.	
	The HW undercut is shotcreted, overcuts are not reinforced.	
Outokumpu Kemi	Drifts for the primary stopes are developed laterally from the footwall through the ore zone.	Fernberg, 2007
	Uphole and downhole drilling methods were tested; Downhole production drilling was selected	
	Uphole drilling is 30% more efficient, but due to poor rock mass conditions it would be to unsafe;	
Zinkgruvan - Burkland	Stope access is typically developed in the footwall from the ramp system (5 x 5m size);	Daffern, 2017
	Stope accesses are developed on the upper horizon for drilling and on the lower lvl for mucking	
	Slot raises in the ore zones are mined using raiseborers or long hole drilling by drop raising.	
	Cable bolts are installed in the hangingwall of stopes from the sill in a fan pattern;	
LM - Neves-Corvo	Upper and lower access crosscuts are driven from FW drifts, across the orebody till HW contact;	Newall, 2017
	Top access is normally opened up to the full 12m stope width and appropriate support installed;	
	Used support: Cablebolts and Shotcrete; A slot raise is opened at the HW-end of the stope	
	Vertical rings are drilled and blasted on retreat to the FW; Mucking is done from lower access;	
	The back of former drilling lvl, is slashed out to establish new mucking lvl for next stope above.	
Pyhasalmi Mine	A single 4.5 x 4.5 meter drift is driven in the center on the top level of the stope;	Hustrulid, Bullock, 2001
	Two parallel drifts are driven on the sub- and loading level, at the left and right border of the stope	
	In some areas the top drifts are 12m wide and oval-shaped, to imporve stability;	
	In the 50m sublevel stopes and in the 25-m bench stopes, the slots are 25-m downhole raises.	
	Cable bolts are employed in the ore drifts to support the stopes.	
Mittersill Mine	Up to five levels are created in each stope, one bottom drift, three sublevel drifts and one top drift	Gaul, 2008
	The mining direction utilized in transverse stopes is top down; For longitudinal stopes bottom-up;	
	In transverse stopes the created void extends over several sublvls reaching heights of up to 80m.	
	In longitudinal stopes each level is mined and backfilled entirely bevor the lvl above can be mined	
	Raiseborers are used to create free space; Uphole and downhole drilling methods are in use.	
Breitenau Mine	In a stope over- and undercuts are established, vertically 11 meters apart.	Moser, 2019
	The roof of both drifts is reinforced with mechanical end anchors.	
	The overcut is used for downhole drilling and blasting; The undercut is used for mucking;	
	Depending on the thickness of the ore, the stope is backfilled completely or to the overcut level.	
	As the case may be, the created overcut can be used as undercut for the stope above.	
Williams Mine	Two stoping methods are used: Longhole stoping (~70%) and the newer Alimak method (~ 30%)	Hustrulid & Bullock, 2001
	Longhole stoping: The blasting is carried out in two seperate lifts (In prim. as well as sec. stopes)	
	Blasts are initiated around a ~1m diameter borehole, located in the center of the stope;	
	The hole initially severes as a fill raise for the stope below and later as a slot for blasting;	
	Alimak method: As initial slot a 3x3m rais is driven into the ore and ground support is installed;	Cox, 2017
	The HW will be cable bolted; The stopes are mined off along strike (longitudinal stoping)	
	The drilling and charging is performed in both strike directions from the Alimak platform;	

Figure 84: Secondary development - Stope preparation

4.3.6.2 Secondary development – Design aspects

To get a visual impression of the described preparation processes, this section focuses on the design possibilities. In the following figure, some of the previously-mentioned mines, are illustrated.

