DESIGN OF SUPPORT SYSTEM BY OVERHAND CUT AND FILL MINING METHOD IN UNDERGROUND GOLD MINE, INDONESIA

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ABSTRACT

Although mineral resources have mainly been extracted by open pit mining method in Indonesia, the underground mining will be promoted in terms of the increasing mining depth and the environmental protection. Overhand cut and fill mining method is used in steeply dipping ore bodies in strata having a relatively weak strength and comparatively high grade ore. In cut and fill mining method, as the mined voids are backfilled with waste rock or tailings, the surface subsidence due to mining operation can be controlled and the impact on the environment is small. Generally, the roadways are developed from the surface and then the cross-cut is derived from roadway to the ore bodies to extract ore. As both the hanging wall and footwall are weak compared with orebodies, overhand cut and fill mining method is applied in Cibaliung underground gold mine, Indonesia. However, failures have been occurred in roadway, cross-cuts and stopes due to the extraction of ore in the area where the ore bodies are heavily fractured. Hence, a reassessment of the current support system and a development of more effective one have to be conducted in order to continue an effective and safe mining operation under these conditions. From these backgrounds, the effect of an induced stress due to the overhand cut and fill mining operation on the stress conditions around the roadways/openings have been evaluated in Cibaliung underground gold mine, Indonesia and then a design guideline for an appropriate support system in roadways, cross-cut and stopes have been proposed in this study, consisting of seven chapters as follows:

Chapter 1 introduces the background of this research, geotechnical issues and mining technology related to this research topic and an involved outline of the dissertation.

Chapter 2 describes the current support system and its design guideline used in Cibaliung mine. Moreover, as this support system has been developed and used in another gold mine, Pongkor underground gold mine in Indonesia, the effect of this support system on the stability of roadways in Pongkor mine and it characteristics are discussed by means of numerical analysis.

Chapter 3 discusses the effect of current support system on the stability of roadways in Cibaliung mine. According to the results obtained from the numerical analysis, the current support system does not work well in this mine. This is because the geological conditions such as rock mass fracture state, mechanical properties of rock, in-situ
stress conditions are different with those of Pongkor mine. Moreover, the effect of additional stress induced by mining operation on the stress and rock mass conditions around roadways are not evaluated and considered in the current support design. Hence, it can be said that a new design guideline of support system considering these factors has to be developed in this mine.

Chapter 4 describes empirical and numerical methods in order to evaluate the effect of induced stress of stope on the stability of roadways in quantity. Based on these results, the prediction chart of maximum tangential stress factor is proposed. This prediction chart includes the features that the maximum principle stress works only in the tangential direction along the wall and the stress conditions around the roadway is changed into the initial stress condition with increasing the distance from stope to roadway. It can be said from this prediction chart that the failure of roadway may be occurred when the distance from roadway to stope is less than 20m and the location of the maximum principle stress factor is changed as the distance between roadway and stope decreases. Based on these results, it is made clear that the stability of roadway cannot be maintained because enough supports are not installed in the area that the stress concentration factor is large in the current support system. Hence, the stress condition around the roadway have to be evaluated precisely and then the support system has to be designed considering the stress condition in order to maintain the stability of roadway affected by mining operation in stope. The stability of the roadway can be maintained under the condition that a 15cm thick layer of shotcrete and H-beam with spacing 0.6m are installed, and the length of the rock bolt is changed from 1.8m to 2.4m and they are installed at 1.0m intervals on the sidewall roadway far from stope, and done at 0.5m intervals on the roof and the sidewall roadway near from stope.

Chapter 5 discusses the design of a support system for the cross-cut which connects the roadway and orebodies/stopes and is developed in hanging wall by means of numerical analysis. It can be said from the results that the stability of cross-cut cannot be maintained by using the current support system when the distance of cross-cut and stope is less than 20m because a failure zone caused by induced stress of stope is developed in the roof. This situation could be improved by the application of a cable bolt. The stability of the roof of the cross-cut can be improved by installation of 11m cable bolt in the vertical direction. Besides, the anchor effect is also obtained by connecting the roof and sill pillar, which is left as a safety pillar in the upper part of the
stope, by using cable bolt. However, the failure zone still develops at the roof under the condition that the distance between the cross-cut and stope are less than 10m. Therefore, standing support such as steel arch and cribs has to be installed in order to maintain the stability of the cross-cut.

Chapter 6 discusses an appropriate support system for the stope using numerical analysis by changing the thickness of the sill pillar and the installation pattern of rock bolts with a focus on the condition of fractures in the ore body. It can be said from the results that the stope can be maintained by using the current support system under the rock mass condition that RQD value, which is index for the description of rock mass fractures state, is larger than 60. On the other hand, in case that the value of RQD is less than 60, the length and the number of installation rock bolts should be changed from 5m to 7.5m and from 3 pieces to 9 pieces, respectively. Furthermore, not only changing the thickness of the sill pillar and the pattern of the construction of the rock bolt, but also by installation of standing support such as rigid arch and cribs or increasing the strength of backfilling materials are required in order to maintain the stability of the stope when the extraction level reaches to 150m depth.

Chapter 7 concludes with the results of this study.
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CHAPTER 1

INTRODUCTION

1.1 GOLD MINING INDUSTRY IN INDONESIA AND ITS MINING’S PROBLEM

Mineral resources are non-renewable nature but essential for national development. Non-renewable means that mineral resources cannot renew itself at a sufficient rate to be able to sustain for increasing demand. The increasing consumption of mineral leads to depletion of resources since the quantity of non-renewable resources is finite or limited resources. This perspective leads to optimize a mineral exploitation and becomes the essential principal of mining activity. Despite of advantages of mining, it is undeniable that some problems could have occurred as a result of mining activities. One of the examples of the negative impacts caused by mining activities is an environmental issue.

There are many factors that must be evaluated in the choice of a mining method. Some of primary variables that must be considered are spatial characteristics of deposit, geologic conditions, geotechnical properties, economic considerations, technological factors and environmental concerns (Boshkov and Wright, 1973; Morrison and Russel, 1973; Hartman and Mutmansky, 2002; Sahriar et al., 2007). Mining methods are generally classified into three types: surface mining, underground mining and underwater mining. Another method of mining that recently implemented is in-situ mining or novel mining. All of mining methods can be applied broadly; however, only surface and underground mining are usually operated in conventional extraction methods of mining sites as a practical way. Surface mining method is an open-air excavation for removing minerals inside of the earth and used to extract mineral from any deposits that are located close to the earth’s surface.

Surface mining needs a large capital investment but generally in high productivity, low operating cost and good safety conditions (Hartman and Mutmansky, 2002). Meanwhile, underground mining is a method to extract minerals inside of the earth whereas the location of mining activity is not directly connected to the open air.
The problems of underground mining are more complicated than that of the surface mining. Underground failure can be found and fatality could occur to mine workers. In addition, because of the instability, the mining operation may be disrupted and reduced productivity while cost will be increasing. Therefore, a good mine design is paramount to prevent underground stability.

Development of mining industry plays a significant role in Indonesia economic growth that accounted for about 11.77 percent of Gross Domestic Product (GDP) in 2012 (Badan Pusat Statistik, 2013). The mining contribution to Indonesia's GDP is given in Figure 1.1. Based on the Figure 1.1, the mining sector gives an important portion to the economic growth in Indonesia, as mining ranked as the third highest in the term of GDP. It means that the increasing of mining activity also directly affects the national GDP in Indonesia.