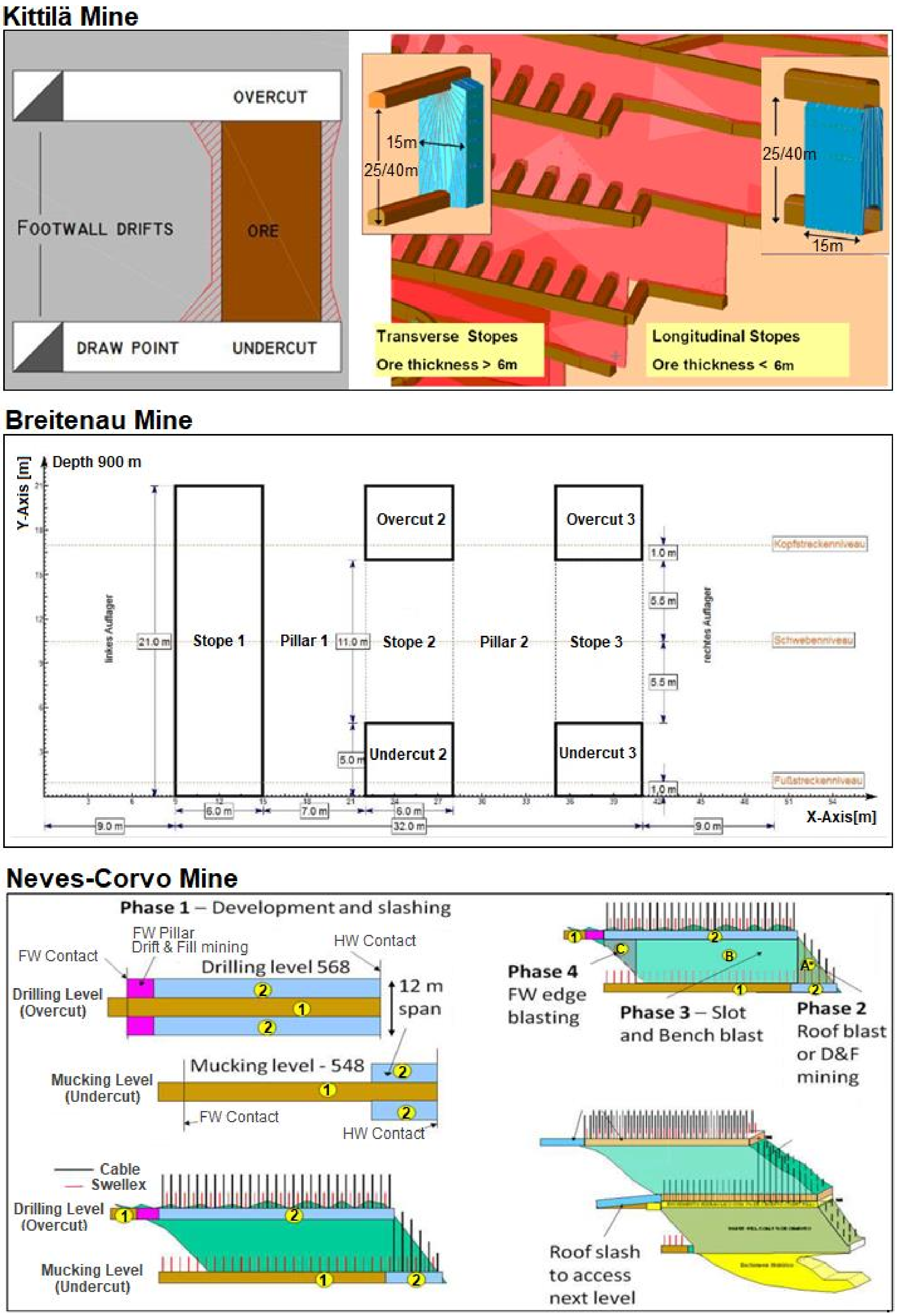


Figure 85: Extraction preparations (Tommila, 2014), (Doucet,2009), (Moser,2019), (Newall,2017)

Observing these drift structures, it is noticeable that all displayed layouts show strong similarities to the extraction preparations described in the last section. Analyzing the data from Figure 84, the same applies to the Garpenberg (Lapperget orebody), Zinkgruvan (Burkland orebody) and Kemi mine. A difference that can be recognized however, concerns the established overcuts in the Neves-Corvo mine. Here drift & fill mining is utilized to broaden these sublevels to 12 meters. The other mines by contrast only use a width of around 5 meters for their overcuts. Another interesting difference concerns the subdivision of the blasting sections. Since the Nerves as well as the Corvo orebody measures a much shallower dipping angle (~30°) than most of the other orebodies, a greater effort must be made to partition the stopes.

Since all mines mentioned so far apply single lifts for the extraction process, the Pyhasalmi mine as well as the Mittersill mine are good examples to illustrate the stope preparation in multi-lift stopes.

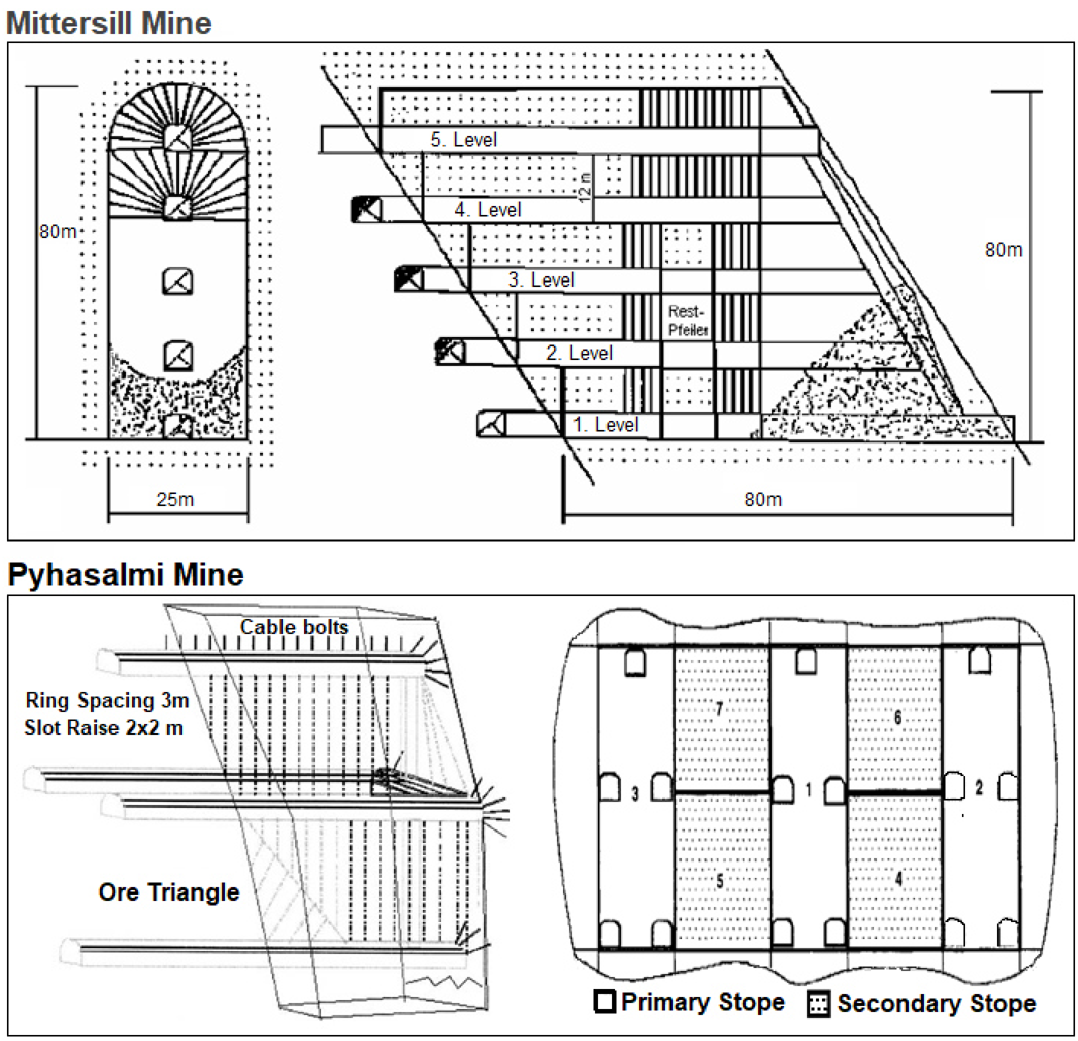


Figure 86: Extraction preparations (Gaul,2008) (Hustrulid & Bullock, 2001)

Especially the extraction preparations applied in the Pyhasalmi mine seems to be quite interesting, as all primary stopes are mined in one stage using three levels (reaching heights of up to 50 meters), while all secondary stopes are mined in two stages via bench and fill stoping. Another feature that stands out, is the horizontal arrangement as well as the number of established drifts, used for undercuts and drilling-sublevels. What should be noted is that this layout was illustrated by Hustrulid & Bullock in 2001 and has been changed at some point in time. According to a more recent report (First Quantum, 2020), the applied stope height concerning primary stopes now measures 20 to 30 meters, while also single draw points and overcut drifts are utilized. Unfortunately, there is no information on the reasons regarding the changes of the layout.

Analyzing the drift structure of the Mittersill mine, all sublevels are developed in the horizontal center of the stope. This applies to both, transverse and longitudinal stopes. However, when utilizing longitudinal mining, the stope is excavated in slices using a bottom-up sequence. In this process each slice is completely mined out and backfilled before the next slice above is mined. The applied mining direction in transverse stopes is top-down and displayed in figure 86. Drilling and blasting is done on multiple levels, which at some point leads to a stope height of up to 80 meters. In regards to drilling process they stated that, if high accuracy is needed the drilling is done top down, otherwise bottom up drilling is utilized. (Gaul, 2008)

A further element which is applied quite frequently in relation to stope preparation, are cable bolts. As illustrated in figure 85 and 86 (Neves-Corvo and Pyhasalmi mine) these support elements are primarily installed in overcuts (stope roofs) and the hanging wall. Thereby fan patterns are utilized for the arrangement of the bolts. The main objective of these elements is to reduce the quantity and size of outbreaks, by stabilizing the stope boundaries. Since there is a distinct correlation between the utilization of cable bolts and the reduction of dilution (Tommila, 2014), a lot of research is being done on this topic.

4.3.6.3 Secondary development – Critical parameters

In this section, all parameters related to the layout of the secondary development involving the stope preparation, will be listed. To summarize the geometrical aspects an illustration will be presented afterwards.

Structural paraments which have to be determined:

- Positioning of Footwall drift / Distance to orebody or stope
- Footwall drift cross section
- Cross sections and shape of sublevels (overcut, undercut and drilling levels)
- Sublevel spacing
- Number of sublevels per stope
- Support requirements for sublevels, hanging- and footwall
- Strategic constructions (Evasion possibilities, dumping areas for ore / waste rock)

Paraments and aspects which must be considered:

- Rock mass quality (Especially ore and hanging wall)
- Deposit geometry
- Extraction related aspects
  - Available equipment
  - Type and Size of equipment (top-down or bottom-up drilling)
  - Experience of the personnel (drilling accuracy)
  - Positioning and size of slot raise
- Logistic / Transport system
- Backfill system (Backfill type)

To get an overall consensus of the listed parameters and mentioned design aspects, the next figure illustrates a stope preparation layout, which can be derived from the researched data.