![GDP of Indonesia](image)

Figure 1.1 Gross Domestic Product of Indonesia from 2008 to 2012 (Badan Pusat Statistik, 2013)

Indonesia is ranked as one of the top ten gold producing countries in the world (see Figure 1.2). About 5% of total gold in the world is produced in Indonesia. Total export of gold in 2011 was US$2,220 million (Oliphant, 2013). The total gold production in Indonesia and distribution of Indonesian gold mine can be seen in Figure 1.3 and Figure 1.4, respectively. However, some mineral resources in Indonesia are scattered in the protected forest. Therefore, the environmental aspect for mining activities should be considered. The conservation Law No. 28 year 2011 of Republic of Indonesia government explicitly regulates the mining activities in the protected forest. Referring to
this regulation, mining activities cannot be acted in the protected forest area. However, underground mining could be permitted under some requisites with high supervision from government.

Figure 1.2 Top ten gold producing countries in the world (Gold Investing News, 2013)

Figure 1.3 Indonesia gold production (BadanPusatStatistik, 2013)
Generally, there are two types of major gold deposits in Indonesia i.e. porphyry ore deposit and epithermal ore deposit. Porphyry ore deposits are the world’s most important source of metal that associated with copper. Grasberg in Papua and Batu Hijau in Sumbawa are copper porphyry deposits that contain major quantities of gold and silver. The porphyry ore deposits are distributed along tectonic margins where oceanic tectonic plates subduct down into the earth’s mantle. Porphyry ore deposits often have circular or elliptical shapes in the cross section view and have vertical deposit dimensions similar with the horizontal one. Mineralization zoning comprises a broad shell which is centered over around on small cylindrical porphyry intrusive stocks, and sometimes directly overlies larger underlying intrusions or batholiths. The ore deposits are typically has large tonnage.

The second type of ore deposit is epithermal ore type. Based on the genesis, epithermal ore deposits can be classified into two main types i.e. high sulphidation ore and low sulphidation ore. Each of these types has its own characteristics. Silver and gold contents tend to be higher in low-sulphidation type deposits than in high-sulphidation type deposits. Epithermal ore deposits usually develop along the vein and
follow the existing geological structure. The ore deposits of epithermal are usually less than porphyry ore deposits in tonnage and some of them are also low in grade.

Due to the environmental impact, underground mining will become a solution to excavate the ore reserves. However, underground mining is more complex than the open pit mining and some of the reserves categorized as marginal and under weak geological conditions. Moreover, to reach the economic values unusual access is developed in hanging wall side and gives high risk to failure and subsidence. Therefore, designing a good support system around the roadway of hanging wall access in underground mining is very important in Indonesia.

1.2 CIBALIUNG UNDERGROUND GOLD MINE AS A FIELD OF RESEARCH

The case study was carried out in Cibaliung underground gold mine. Cibaliung mine is located in Pandeglang regency, Province of Banten, Indonesia. The geographic coordinate is in 105°38’00 E and 6°46’00 S and it is situated about 200 km from Jakarta in the western part of Java Island (Figure 1.5). The location of Cibaliung underground gold mine is 70km approximately from Pongkor underground gold mine area. Both Pongkor and Cibaliung mines have many similarities in terms of ore type deposit (epithermal ore deposit), geological type and mining method (overhand cut and fill mining).

![Figure 1.5 Location of Cibaliung and Pongkor underground gold mine](image-url)
1.2.1 Geological Information

Geologically, the host rock of Cibaliung deposit is the Honje Igneous Complex. The unit of the Honje Igneous Complex seems to be quite similar to Bayah Dome, where Gunung Pongkor and Cikotok mineral districts are located. The oldest rock unit of the Honje Formation is composed of a thick sequence of basaltic andesite to andesite flows and volcanic breccias with some intercalated sediments. They are extensively folded, faulted and exposed in a north trending horst bounded by west dipping normal faults along the Java coast to the west, and east dipping normal faults below the sedimentary basin to the east. Furthermore, the Honje Formation is intruded by sub-volcanic andesite and diatreme breccia. All of the above-mentioned are pre-mineral rocks which are uncomfortably overlain by the post mineral Cibaliung tuff, generally covering the NW trending graben area (Marjoribanks, 2000). The lithology of country rocks surrounding Cibaliung underground gold mine generally are altered by chlorite-adularia in footwall and smectite-illite in hanging wall. Mechanical and physical properties of Cibaliung rocks are given in Table 1.1. Geomorphology of Cibaliung is a hilly area about 30 – 300m above sea level and the slope is around 10–25%. The highest hill is Gunung Honje (620m) where located in western of underground location, however in the mine area the topography is flat.

The prospects of gold in Cibaliung occur within North-West trending structural with 3.5km in width and at least 6km in length. It is fault bounded and is considered to be a graben (Marjoribanks, 2000). Ore shoots in the Cikoneng and Cibitung areas occur in complex dilational jogs and cymoid bends that are formed at intersection between North-West, North-North West and North-North East fault systems (Angeles, 2002). The geological map of Cibaliung mine area is given in Figure 1.6, where two main ore shoots can be seen here i.e. Cikoneng and Cibitung areas. Cikoneng ore shoot geometry is estimated 250m in length, 2-15m in wide, 200m in depth, whereas Cibitung ore shoot geometry is 150m in shoot length, 1-8m in wide and 300m in depth. Figure 1.7 shows the subsurface geological map.

Based on geotechnical investigation report, ratio between vertical and horizontal in-situ stress is 4.8 MPa : 9.2 MPa or similar with 1 : 2, which mean the horizontal stress in Cibaliung area is higher than that the vertical stress (Campi and Dugan, 2004).
Figure 1.6 Geological map of Cibaliung area

Table 1.1 Mechanical and physical intact rock properties

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Breccia Smectite Hanging wall</th>
<th>Rock type Breccia Chlorite Footwall</th>
<th>Quartz vein Ore body</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>min</td>
<td>Max</td>
<td>mean</td>
</tr>
<tr>
<td>$E$ (GPa)</td>
<td>6</td>
<td>37</td>
<td>21</td>
</tr>
<tr>
<td>$\nu$</td>
<td>0.25</td>
<td>0.5</td>
<td>0.36</td>
</tr>
<tr>
<td>$c$ (MPa) peak</td>
<td>7</td>
<td>19</td>
<td>11</td>
</tr>
<tr>
<td>$\phi$ (°) peak</td>
<td>11</td>
<td>41</td>
<td>26</td>
</tr>
<tr>
<td>$\sigma_l$ (MPa)</td>
<td>0.2</td>
<td>3.5</td>
<td>2.4</td>
</tr>
<tr>
<td>$\rho$ (gr/cm$^3$)</td>
<td>2.5</td>
<td>2.51</td>
<td>2.5</td>
</tr>
</tbody>
</table>
Figure 1.7 Subsurface lithology around Cibaliung underground mine
1.2.2 Type of Openings of Cibaliung Underground Gold Mine

In Cibaliung mine, the underground development openings consist of five main openings:

1. Portal

Portal is defined as the intersection of the tunnel or underground mine with the surface. The first portal was built in 2006 in Cibitung area. However, this portal has been closed due to the failure of the support system of the portal. This accident costs about 6 million USD. After the first portal collapsed, the second portal was built. The location was moved to Cikoneng area. This portal is still operated until now and become the only entrance to the underground mine of Cibaliung. The geometry of Cikoneng portal is 4.2m in wide and 4.8m in height. Figure 1.9 shows the portal of Cibaliung underground mine.

![Figure 1.8 Cibaliung underground portals](image)

2. Roadway

Roadway is one of the most important part of underground mine owing to its function. The roadway is the main access for mine workers, materials and equipment transport. The design of roadway of Cibaliung gold mine is ramped down (decline) with 4.2m in wide and 4.8m in height. The slope of this ramp is approximately less than 15%. In Cikoneng area, the roadway is designed from portal to the hanging wall position. From the portal, the roadway is developed parallel to the ore vein at the hanging wall side.
and then through off the ore vein on the footwall position (relatively rectangular). However in Cibitung, the roadway is started from portal to the footwall position. Figure 1.9 shows the Cibaliung underground gold mine map.