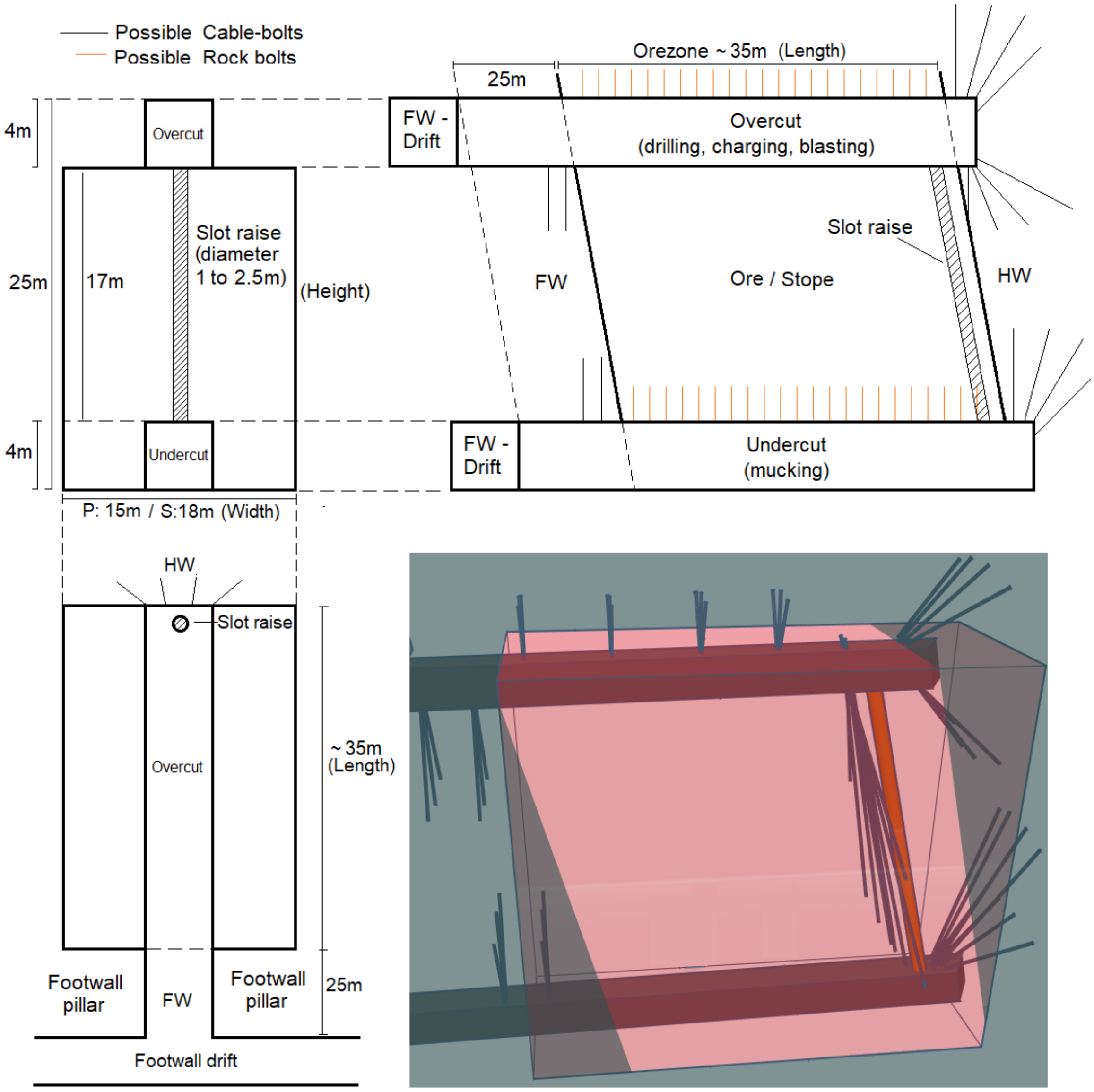


Figure 87: Stope extraction - Model layout

What should be noted, all displayed dimensions are either frequently used- or averaged values obtained from the researched mines and should only serve as an example.

4.3.7 Primary development

In this chapter the primary development of a sublevel stoping operation, will be discussed. Similar to the previous chapter, the first section provides a brief overview concerning the utilization of the primary infrastructure, involving the researched mines. Following on from this the selection process will be analyzed and important criterions related to that process are highlighted. Last but not least, all subparts of the primary development are outlined individually. This will also be done in reference to the investigated sublevel stoping operations.

4.3.7.1 Primary development – Overview

To get an overview concerning the general utilization of the primary infrastructure, the following table provides data, showing various application possibilities.

Mine	Prod. [t/a]	Depth [m]	Depth ext. [m]	First Mine Access	Haulage	Source
Garpenberg Lappberget	2 000 000	500	1400	Connection drift to Garpenberg North Ramp (at 500 m)	Shaft - surface to 1175	(Santis, 2020), (Van Koppen, 2008), (Högnäs, 2018), (Boliden-broschyr)
Agnico - Kittilä	1 800 000	70	1500	Ramp - From surface	Ramp till 500 Shaft *planned* to 1044	(Bertram, 2020), (Tommila, 2014), (Coucet, 2009)
Kemi mine	2 400 000	0	>1000	Ramp - From surface	Shaft - surface to 600	(Rikberg, 2019), (Mindat.org, 2020)
Zinkgruvan	1 400 000	200	1500	Ramp - From surface	Shaft - surface to 900	(Lundin Mining, 2020)
Neves-Corvo	2 000 000	230	1200	Ramp - From surface	Shaft - surface to 600	(Newall, 2017)
Pyhäsalmi Mine	1 100 000	0	1416	Ramp - From surface	Shaft - surface to 660 Shaft - surface to 1450	(Hustrulid, 2001)
Mittersill Mine	530 000	0	1000	Ramp - From surface	Ramp	(Gaul & Eggenreich, 2016)
Breitenau Mine	500 000	100	1100	Ramp - From surface	Ramp	(Schenkl, 2014)
William Mine	2 000 000	0	1400	Ramp - From surface	Shaft - surface to 1310	(Hustrulid, 2001)
Brunswick Mine	3 600 000	0	1200	Ramp; Shaft	Shaft	(Hustrulid, 2001)

Figure 88: Mine Access and Haulage System of researched mines

Observing this data, it is apparent that every mine individualized specific parts of their primary development, to achieve the intended production in the best possible / cost-efficient way. What should be noted, all displayed parameters are essential for the decision-making process concerning the applied primary development and will be analyzed in more detail in the subsequent section.

4.3.7.2 Primary development – Selection process and design criterions

The overall selection process regarding a primary infrastructure is quite complex, as various topics have to be considered. In order to make reasonable layout and design decisions, all relevant aspects have to be weighed and analyzed. The following flow sheet, developed at the Queens University, contains all involved topics and offers a useful indication of a possible selection process.

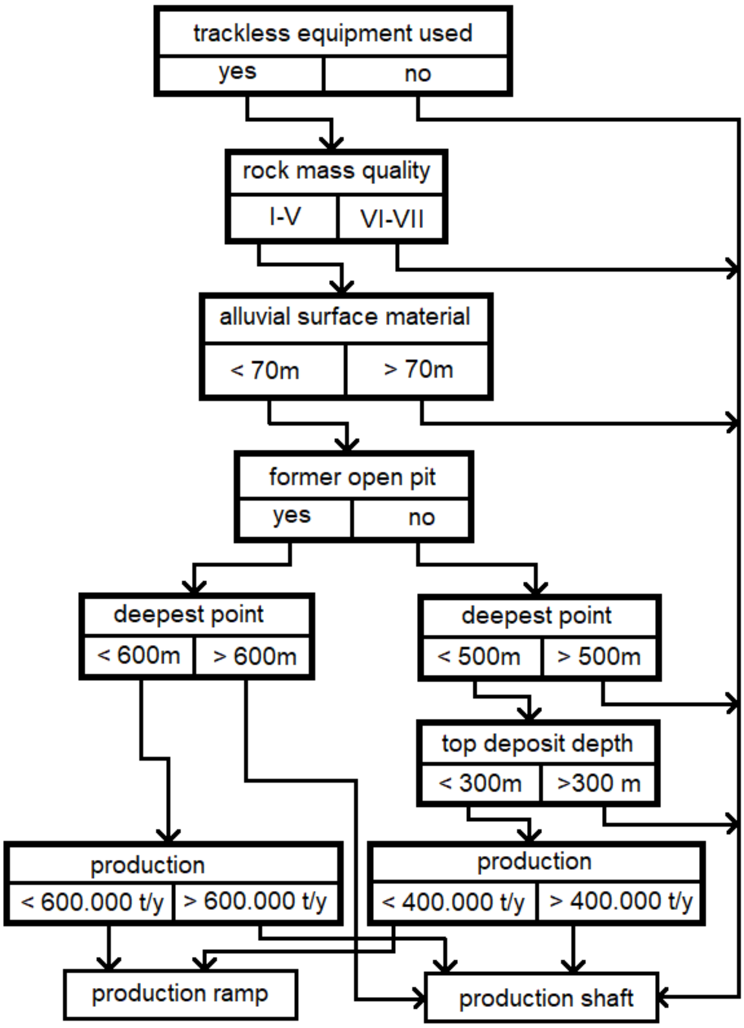


Figure 89: Selection of main access (De Souza – Queens University, 2010)

Observing this flow sheet, it is apparent that there are on the one hand rather simple / distinctive points like “former open pit” and on the other hand topics like “rock mass quality”, which usually vary and can only be estimated. Furthermore, it is not uncommon that results / values lie within intermediate ranges, making the decision for a primary development even more difficult. For this reason, various points should not only be analyzed individually but also collectively. In general, there are three major topics that are closely interrelated. These topics will now be elaborated.

Rock mechanics

One of the most important aspects concerning the selection process, is the rock mass quality of ore and country rock. According to this diagram (figure 89), ramps should mainly be utilized in moderate to excellent rock mass conditions. If the rock mass environment indicates a worse quality (poor to very poor), the construction of a shaft seems more appropriate. According to (De Souza, 2010) there are various reasons for this. An extensive requirement for ground support, would make the development of a ramp much more expensive. In that context, not only the material consumption is affected by a poor quality, but also the time required for construction increases. Furthermore, in the case of frequent usage (haulage), the maintenance costs may also become more complex to deal with, as stability is more difficulty to ensure. An important factor in this context are the required drift meters. This topic will now be highlighted.

Depth

Observing the depth values from figure 88, it is noticeable that the majority of investigated orebodies, only measure a top deposit depth of less than 200 m. Furthermore, various mines also utilized an open pit mine in the beginning of their production. These circumstances make it quite favorable to construct a ramp as a first access. What stands out is the Lapperget orebody in Garpenberg, measuring a top deposit depth of 500 m. The main reason for the presence of a ramp access, is the neighboring orebody (Garpenberg North), which had its roots on the surface and was mined prior to the Lapperget orebody. A connection drift at around 900 m was enough to provide access to the ground surface. With the exception of this case, every researched mine in chapter 3, utilized a ramp-system for access and haulage purposes in the first place. However, what is also evident, nearly every operation (7 out of 10 mines) transitioned to a shaft transportation at some point. The question that arises, at which depth is a change of the haulage system appropriate? A value which is mentioned quite frequently in that context lies around 300 to 350m (figure 89). However, according to (McCarthy et al., 1993) this value does not agree with recent experiences, as various Australian mines suggest a value of 500 m or more. He further states that this question cannot be answered in general, as many factors such as mine life, production, mining method, etc., play a significant role. A conclusion he nevertheless came to is that the optimal changeover depth becomes shallower as mine life increases.

Production aspects

The two biggest advantages offered by a ramp access, is the high flexibility concerning maintenance and the better transportation possibilities in regards to heavy mining equipment. As production increases, these advantages become even more important, since a greater number of machines are required. Observing the flow sheet, a production of 400.000 to 600.000 t/y, seems to be the edge of efficiency in regards to truck haulage, if the deposit indicates a depth of ~500 to 600 meters. This also corresponds to the investigated data. Observing figure 88 it is noticeable, that nearly all stoping operations, showing a production lager than 600.000 t/y, apply skip systems for the transportation of ore and waste rock. An interesting case in this regard is the Kittilä gold mine, which seems to be standing out with a production of more than 1.5 Mt per year. However, the construction of the ramp made sense since the deposit started at a depth of 70 meters. Furthermore, they also stated that to maintain efficiency, a shaft is needed for the extraction of the deeper regions.

4.3.7.3 Primary development – Components and Critical parameters

In this section the three main elements that represent the primary development are going to be discussed. This includes the main ramp, the haulage shaft and the production ramp.

Main Ramp

The first infrastructure which will be analyzed in more detail is the main ramp. In general, the main ramp is characterized by multiple purposes. Analyzing figure 88, it is apparent that one of its main functions is to provide initial access from the surface to the orebody. In many cases this access is then further used for all movements (in and out of the mine) concerning men, machines, needed materials as well as produced ore and waste rock. In addition, it usually forms an important part of the overall ventilation structure. Regarding the design process, there are several parameters which are rather defining for a ramp system. These will now be highlighted.