![Underground mine map](image)

Figure 1.9 Underground mine map

3. Crosscut

Crosscut is an opening that connects the main roadway to the ore zone. The crosscuts are developed perpendicular to the main roadway and strike of ore vein. The geometry of crosscut in Cibaliung underground mine is similar with its main roadway i.e. 4.2m in wide and 4.8m in height.

4. Shaft

There are many mines that use a shaft for mine workers, materials and equipment transport. However, in Cibaliung the main function of shaft is used for ventilation. The Cibaliung underground gold mine has two shafts that located in Cikoneng ore shoot and Cibitung area. Figure 1.11 shows the ventilations shaft of Cibaliung mine.

5. Stope

Stope is an excavation opening of ore. The ore is mined in the ore zone and then brought out by haulage equipment to the stockpile. The standard opening of stope in Cibaliung underground gold mine is 5m in height and 5m in wide.
1.2.3 Cibaliung Mine’s Problems and Support System Determination

Since the mine site is operated in a protected forest, the environmental impact on the forest is one of the reasons to select underground mining method at Cibaliung mine. The ore geometry is 1-15m in wide, 150-250m in length and located up to 200 to 300m in depth. The Grade of ore is about 6 g/t of gold, which is categorized them as marginal ore deposit according to gold grade in Indonesia (Table 1.2). Therefore, the strategy for the ore excavation should be prepared to achieve more economic values.

Table 1.2 Low sulphidation epithermal gold deposits (PT. CSD-Cibaliung, 2011)

<table>
<thead>
<tr>
<th>Deposit</th>
<th>Tonnage (Mt)</th>
<th>Au (g/t)</th>
<th>Au (t)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Gosowong</td>
<td>1.0</td>
<td>27</td>
<td>27</td>
</tr>
<tr>
<td>Kencana</td>
<td>1.7</td>
<td>41</td>
<td>70</td>
</tr>
<tr>
<td>Pongkor</td>
<td>6.0</td>
<td>17</td>
<td>103</td>
</tr>
<tr>
<td>LebongTandai</td>
<td>2.8</td>
<td>15</td>
<td>43</td>
</tr>
<tr>
<td>LebongDonok</td>
<td>2.9</td>
<td>14</td>
<td>41</td>
</tr>
<tr>
<td>Cibaliung</td>
<td>1.3</td>
<td>6</td>
<td>7.8</td>
</tr>
</tbody>
</table>

Cibaliung mine uses roadway as the main access to enter the underground and reach the ore. In order to cut the distance between roadways to stope, the roadway of Cibaliung underground gold mine is not only developed at the footwall side, but also at the hanging wall side. The hanging wall roadway is not common, since stress distribution is higher occurred than that on footwall. Usually the roadway is designed
spirally at the footwall. However, the roadway is developed from hanging wall side and through the ore to continue at the footwall side or the otherwise (see Figure 1.10). Squeezing is often found on the hanging wall roadway at Cibaliung mine. This squeezing might be influenced by the low rate of rock mass condition. As listed in Table 1.2, the rock properties of rock mass (rock quality) in hanging wall are lower than that in footwall. Consequently, the roadway in hanging wall has higher risk to fail than that in footwall. The picture of squeezing on hanging roadway at Cibaliung underground gold mine is given at Figure 1.11.

1.3 OBJECTIVES OF THE RESEARCH

As described in the introduction, there are problems occur in gold mines in Indonesia such as the marginal ore deposit and the ore location in the protected forest. Considering the environmental condition, several mines apply cut and fill underground mining method to excavate the ore. To obtain the production target economically, roadway at the hanging wall of cut and fill mining method is developed. Consequently, high risk of failures occurred at roadway and cross cut due to weak rock properties which usually associated with hanging wall (see Table 1.1) and high risk of failures occur at hanging wall side of stope due to high stress concentration.

From this background, the primary objective of this research is to design support system in cut and fill mining method for the roadway and cross cut that are developed at hanging wall side as well as stope.

In order to achieve the study objectives, Cibaliung underground mine in Indonesia was chosen as representative research site. Numerical analysis is proposed as a method in this study. Appropriate support system at roadway, cross cut and stope are studied using PHASE™ and FLAC3D software.
1.4 LITERATUR REVIEW

Compared with surface mining, underground mining is actually more complex in terms of the failure problems, equipment and cost. However, conducting a surface mining may require the operation on greater depth, which sometimes make surface mining become uneconomical, in addition to the environmental impact it may cause. These factors then motivate operators to select an underground mining for their mining activity.

Cut-and fill mining is one of the underground mining methods. Cut and fill mining removes ore in horizontal slices followed by drilling and blasting. The muck is then loaded and removed from the stope. When the stope has been mined out, voids are backfilled with waste rocks. The fill serves as a support the stope walls and a working platform for the equipment, when the next slice is mined. Figure 1.12 shows the cut and fill mining method.

Figure 1.11 Squeezing problems in hanging wall roadway at Cibaliung underground mine
The cut and fill mining is mostly applied for steeply dipping orebodies in stable rock masses. Depending on the orebodies shape, it can also be applied as selective mining. There are two types of cut and fill mining methods based on ore excavation directions; overhand method and underhand method.

Overhand is a method to extract the ore from bottom upwards. Intact vein is hanging over the stope and the lower void is filled with waste materials. The advantage of this method is location of backfill under the working area enable the use of backfill material with relatively low strength to reduce mining costs. However, it is difficult to extract the ore until the drilling of the main tunnel reach the bottom. In addition, this method requires longer time and higher costs for mining preparation.

Underhand method is the method to extract the ore deposit from the top downwards. The ore is drilled from the shallow level and continue by mining the deep deposits along the stope. This method enables us to obtain the ore product from the earlier stage. However, because of the backfill is at the roof of the mine worker, the stability of the backfill become a priority. Consequently, the mining cost becomes expensive. As a compromise, a combination between overhand to excavate the ore

Figure 1.12 Cut and fill underground mining method

"Fill" is some combination of tailings and cement
and continue develop the roadway at deeper area is proposed, especially for the mine with marginal deposits.

Underground mining activities pose geotechnical hazard. Problems such as squeezing, rock bursting, rock falling and subsidence usually occur in underground mining activities (Struthers et al., 2000; Mercier-Langevin and Hadjigeorgiou, 2011). The failure might occur in the development area, production area or surface in the general area. The failures is determined by the geological condition, rock masses, and the applied mining method (Hoek and Brown, 1980).

In the rock failure as well as mine subsidence, stress-strength factor plays a significant role for overall stability condition of the underground mines (Hoek and Brown, 1980). The deformation of underground might occur when the yielding of intact rock under a redistributed state of stress following excavation exceeds its strength. If this deformation takes place instantaneously, it is called rock bursting. When the deformation takes place gradually, it is termed as squeezing. Rock burst is explosive failure of rock which occurs when very high stress concentrations are induced around underground openings. The problem is very acute in deep level mining in hard brittle rock. Rock bursting occurs when the stress induced in hard rock, while squeezing occurs when the stress induced in weak rock. Squeezing conditions are caused by a variety of failure mechanisms, but are typically characterized by a reduction in the cross sectional area of an excavation as a result of a combination of induced stresses and relatively weak material properties (Potvin and Hadjigeorgiou, 2008).