Structural paraments which have to be determined:

- Inclination
- Cross section
- Turning radius
- Evasion areas
- General positioning related to the deposit location

To get an insight concerning applied values, the next table highlights various sublevel stoping operations, including information on these geometrical parameters.

<table>
<tr><th>Mine</th><th>Inclination</th><th>W x H</th><th>Turning radius [m]</th><th>Source</th></tr>
<tr><td>Garpenberg North</td><td>1:7</td><td></td><td></td><td>(Mining-technology, 2006)</td></tr>
<tr><td>Agnico - Kittilä</td><td>1:7</td><td>5 x 5</td><td></td><td>(Tommila, 2014)</td></tr>
<tr><td>Kemi Mine</td><td>1:7</td><td>8 x 5,5</td><td></td><td>(Fernberg, 2007)</td></tr>
<tr><td>LM - Zinkgruvan</td><td></td><td>5 x 5,4</td><td></td><td>(Daffern,2017)</td></tr>
<tr><td>Pyhäsalmi Mine</td><td>1:7</td><td></td><td></td><td rowspan="5">(Hustrulid, 2001)</td></tr>
<tr><td>Williams Mine</td><td>15%</td><td>4.8 x 3,5</td><td></td></tr>
<tr><td>Lamefoot Mine - Washington (NE)</td><td>12%</td><td>5.5 x 4,6</td><td>20</td></tr>
<tr><td>Kola Peninsula - Russia (NW)</td><td></td><td>5 x 4,5</td><td></td></tr>
<tr><td>Stillwater Mine - USA, Montana</td><td>15%</td><td></td><td>15,2</td></tr>
</table>

Figure 90: Ramp systems used in stoping operations

The inclination is one of the most important parameters for the entire logistic system, as it effects the transport time, fuel consumption, developed drift meters, time for production to start and numerous other factors in a direct or indirect way. Observing this table, it is apparent that usually values between 12% and 15% are applied for the inclination of main ramp systems. This value range has proven to be quite efficient in terms of transportation and logistics. Higher inclinations are rather exceptional, as the efficiency of fuel-powered vehicles decreases significantly.

Another geometrical parameter similarly important, is the used ramp cross section. Since, in most cases, the decline is not constructed within the orebody, the cross section should be as small as possible, but at the same time as large as required for the trucks to maneuver. However, each mining operation is somewhat unique in terms of production and logistic requirements, resulting in detailed layout differences. This can also be derived from the researched data. Observing the table above, it is apparent that the cross sections of all listed ramps is (slightly) different. This particularly concerns the height of the applied profile. The width seems to be a bit less variable as a value of 5 meters is commonly used. However, the applied cross section of the Kemi mine illustrates, that significant differences can also occur in the width.

Production ramp

The production ramp has the purpose to provide direct access to specific parts of the orebody, for production and transportation vehicles at various depths. Its purpose is to connect all sub- and main levels, which are operational and which are currently being development. The key parameters concerning the design process are rather similar to those mentioned for the main ramp. However, what must be considered is that the production ramp has slightly different requirements, as numerous production and transportation vehicles have to be maneuvered and repositioned constantly. This implies that inclinations and cross sections have to be adjusted and the accessibility to ore- and waste passes have to considered.

The general placement of the production ramp is typically in the footwall. However, in some cases it has to be constructed in the orebody itself or in the hanging wall (only rarely). An example for a ramp constructed in the orebody is the Breitenau mine in Austria. Here the compact magnesite features better rock mass properties than the footwall material, which among other facts led to the decision to place the ramp into the orebody. This resulted in numerous advantages and disadvantages. On the one hand little to no waste is produced during the construction of the ramp. On the other hand, the incline is increasingly affected by the surrounding mining activities, causing more frequent stress changes in ramp pillars. (Siefert, 2003)

Main-Shaft (Hoisting shaft)

Similar to a ramp infrastructure, the main shaft is characterized by multiple purposes concerning logistics and transportation. The main purpose however, is to hoist ore and waste rock from specified levels of the mine to the ground surface. Depending on the applied size (shaft diameter), there are multiple possibilities to structure a shaft, resulting in more or less functionality. The overall production quantity depends very much on the shaft dimensions, depth and design of the shaft. To get a more detailed insight concerning applied parameters, the following table highlights various hoisting shaft systems, which were designed for sublevel stoping operations.

Mine	Shaft loading at [m]	Shaft diameter [m]	Production [Mt]	Skip Capacity [t]	Source
Garpenberg - Lappberget + North	1150	6	2,5	2 x 28,5	(Boliden-broschyr)
Agnico - Kittilä (*in Construction*)	1000	5,6	2,5 (*Planned*)	/	(IM-International Mining, 2019)
Outokumpu's Kemi	570	5,2	2,4	26	(Rikberg, 2019), (ABB, 2004)
LM - Zinkgruvan	850	5,5	1,4	/	(Daffern,2017)
LM - Neves-Corvo	600	5	2	2 x 17,8	(Newall, 2017)
Pyhäsalmi Mine	1400	5	1,1	21,5	(ABB - Industries, 2020)
Williams Mine	900	7	2	2 x 22	(Hustrulid, 2001)

Figure 91: Hauling Shafts – Used in stoping operations

Observing this table (figure 91) in context with figure 88, it is apparent that shafts are generally utilized when deposits exceed a depth of approximately 1000 meters. As mentioned in the previous section, not only the depth but also the production is significant in this regard. According to the flow sheet, a shaft becomes rather lucrative when the production of an underground mine exceeds 600kt per year. Looking at the values in the table above, all mines applying shaft systems, report an annual production of at least 1 million tons. Consequently, this is also in agreement with the suggestions of the flow sheet. However, according to (McCarthy et al., 1993) the total costs of transportation (including maintenance and ventilation), can already be more in favor for shaft systems, starting from a production of 250k tons per year. The example in which they mathematically proved this, considered a truck (50 tons) transportation from a depth of 600 meters, which again corresponds with the depth limits stated in the flow sheet.

In regards to the investigated shaft diameters, the dimensions vary between 5 and 7 meters. What is notable, although there is a difference of 2 meters in the shaft diameter, the Neves-Corvo and the Williams mine show the same production quantity. An even higher production (2,4 Mt) can be assigned to the Kemi mine, indicating only a slightly larger diameter of 5,2 meters. This illustrates quite well the interrelation between shaft radius, depth and applied skip system.

4.3.8 Backfill

Last but not least the utilization of backfill in current sublevel stoping operations will be analyzed. To get an overview, real world examples covering application and handling, will be presented in the first section. Following on from this, various backfill strategies are examined more closely, by highlighting important aspects of the individual backfill types. Last but not least, the critical parameters and selection criteria are elaborated.

4.3.8.1 Backfill – Overview

The decision of which backfill material should be applied to replace the void of an extracted stope, depends on various factors. This not only involves rock mechanical and geological aspects but also layout specific factors. For example, depending on whether a created stope is planned to be primary, secondary or tertiary, it is reasonable to apply different types of backfill. To get an impression, the following table presents information, obtained from various sublevel stoping mines.

Backfill utilization		
Mine	**Description**	**Source**
Garpenberg Lappberget	Paste fill is produced in plant on surface and distributed underground by pipelines	
	Paste is used as backfill in major parts of Lappberget (Primary and Secondary stopes).	(Van Koppen, 2008)
	Secondary stopes are only partly filled with rockfill.	
Agnico - Kittilä	Past is produced in backfill plant on surface and delivered through a past line system.	
	Primary stopes are backfilled with paste fill or cemented rockfill	(Tommila, 2014)
	Secondary stopes are filled with rockfill.	
Outokumpu's Kemi	Cemented slurry is placed in Primary stopes.	(Lamberg, 2016)
	Secondary stopes are backfilled with cemented slurry as well as waste rock.	
Zinkgruvan - Burkland	Hydraulic fill was used over a long period of time, but had to be replaced with paste fill.	(Fernberg,2007)
	Primary stopes are now backfilled via paste fill and Secondary stopes via waste rock.	
LM - Neves-Corvo	All Optimised bench & fill stopes are mined in a bottom up sequence with paste backfill.	(Newall, 2017)
Pyhäsalmi Mine	Primary stopes are mined using sublvl stoping (Height 50m). Consolidated backfill is used.	(Hustrulid & Bullock, 2001)
	Secondary stopes are split into two benches, each 25 meters high. Waste rock is used.	
Mittersill Mine	Usage of Hydraulic fill, pastefill and rockfill. All excavated areas are all backfilled.	(Gaul, 2008)
Breitenau Mine	Stopes are excavated and later backfilled with paste fill.	(Wagner, 2015)
	After backfilling, the adjacent stope is excavated.	
William Mine	Cemented rock fill is used in primary stopes while dry fill is used in secondary stopes.	(Hustrulid & Bullock, 2001)
Brunswick Mine	Used a primary / secondary stope pattern with delayed backfill in the beginning.	(Hustrulid & Bullock, 2001)
	Switch later to Pyramidal pillarless open-stope mining with rapid paste backfilling.	

Figure 92: Utilization of backfill in sublevel stoping operations

What should be noted, there are two backfill approaches which must be distinguished. This is 'delayed' backfill, in which a stope is completely filled in a single operation and 'rapid' / 'cyclic' filling where the backfill material is placed in sequential lifts.

Observing this data, certain points concerning the application become apparent. On the one hand, in all researched cases primary stopes are backfilled with paste fill, cemented rockfill or hydraulic fill. Secondary stopes on the other hand are only partially filled with paste fill, but most often with rockfill / waste rock. The utilization of hydraulic fill by contrast, is only used in two of these stoping operations. The reasons for these backfill applications will be now be examined in more detail.

4.3.8.2 Backfill – Application and Types

Overall there are three main types of backfill, which are commonly used in sublevel stoping operations. These are rock fill, paste fill and hydraulic fill. Each type differs in effectiveness and applicability concerning the prevailing rock mass quality and is further characterized by various economic advantages and disadvantages. In addition, the application purpose of each type also differs to a certain extent. To get an overview which general objectives are important for sublevel stoping mines, a list is made in the following.

<u>Backfill objectives:</u>

- Provision of stability
- Possibility for underground waste disposal
- Preparation of an appropriate working floor
- Better selectivity and increase of ore reserves
- Reduction of heat inflow by reducing the total face length

To achieve these goals the vast majority of sublevel stoping operations use multiple backfill types. A very common strategy in this regard is the utilization of paste fill in combination with rock fill. Analyzing the data in figure 92, it is apparent that this also applies to most mining operations mentioned in the table. The reasons for this are quite simple. On the one hand waste material, which would require costly transportation and disposal, gets a new purpose as passive support. Furthermore, since it is produced by blasting the rock properties are quite favorable for a combined usage with other backfill types. Paste fill, on the other hand, entails very important strength properties. The high stiffness and low porosity make it very suitable as fill material and as a medium to create solid working platforms. Additionally, it can be used as a support for tunnels. Furthermore, in comparison to hydraulic fill no access water is needed and there is no risk of liquification.