Some researchers have been researched for the squeezing potential of rock around tunnels. Terzaghi (1946) is the first scientist who depicts rocks squeezing and rocks swelling. Meanwhile, Muirwood (1972) is the first scientist who proposed that the competency factor is defined as the ratio of uniaxial strength of rock to overburden stress and assess the stability of tunnels. In 1979, Nakano (1979) used the Muirwood’s parameters to recognize the squeezing potential of soft rock tunneling. Saari (1982) continued Nakano’s study and suggested the use of intensity of tangential strain of tunnels as a parameter to assess the squeezing degree of the rock. Barton et al. (1974), and Grimstad and Barton (1993) describe that the induced stress surrounding opening can reduce the Q-value of rock mass which influences the support system requirement. The relation between induced stress and rock strength of the rock mass around an underground opening is known as Stress Reduction Factor (SRF) as one of
six other parameters of Q-system rock mass classification. The SRF also can be used to predict the failure in underground mining.

Over the last decade, application of shotcrete for ground support and control in infrastructure, development and production excavation in underground mines has been increased. To improve the effectiveness of shotcrete, advances combined have been made in mix design, testing, spraying technology and admixtures. Before it is applied in underground mining, shotcrete has long been an essential part of support and reinforcement system in underground civil construction (Kovari, 2001), Rose (1985), Morgan (1992) and Franzen (1992)) reviewed the development of shotcrete technology. In underground mining, shotcrete is now used to good effect not only for infrastructure excavations in weak ground (Yumlu and Bawden, 2004), for rehabilitation, and in heavy static or pseudo-static loading conditions (Tyler and Werner, 2004), but as a component of support and reinforcement system for dynamic or rockburst conditions (Le et al. 2003, 2004). Current shotcrete support design methodology depends on very heavily upon rules of thumb and experience. Wickham et al. (1972), Bineniawski (1989), and Grimstad and Barton (1993) gave recommendation regarding shotcrete thickness based on their rock mass classifications. Shotcrete cannot prevent deformation from taking place, especially in high stress conditions. It can, however, assist in controlling deformation, particularly when used in combination with rock bolts, dowels or cables.

Rock bolts have been used for many years for the support of underground excavations and a wide variety of bolt types have been developed to meet different needs which arise in mining and civil engineering. Rock bolts generally consists of plain steel rods with a mechanical anchor at one end and a face plate and nut at the other. In applications, for short term the bolts are generally left ungrouted. While for more permanent applications in rock which corrosive groundwater is present, the space between the bolt and the rock can be filled with cement or resin grout (Hoek et al. 1993). In installation, tensioning of rock bolts is important to ensure that all of the components are in contact and that a positive force is applied to the rock. Support systems for permanent openings in mining activities required higher degree of safety than for other mine openings. These excavations are usually designed for long operational life. In some cases where corrosion is a problem, galvanized or stainless steel rock bolts have been used in attempt to control corrosion problems. However, fully grouted rock bolts are usually more effective and economical.
In order to support this research, numerical method is applied. Numerical methods have become very popular in recent years with the following various conveniences such as both stress and displacements in a block can be calculated, various constitutive relations (e.g., anisotropic, plastic, etc.) can be employed, and complex underground opening geometries can be handled and parametric studies can be conducted. Recently, there are a vast number of different numerical methods available. In general, they are categorized into two groups: continuum-based methods (e.g., the FDM (Finite Difference Method), FEM (Finite Element Method), BEM (Boundary Element Method), etc.) and discontinue-based method (e.g., distinct element codes such as UDEC, 3DEC, etc., and discontinuous deformation analysis (DDA) (Chen, 2013).

1.5 OUTLINE OF THESIS

Chapter 1 introduces the current situation of underground gold mine in Indonesia, background of this research, the importance of support system, problem statement and objectives.

Chapter 2 describes the study of support system of Pongkor underground gold mine that already well established. The discussion focuses on support system determination in roadway. This study aims to make support system guidance for Cibaliung underground mine. Thus, besides discussion of support system, this chapter also discusses similarities and differences of Pongkor underground mine and Cibaliung underground mine.

Chapter 3 describes the application of standard support system of Pongkor that developed based on mine engineers experiences and practical of Rock Mass Rating (RMR) to Cibaliung underground gold mine. The support system adoption is carried out by considering the differences of shape and position of roadway and lateral in-situ stress ratio. Based on these differences, obstacles of adoption process is discussed in order to modify Pongkor underground mine’s support system thus suitable for Cibaliung underground gold mine.

Chapter 4 describes the modification process of Pongkor underground mine’s support system. The modification was proposed based on empirical and numerical analysis. The empirical analysis was derived by combining tangential induced stress with rock mass Q-system to predict support system. The numerical analysis was carried out to
determine rock bolt density based on induced stress concentration surrounding roadway opening. The aim of this study is to design more appropriate support requirements and increase the roadway stability.

Chapter 5 describes the determination of support system at cross cut access based on support system that installed at roadway. A modification of roadway’s support system was conducted in order to achieve proper requirement of cross cut access by consideration of influence of lateral stress ratio on cross cut access and position of cross cut in regards to vein.

Chapter 6 describes the determination of support system at stope which excavated from hanging wall side. One of the effects of cross cut access in the hanging wall method is failure zone at hanging wall side’s stope increase with slice sequence of stope excavation of overhand cut and fill mining method. Since Cibaliung underground mine continues to excavate at deeper ore after a serial slice of overhand cut and fill mining method finished thus in this chapter we also discuss sill pillar application.

Chapter 7 concludes the result of this research.
CHAPTER 2

STUDY OF ESTABLISHED PONGKOR UNDERGROUND MINE’S SUPPORT SYSTEM

2.1 BACKGROUND

West part of Java Island is home to the largest gold deposits in Java Island, Indonesia. Large gold deposits could be found at Cikotok, Cikodang, Pongkor and Cibaliung (see Figure 2.1). However, Cikotok and Cikodang gold deposits have been empty since 1895 after mined by Colonial Dutch Government from 1722. Whereas, gold deposits of Pongkor and Cibaliung are still mined by ANTAM, a state mining company of Indonesia.

Pongkor and Cibaliung have similarity on: (1) stratigraphy of gold deposit (Marcoux and Milesi, 1994); (2) location near to a national park (see Figure 2.1); (3) operations under an ANTAM’s Contract of Work (COW). Thus, based on those similarities and their strong experiences on applying cut and fill method at ANTAM Pongkor Underground gold mine since 1994, which successfully reduce mining cost and environmental impact, ANTAM adopted Pongkor underground gold mining method to Cibaliung underground gold mine. The ANTAM Cibaliung underground gold mine which is started their operation at 2006. Besides discuss on support system on Pongkor Underground gold mine, this chapter discusses on similarity and differences between Pongkor underground gold mine and Cibaliung underground gold mine, and its obstacles to transfer technology of support system.

2.2 PONGKOR UNDERGROUND GOLD MINE

2.2.1 Geological Condition of Pongkor underground mine

Pongkor underground mine is located near to National Park of Gunung Halimun Salak which protected by the Government. Geologically, Pongkor area is located at the northeastern of the Bayah dome. Pongkor deposit occurs in a sequence of Tertiary igneous rocks consist of tuff breccia, lapilli tuff and intrusive andesite.
Generally footwall and hanging wall of Pongkor underground mine consists of tuff – breccia tuff. The tuff has been commonly altered by propilite and argillic at partial location (Basuki et al., 1994; Marcoux and Milesi, 1994; Milesi et al., 1999; Syafrizal et al., 2005). Pongkor deposit is gold epithermal with reserves of gold at least 2.44 Moz and grades 16.4 g/t Au in average. The mineralized body is approximately 4.2m thick, 700-2,500m long and 200m height. The strike of ore is N150°E and dip is 70-85°W. Geological structures of Pongkor are dominantly Northwest-Southeast and Northeast-Southwest direction. The common geological structures are joints filled quartz mineral, calcite, clay or combination of them or even not filled at all, and faults which always filled by clay material (Basuki, et al., 1994, Marcoux and Milesi, 1994). Recently, there are three major veins in Pongkor Gold mine that is Ciurug, Kubang Cicau and Gudang Handak. The in-situ stress regime in Pongkor is vertical stress ($\sigma_v$) = 3.88 MPa and horizontal stress ($\sigma_h$) = 3.62 MPa (Panjanita, 2005).