Hydraulic fill is only used in two of the listed sublevel stoping operations. In one of them only for a certain period of time. Although this backfill type has some infrastructural / organizational disadvantages compared to paste fill, like the additional process of de-sliming and the utilization of massive barricades, it is known to be the fastest system to fill volumetric large cavities. Moreover, there is no need for high pressure pumps and the use of costly binder is also not mandatory. However, one of the main reasons why hydraulic fill is applied less frequently overall, has to do with the rock mass conditions and the large quantities of utilized water. An example which highlights this quite well, is the problem which emerged in the Zinkgruvan mine.

The main mining method which was used in Zinkgruvan over a long period, was sublevel open stoping with the utilization of hydraulic fill. This method, only slightly adapted, should also be applied in the newly tapped Burkland orebody. Unfortunately, the mining had to be stopped after the excavation of the first stope, as the rock mass conditions were worse than expected. These circumstances led to the decision to change the current method to longhole open stoping. Furthermore, the hydraulic fill also caused more and more difficulties, as the cracked rock near the draw points could not be sealed up with the bulkheads. The only way to overcome these massive problems was to change the backfill system from hydraulic fill to paste fill completely. (Fernberg et al., 2007)

This example further illustrates that the selection of a backfill system / material has to be based on many different aspects and has to be adapted in case the conditions or layout changes. The next section will summarize all mentioned aspects and parameters.

4.3.8.3 Backfill – Critical parameters

Last but not least, this section highlights all aspects which are relevant for the choice of a backfill system. The following list provides an overview of all essential parameters.

<u>Paraments and aspects which must be considered:</u>

- Main objective of the applied backfill material
  - Provision of stability
    - Needed support pressure
  - Underground waste disposal
    - disposal costs / benefits of waste material
  - Preparation of working floor
- Rock mass condition of deposit
  - Rock mass quality
  - Rock mass strength
  - Jointing and discontinuities
- Mine layout
  - Stoping Method
  - Stope Sequence
- Environmental factors
  - Surface subsidence
  - Mine water handling

A parameter that has not been further specified, concerns the provision of stability and is known as the required support pressure. As this topic is rather extensive, it will only be highlighted briefly.

Backfill is categorized as a passive support system, meaning that the produced support pressure depends on the reaction of the host rock. This reaction in turn is influenced by a number of factors. The main parameters which are responsible for the provided support pressure, are listed in the following.

- Rate of volumetric closure
- Timing of placement
- Stiffness of fill material

The overall result after placing backfill, should be a lessened increase of stresses in the surrounding rock mass and a higher strength in the side walls by adding additional confining pressure. However, each backfill type is characterized by various advantages and disadvantages, leading to different results concerning the fulfillment of these three specifications. Therefore, it is essential to determine the required support pressure and thus to apply the proper material.

4.4 Conclusion – State-of-the-Art research

The design process of major mining structures, such as a sublevel stoping operation, encompasses many different subject areas and requires a systematic breakdown into individual structural- and process related elements. The difficulty, is that all elements have to be analyzed and designed individually, but also coordinated and aligned with each other. The key to develop an efficient and functional design for all elements, which also corelates with the overall structure, is to use proper design methods in certain phases of the project. In that context, an important question to be asked is: “Which design method would be most appropriate for this element, during this phase of the project, with the current amount of data?”

The final decision of a design method is usually an interplay of experience and data interpretation. However, as mentioned in 4.3.1, it is recommended to apply multiple types of methods, as each type has its own preferences. To benefit from all aspects, analytical, empirical and numerical methods should be (to a certain extend) applied in combination. A reasonable approach that could be concluded from the investigation can be summarized as follows.

Since only few information is available at the beginning of a mining operation, results of empirical methods should potentially be assigned more relevance than results from numerical models running on little data. However, it has also become apparent that not every element has been studied to the same extent in the past. This led to the situation that certain topics (elements) offer more accurate (empirical) information than others. As the amount of data increases with the progress of mining, more relevance can be attributed to numerical models. However, even with larger databases, a degree of uncertainty concerning specific parameters will always remain. Through experience and back analyses of stable, unstable and failed stopes, location specific stability charts can be created, which can then be used as a guide and alternative to the numerical models. Furthermore, to identify high stresses and significant deformation in time, analytical methods can be applied continuously during a mining project. The main objective of such an approach is to generate clear and valid data, which can then be transformed into information through right interpretation. Since mining in general has a lot to do with experience, interpretations can also vary. For this reason, it is useful to additionally obtain expertise’s from specialists for specific elements or issues.

Concerning the investigated element-specific parameters and related design aspects, the following could be concluded.

The identified critical parameters as well as the associated effects are quite unique for all presented elements and most often feature a high degree of complexity in terms of interrelation. The challenge concerning the development of a functional and effective design, is to conclude how and to what extent critical parameters, like specific geometrical dimensions, influence the stability of certain elements. Significant effects which were presented in this regard are, the effect of confinement, the scale and shape effect. Example values for several parameters are outlined and discussed in chapter 3. Further aspects, which influence the design of a sublevel stoping structure, are the prevailing rock mass conditions as well as the stress environment. Both topics are strongly interrelated and have a significant impact on the individual elements and the overall layout of a sublevel stoping mine. In order to manage these difficulties, specific extraction sequences are applied to redistribute stresses and control the behavior of the rock mass. In this regards the investigation revealed that 9 out of 10 mines apply a sawtooth shaped primary - secondary stope sequence / structure with subsequent backfill, to mine the deposit and control the stresses. All mines used paste fill, cemented rockfill or hydraulic fill for primary stopes and most often rock fill for secondary stopes.

Concerning the primary infrastructure, the research concluded that 8 out of 10 mines utilize shaft / skip systems for ore transportation and 10 out of 10 mines additionally used ramp systems as a first access method to reach the orebody. However, since this topic is quite dependent on various geometrical characteristics of the deposit as well as the size of the mining operation, the reader is referred to chapter 4.3.7 in which this subject is discussed in more detail.

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