2.2.2 Evaluation of Rock Mass Properties in Pongkor

Rock mass is important to design underground mining and to avoid stability problems. Rock strengths that we obtained from laboratory as shown in Table 2.1
represent intact rock strength which does not consider geological structure thus it is not proper if used into numerical analysis for designing a large object like tunnels, slopes, etc. The geological character of rock material, together with the visual assessment of the mass it forms, is used as a direct input to the selection of parameters relevant for the prediction of rockmass strength and deformability.

Table 2.1. Intact rock properties of Pongkor (Pongkor laboratory test)

<table>
<thead>
<tr>
<th>Location</th>
<th>UCS (MPa)</th>
<th>E (GPa)</th>
<th>v</th>
<th>γ (MN/m³)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Kubang Cicau</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Ore</td>
<td>24-36</td>
<td>7.8-12.9</td>
<td>0.25-0.27</td>
<td>0.024</td>
</tr>
<tr>
<td>FW</td>
<td>38-53</td>
<td>6.8-11.3</td>
<td>0.24-0.25</td>
<td>0.023</td>
</tr>
<tr>
<td>HW</td>
<td>26-72</td>
<td>6.3-18.3</td>
<td>0.25-0.26</td>
<td>0.022</td>
</tr>
</tbody>
</table>

The Geological Strength Index (GSI), introduced by Hoek (1994) and Hoek, Kaiser and Bawden (1995) provides a number which, when combined with the intact rock properties, can be used for estimating the reduction in rock mass strength for different geological conditions. This system is presented in Table 2.2. During the early years of the application of the GSI system, the value of GSI was estimated directly from RMR with equation as follows:

\[
GSI = RMR_{89} - 5
\]  \hspace{1cm} (2.1)

where, \( RMR_{89} \) is the 1989 version of Bieniawski’s RMR classification (Bieniawski, 1989).

Based on the value of GSI, rock mass could be evaluated. The value of the Hoek-Brown constant for the rock mass, \( m_b \) could be estimated by using equation:

\[
m_b = m_i \left( \frac{GSI - 100}{28} \right)
\]  \hspace{1cm} (2.2)

where \( m_i \) is the value of the Hoek-Brown constant of the intact rock which can be estimated by adopting Table 2.2.

According to Table 2.2, the value of constant \( m_i \) is depend upon mineralogy, composition and grain size of the intact rock. It has been discussed on geological condition of Pongkor that the ore body consists of quartz vein. While, the country rock both hanging wall and footwall are tuff-breccia tuff. Based on Table 2.2, the values of
\( m_i \) in Pongkor should be 20 for the ore, and 13 for the country rocks of both hanging wall and footwall.

Related with Equation (2.2), when the GSI > 25 (the rock masses of good to reasonable quality) the original Hoek-Brown criterion is able to estimate the constant \( s \) and \( a \).

Table 2. 2 Values of the constant \( m_i \) for intact rock, by rock group

<table>
<thead>
<tr>
<th>Rock type</th>
<th>Class</th>
<th>Group</th>
<th>Texture</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td>Coarse</td>
</tr>
<tr>
<td>SEDIMENTARY</td>
<td>Clastic</td>
<td></td>
<td>Conglomerates (21±3)</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>Breccias (19±5)</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>Carbonates</td>
</tr>
<tr>
<td></td>
<td>Non Clastic</td>
<td>Evaporites</td>
<td>Gypsum (8±2)</td>
</tr>
<tr>
<td></td>
<td>Organic</td>
<td></td>
<td>Chalk (7±2)</td>
</tr>
<tr>
<td>METAMORPHIC</td>
<td>Non foliated</td>
<td>Marble (9±3)</td>
<td>Hornfels (19±4)</td>
</tr>
<tr>
<td></td>
<td>Slightly foliated</td>
<td>Migmatite (29±3)</td>
<td>Amphibolites (26±6)</td>
</tr>
<tr>
<td></td>
<td>Foliated**</td>
<td>Gneiss (28±5)</td>
<td>Schists (12±3)</td>
</tr>
<tr>
<td>IGNEOUS</td>
<td>Plutonic</td>
<td>Light</td>
<td>Granite (32±3)</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Dark</td>
<td>Gabbro (27±3)</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Norite (20±5)</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Hypabyssal</td>
<td>Porphyries (20±5)</td>
<td>Diabase (15±5)</td>
</tr>
<tr>
<td></td>
<td>Volcanic</td>
<td>Lava</td>
<td>Rhyolite (25±5)</td>
</tr>
<tr>
<td></td>
<td>Pyroclastic</td>
<td>Agglomerate (19±3)</td>
<td>Breccia (19±5)</td>
</tr>
</tbody>
</table>

\[
s = \exp \left( \frac{\text{GSI} - 100}{9} \right) \quad (2.3)
\]

and

\[
a = 0.5 \quad (2.4)
\]
When the GSI < 25, i.e. rock masses of very poor quality, \( s = 0 \), and \( a \) in the Hoek-Brown criterion can be estimated as following equation:

\[
a = 0.65 - \frac{GSI}{200}
\]  \hspace{1cm} (2.5)

Uniaxial compressive strength of the rock mass is the value of \( \sigma_1 \) when \( \sigma_3 \) is zero. From the Hoek-Brown criterion, when \( \sigma_3 = 0 \), it gives the uniaxial compressive strength as,

\[
\sigma_1 = \sigma_{cm} = (s\sigma_{ci})^a
\]  \hspace{1cm} (2.6)

By knowing the value which determined by using Equation (2.2) to Equation (2.6), the intact rock properties (\( E \)), tensile strength (\( \sigma_t \)), angle of friction (\( \phi' \)) and cohesive strength (\( c' \)) can be estimated by using Equation (2.7) to Equation (3.0), respectively.

\[
E_i = MR \times \sigma_{ci}
\]  \hspace{1cm} (2.7)

\[
\sigma_t = -\frac{s\sigma_{ci}}{m_b}
\]  \hspace{1cm} (2.8)

\[
\phi' = \sin^{-1}\left[\frac{6am_b(s+m_b\sigma_{3n})^{a-1}}{2(1+a)(2+a) + 6am_b(s+m_b\sigma_{3n})^{a-1}}\right]
\]  \hspace{1cm} (2.9)

\[
c' = \frac{\sigma_{ci}[(1+2a)s+(1-a)m_b\sigma_{3n})(s+m_b\sigma_{3n})^{a-1}}{(1+a)(2+a)(1+6am_b(s+m_b\sigma_{3n})^{a-1})/((1+a)(2+a))}
\]  \hspace{1cm} (2.10)

Where, \( MR \) is modulus ratio which given by Table 2.3.

The result of estimation of Pongkor rock mass properties is resumed at Table 2.4. The modulus elasticity given by Table 2.4 is much smaller than modulus elasticity given by Table 2.1. It happened because modulus elasticity of Table 2.1 does not consider the geological condition, while the modulus elasticity of Table 2.4 considers the geological condition of rock mass.
Table 2. 3 Guidelines for the selection of modulus ratio (MR) values in Equation (2.7)

<table>
<thead>
<tr>
<th>Class</th>
<th>Group</th>
<th>Texture</th>
<th>Coarse</th>
<th>Medium</th>
<th>Fine</th>
<th>Very fine</th>
</tr>
</thead>
<tbody>
<tr>
<td>Clastic</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>Conglomerates</td>
<td>300-400</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>Breccias</td>
<td>230-350</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Non-Clastic</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Carbonates</td>
<td></td>
<td>Sandstones</td>
<td>200-350</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>Siltstones</td>
<td>350-400</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>Shales</td>
<td>200-300</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>Greywackes</td>
<td>350</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>Dolomites</td>
<td>350-500</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Non-Foliated</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>Gypsum</td>
<td>(350)**</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>Anhydrite</td>
<td>(350)**</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Organic</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>Chalk</td>
<td>1000</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Slightly foliated</td>
<td></td>
<td>Amphibolites</td>
<td>400-500</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>Gneiss</td>
<td>300-750*</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Foliated*</td>
<td></td>
<td></td>
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<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>Schists</td>
<td>250-1100*</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>Schist</td>
<td>300-800*</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Plutonic</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Light</td>
<td></td>
<td>Granite</td>
<td>300-550</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>Diorite</td>
<td>300-350</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>Granodiorite</td>
<td>400-450</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Dark</td>
<td></td>
<td>Gabbro</td>
<td>400-500</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>Dolerite</td>
<td>300-400</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>Nonite</td>
<td>350-400</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Hypabyssal</td>
<td></td>
<td></td>
<td></td>
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<td></td>
<td></td>
</tr>
<tr>
<td>Porphyries</td>
<td></td>
<td></td>
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<td></td>
<td></td>
</tr>
<tr>
<td>Volcanic</td>
<td></td>
<td></td>
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<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Lava</td>
<td></td>
<td>Rhyolite</td>
<td>300-500</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>Dacite</td>
<td>350-450</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>Andesite</td>
<td>250-450</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Pyroclastic</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>Agglomerate</td>
<td>400-600</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>Volcanic breccia</td>
<td>(500)**</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>Tuff</td>
<td>200-400</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Table 2. 4 Rock mass properties of Pongkor materials

<table>
<thead>
<tr>
<th>Location</th>
<th>RMR</th>
<th>$E$ (MPa)</th>
<th>$\sigma_t$ (MPa)</th>
<th>$\phi_{\text{peak}}$</th>
<th>$c_{\text{peak}}$ (MPa)</th>
<th>$\phi_{\text{res}}$</th>
<th>$c_{\text{res}}$ (MPa)</th>
<th>$\nu$</th>
<th>$\rho$ (MN/m³)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Kubang Cicau</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Ore</td>
<td>28</td>
<td>1,035.39</td>
<td>0.004</td>
<td>28.2</td>
<td>0.93</td>
<td>14.1</td>
<td>0.47</td>
<td>0.25</td>
<td>0.024</td>
</tr>
<tr>
<td>FW</td>
<td>30</td>
<td>1,461.81</td>
<td>0.010</td>
<td>25.19</td>
<td>1.32</td>
<td>12.6</td>
<td>0.66</td>
<td>0.24</td>
<td>0.023</td>
</tr>
<tr>
<td>HW</td>
<td>30</td>
<td>1,209.17</td>
<td>0.007</td>
<td>25.19</td>
<td>0.90</td>
<td>12.6</td>
<td>0.45</td>
<td>0.25</td>
<td>0.022</td>
</tr>
</tbody>
</table>
2.3. INITIAL NUMERICAL SIMULATION MODEL

As it is known, the element type, size and boundary range can affect the simulation results for numerical model. Therefore, some significant researches about the suitable element type, size and boundary range for underground stability will be implemented. Moreover, tangential stress approach will be used for analyzing the stress distribution surrounding underground opening.

In 1898, Kirsch developed analytical solutions for the stress distribution. Other researcher following the Kirsch published solutions for excavations of various shapes in elastic plates such as Love (1927), Muskhelishvili (1953) and Savin (1961). In 1987, Brown summarized these solutions and their application in rock mechanics. Figure 2.2 shows the induced stresses surrounding the circular opening. The induced stresses consist of radial stress, tangential stress and shear stress. Some equations were developed for those induced stresses solution.

![Diagram showing stress distribution](image)

Figure 2.2 Kirsch equations for stresses in the material surrounding a circular opening

Radial

$$\sigma_{rr} = \frac{p}{2} \left[ (1 + k) \left( 1 - \frac{a^2}{r^2} \right) - (1 - k) \left( 1 - 4 \frac{a^2}{r^2} + \frac{3a^4}{r^4} \right) \cos 2\theta \right]$$ (2.11)

Tangential

$$\sigma_{\theta\theta} = \frac{p}{2} \left[ (1 + k) \left( 1 + \frac{a^2}{r^2} \right) + (1 - k) \left( 1 + \frac{3a^4}{r^4} \right) \cos 2\theta \right]$$ (2.12)
Shear stress 

\[ \tau_{r\theta} = \frac{p}{2} \left[ (1 - k) \left( 1 + \frac{2a^2}{r^2} - \frac{3a^4}{r^4} \right) \sin 2\theta \right] \]  

(2.13)

where:

- \( \sigma_{rr} \) = radial stress
- \( \sigma_{\theta\theta} \) = tangential stress
- \( \tau_{r\theta} \) = shear stress
- \( p \) = vertical stress
- \( a \) = radius of opening
- \( r \) = distance
- \( k \) = horizontal and vertical in-situ stress ratio

For \( r = a \):

\[ \sigma_{\theta\theta} = p \left[ (1 + k) + 2(1 - k)\cos 2\theta \right] \]

\[ \sigma_{rr} = 0 \]

\[ \tau_{r\theta} = 0 \]

According to the equations which mentioned above, it is understood that at the boundary of excavation \((a=r)\) only tangential stress occurs at every point. This tangential stress also acts as the major principal stress at the boundary of excavation. Therefore, this tangential stress is paramount in quantification and determination of rock damages and support system requirements.

The illustration of stresses away from the opening can be seen in Figure 2.3. The figure shows that tangential stress decreases with the increasing distance from the opening and will relatively constant after the influence of stress from opening diminishes. Contrarily, the radial stress increases with the increasing distance from opening and relatively constant after the influence of stress from the opening disappear.
2.3.1 Element Type and Boundary Size

In this research, finite element method (FEM) software, Phase\textsuperscript{2}, was used to simulate the underground stope mine in order to quantify the induced stress surrounding roadway excavation. Due to checking on validity of proposed method, the accuracy of the results by using proposed method has to be cleared by FEM. Phase\textsuperscript{2} software is a two dimensional elasto-plastic finite element program for calculating stresses and displacements around underground openings and slope, and capable used to solve a wide range of mining, geotechnical and civil engineering problems. The finite element method is a well-recognized numerical method which can be used for simulating mine induced stresses around underground excavations (Sepehri et al., 2013; Raji and Sitharam, 2011; Ahmadi et al., 2008). The method can deal with the complex geometry of the excavations, nonlinear behavior of rocks, and heterogeneity of the rock (Liu et al., 2008).

The Phase\textsuperscript{2} software provides four kinds of element types i.e. three-node triangle, six-node triangle, four-node quadrilateral and eight-node quadrilateral. In order to find a suitable element type and size, tangential stress surrounding the opening is calculated. The opening is assumed as single circular and elastic. The initial circular model was built as shown in Figure 2.4. The initial opening is built with diameter (a) 2m, and the tangential stresses are calculated for near opening on distance = diameter (r =
a) to distance = 20 times diameter ($r = 20a$) with two types of outer boundary i.e. singular boundary and rectangular boundary. The procedures of element type and boundary analyses are listed below and the result of simulation is given in Figure 2.5

- The model was built to find out the tangential stress surrounding the opening at the sidewall as a single circular opening (Phase², 1989-2008).

- In order to decrease the effect of the boundary condition on the simulation result, the boundary was set far from the opening.

- The external boundary was changed with the change of distance between stope and roadway to obtain the accurate result

- Based on the initial model, the tangential stresses were compared with the tangential stresses from Eq. 2.12.

![Initial model boundaries for element size and type analysis](image)

Figure 2. 4 Initial model boundaries for element size and type analysis
In order to evaluate a suitability of the model, the induced stresses along distance from near opening generated by simulation were compared with the tangential stresses by Kirsch equation. The total error of tangential stresses can be shown in Table 2.5. It is found that six-node triangle and eight-node quadrilateral have the lowest error on the simulations. From Figure 2.6, it is shown that the remarkable error can be recognized in the results of tangential stress when a three-node triangle or a four-node quadrilateral element types is adopted. On the other hand, the simulation results by using the six-node triangle and the eight-node quadrilateral are stable and have better accuracy. Moreover, the simulation results by using the six-node triangle and the eight-node quadrilateral are very close over increasing the outer boundary distance. When the outer distance is more than 20 times of opening, the tangential stress is close to the Kirsch equation.

<table>
<thead>
<tr>
<th>Element type</th>
<th>3-node</th>
<th>4-node</th>
<th>6-node</th>
<th>8-node</th>
</tr>
</thead>
<tbody>
<tr>
<td>% Error</td>
<td>1.18</td>
<td>1.16</td>
<td>0.83</td>
<td>0.76</td>
</tr>
</tbody>
</table>

Figure 2.5 Tangential stress simulated by different element types
Rectangular outer boundary (see Figure 2.7) was then analyzed based on the result of circular outer boundary that the distance is more than 20m. When the distance of outer boundary and opening ratio is 20 and 35, the result of tangential stress can be seen in Figure 2.8. The result error of tangential stress for each node compared with Kirsch equation can be seen in Figure 2.9 for the boundary distance 20 times of opening and Figure 2.10 for the boundary distance 35 times of opening. The value of tangential stress at boundary opening for six-node and three-node triangle element types is found close to the Kirsch equation. Moreover in this case, it is found that the error of six-node triangle element type is lower about 0.4% than that of the others (see Table 2.6). Furthermore, in case of the distance of outer boundary is 35 times of the opening, the tangential stress near to the opening of six-node triangle and eight-node quadrilateral is close to Kirsch equation, but the error of total tangential stresses of six-node triangle is lower, about 0.036% lower than that of the others (see Table 2.7). From above explanations, it is concluded that the tangential stress is more accurate when the six-node triangle is adopted with boundary distance of 35 times of the opening.
Figure 2. 7 Rectangular outer boundary

Figure 2. 8 Tangential stresses from near opening to 20m from opening with rectangular outer boundary 20x opening (A), and 35 m from opening with rectangular outer boundary 35 x opening (B)
Figure 2. 9 The error of tangential stress of rectangular outer boundary 20x opening

Table 2. 6 The percentage of error for rectangular outer boundary is 20x opening

<table>
<thead>
<tr>
<th>Element type</th>
<th>3-node</th>
<th>4-node</th>
<th>6-node</th>
<th>8-node</th>
</tr>
</thead>
<tbody>
<tr>
<td>% Error</td>
<td>0.49</td>
<td>0.92</td>
<td>0.40</td>
<td>0.47</td>
</tr>
</tbody>
</table>

Figure 2. 10 The error of tangential stress of rectangular outer boundary 35x opening
Table 2. 7 The percentage of error for rectangular outer boundary is 35x opening

<table>
<thead>
<tr>
<th>Element type</th>
<th>3-node</th>
<th>4-node</th>
<th>6-node</th>
<th>8-node</th>
</tr>
</thead>
<tbody>
<tr>
<td>% Error</td>
<td>0.095</td>
<td>0.336</td>
<td>0.036</td>
<td>0.091</td>
</tr>
</tbody>
</table>

2.3.2 Element Size

In order to support the research, the standard of element size is important to optimize the result. In this research, density of node is used as element size. The effect of density of node to tangential stress analysis was described with ratio of opening perimeter boundary to node number of the opening boundary. The result of tangential stress based on various density of node is given in Figure 2.11 for tangential stress at the sidewall and Figure 2.12 for tangential stress at the roof. The figures show that the six-node triangle gave best result when the density of node is 0.15 m/node. Therefore, it is recommended that six-node element type with distance boundary 35 times of the opening and density of node of 0.15 m/node is used to tangential stress analysis. The initial model for tangential stress analysis is given in Figure 2.13.

![Figure 2.11 Tangential stress at sidewall on various node densities](image-url)
Figure 2. 12 Tangential stress at roof on various node densities

Figure 2. 13 Initial model for tangential stress analysis
2.4. INITIAL CONDITION OF NUMERICAL ANALYSIS OF PONGKOR UNDERGROUND GOLD MINE

In this study, a minimum rock mass condition is used for analyses. The minimum rock mass classification in Pongkor mine based on RMR value is 28 at the ore and 30 at both hanging wall and footwall host rocks. The other rock mass properties are given in Table 2.4.

Initial condition of Pongkor without support is analyzed. Pongkor underground gold mine applies overhand cut and fill mining method which ramp up is developed as main roadway. The roadway developed relatively rectangular shape at footwall sides (Figure 2.14). The roadway geometry is designed 4m x 4m in width and height, and distance from stope to roadway is 20m. Ratio of in-situ stress regime between horizontal in-situ stress to vertical in-situ stress (k) of Pongkor is 1 (Pantjanita, 2005). The rock mass condition of footwall in Pongkor based on RMR is 30 which mean poor rock mass. The rock properties of Pongkor given in Table 2.4 are adopted in the numerical simulation. The initial condition of roadway of Pongkor underground gold mine is given at Figure 2.15.

![Roadway excavation with standard profile in Pongkor underground mine](image_url)

Based on the numerical analysis by Phase² software, some failures are occurred which illustrated by yield zone surrounding the roadway. It is seen at Figure 2.3 that the yield zone at the roof is about 1.3m, at the sidewall near stope is about 1.5m, at the sidewall far stope is about 1.7m, and at the floor is about 1.6m. Thus, in order to retain the openings keep stable, a support system should be installed.
2.5 SUPPORT SYSTEM OF PONGKOR UNDERGROUND MINE

The support system in Pongkor underground gold mine is constructed based on RMR rock mass classification (Bieniawski, 1989) and modified based on the engineer experiences and calculated by semi-empirical rock load (Unal, 1985) until the openings getting stable state. The roadway and cross-cut access have standard dimension as 4m in width and 4m in height with a flat roof whereas the stope dimension follows the vein width. The main access of Pongkor underground gold mine from surface to the ore location in subsurface connected by roadway as ramp up and developed in footwall side.

The support requirement in Pongkor roadway consists of two types support requirements, standard support system using H-beam and standard support system using shotcrete. The H-beam has been applied since 1994 when the mining started their excavation, while the shotcrete has just been applied on 2010. A few years ago, shotcrete is not recommended to be adopted in mining industry. This is partly due to the fact that a typical mine has many working faces and it is difficult to schedule the shotcreting equipment efficiently. However, as advancing the technology of shotcreting and considering shotcrete is generally stronger than mesh in regards to prevent small pieces of rock from unraveling from the surface of an excavation and it is corrosion
resistant, shotcrete is recently considered to be a more effective support system useful in excavations such as ramps and haulages where long-term stability is important.

The support recommendation of Pongkor underground gold mine is given in Table 2.8. Whereas, the properties of rock bolt, shotcrete and H-beam is given in Table 2.9, Table 2.10, and Table 2.11 respectively. Rock bolt of roadway is galvanized type consists of 1.4m and 1.8m in length operated by Jackleg, and rock bolt of 2.4m in length operated by Jumbo Drill. Shotcrete divided into two types based on its operational mechanism; manual and mechanical. The standard for compression strength value (UCS) shotcrete is 35 MPa after 28 days.

Table 2.8 Support recommendation of Pongkor underground gold mine

<table>
<thead>
<tr>
<th>Rock mass class</th>
<th>RMR</th>
<th>Ground support</th>
</tr>
</thead>
<tbody>
<tr>
<td>I</td>
<td>80-100</td>
<td>Spot bolt</td>
</tr>
<tr>
<td>II</td>
<td>50-80</td>
<td>Rock bolt (spacing 1m, 1.8m in length) with mesh and strap (2m)</td>
</tr>
<tr>
<td>III</td>
<td>30-50</td>
<td>Rock bolt (spacing 1m, 1.8m in length) with mesh and H-Beam (spacing 1-2m) or shotcrete (5cm thickness)</td>
</tr>
<tr>
<td>IV</td>
<td>0-30</td>
<td>Rock bolt (spacing 1m, 1.8m in length) with mesh and H-Beam (spacing &lt;1m) or shotcrete (10cm thickness)</td>
</tr>
</tbody>
</table>

Table 2.9 Rock bolt properties

<table>
<thead>
<tr>
<th></th>
<th>1.4</th>
<th>1.8</th>
<th>2.4</th>
</tr>
</thead>
<tbody>
<tr>
<td>Length (m)</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Diameter (mm)</td>
<td>33</td>
<td>46</td>
<td>46</td>
</tr>
<tr>
<td>Tensile Capacity (kN)</td>
<td>120</td>
<td>133</td>
<td>178</td>
</tr>
<tr>
<td>Minimum Tensile Capacity (kN)</td>
<td>71</td>
<td>107</td>
<td>120</td>
</tr>
<tr>
<td>Bolt Modulus (MPa)</td>
<td>200,000</td>
<td>200,000</td>
<td>200,000</td>
</tr>
<tr>
<td>Bond Shear Stifness (MN/m/m)</td>
<td>12,000</td>
<td>12,000</td>
<td>12,000</td>
</tr>
<tr>
<td>Yield Strength (MPa)</td>
<td>415</td>
<td>510</td>
<td>554</td>
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</tbody>
</table>
Table 2.10 Shotcrete properties

<table>
<thead>
<tr>
<th>Young Modulus</th>
<th>Poisson Ratio</th>
<th>Compressive Strength</th>
<th>Tensile Yield</th>
<th>Residual Yield</th>
</tr>
</thead>
<tbody>
<tr>
<td>21 GPa</td>
<td>0.15</td>
<td>35 MPa</td>
<td>20 MPa</td>
<td>10 MPa</td>
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</tbody>
</table>

Table 2.11 H-beam properties

<table>
<thead>
<tr>
<th>Dimensions</th>
<th>Section depth (m)</th>
<th>Area (m²)</th>
<th>Moment inertia (m⁴)</th>
<th>Young modulus (MPa)</th>
<th>Poison ratio</th>
<th>Unit weight (kg/m)</th>
<th>Compr strength (MPa)</th>
<th>Tensile strength (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>200x200</td>
<td>0.2</td>
<td>0.0064</td>
<td>4.72e-5</td>
<td>200,000</td>
<td>0.25-0.3</td>
<td>50.4</td>
<td>400</td>
<td>400</td>
</tr>
</tbody>
</table>

In this study, two types of support system which applied by Pongkor underground gold mine are analyzed by numerical modeling. Considering to its properties of rock (see Table 2.4), the rock quality of Pongkor could be categorized as class IV in regards to RMR (see Table 2.5). Based on Table 2.8, ground support required for rock mass class IV is rock bolt spacing 1m, 1.8m in length with mesh, and H-beam (spacing<1 m) or rock bolt space 1m, 1.8m in length with mesh, and shotcrete (10cm). Regarding the H-beam system, Pongkor underground gold mine has been applying rock bolt, space 1m, 1.8m in length with mesh, and H-beam (spacing 0.6m) for sidewall and roof, and shotcrete with thickness 20cm at the floor to prevent floor from heaving. Regarding shotcrete system, Pongkor underground gold mine has been applying rock bolt spacing 1m, 1.8m in length with mesh, and shotcrete (10cm thickness), and at the floor 20cm thickness of shotcrete is installed. Designs of Pongkor support system is given in Figure 2.16.
The numerical simulation of H-beam system and shotcrete system are given at Figure 2.17 and Figure 2.18 respectively. The stability of opening is analyzed in regards to yield zone on numerical simulation. The yield zone around opening without support system and with support system (H-beam system as well as shotcrete system) in order to show the role of support system as well as the difference of H-beam system and shotcrete system is resumed in Table 2.12.

Figure 2.17 Pongkor roadway supported by H-Beam (spacing 0.6m) with rock bolt, spacing 1m, 1.8m in length and mesh

Figure 2.18 Pongkor roadway supported by shotcrete (10cm thickness) with rock bolt, spacing 1m, 1.8m in length and mesh
It is shown at Table 2.12 that the support system could improve the yield zone which also means improvement of the stability of the openings. Regarding the simulation of H-beam system, condition of the H-beam liner is stable and no failure occurred. Liner failure occurred only in the shotcrete at the corner of the floor area. Tension failure of rock bolt is not occurred however a few of shear failure of rock bolt could be found (see Figure 2.17). Meanwhile, liner shotcrete yield occur at the corner of roof and the corner of floor on numerical simulation of shotcrete system. Moreover, tension failure of rock bolt is not occurred however shear failure just occurred at rock bolts. Based on above results, it can be said that roadway of Pongkor underground mine is stable and the support system of Pongkor is applicable.

2.6 COMPARISON OF PONGKOR UNDERGROUND MINE AND CIBALIUNG UNDERGROUND MINE

As one of establish underground mine, Pongkor underground mine become a guidance to other underground mines in Indonesia including Cibaliung underground gold mine. Cibaliung and Pongkor underground mines operated under same mine company PT. Antam, a government mine company. The ore type of Cibaliung and Pongkor underground mines is low sulphidation epithermal gold deposit. Moreover, the locations of both mines are in protected forest. Based on above similarities, Pongkor underground mining method is adopted on Cibaliung underground mine. The adoption consists of:

- Rock mass classification is determined based on RMR
- Cut and fill mining method is applied.
- Support system of Cibaliung underground mine is adopted from standard support system of Pongkor underground mine.

Regarding the application of support system, there are different conditions between Cibaliung and Pongkor underground mines that should be considered.

- As marginal deposit, Cibaliung should develop profitable mining system. In order the economically reason, uncommon roadway is developed in hanging wall side. Whereas, Pongkor underground gold mine develops roadway in footwall side.
- Indication high lateral in-situ stress condition in Cibaliung that ratio between horizontal to vertical in-situ stress \( k = 2 \). Whereas, ratio horizontal and vertical in-situ stress in Pongkor \( k = 1 \).
- Roadway design in Cibaliung underground mine is horseshoe shape, whereas the roadways of Pongkor underground mine is rectangular shape.

Above differences can influence performance of support system. Therefore, application of support system of Pongkor underground mine to Cibaliung underground mine should be evaluated.

2.7 CONCLUSION

As it is known, the element type, size and boundary range can affect the simulation results for numerical analysis. Therefore, some significant researches about the suitable element type, size and boundary range for induced stress analysis has been done. According to the analysis above, it is recommended that underground model should be built by using six-node triangle element type with density of node 0.15 m/node was recommended to optimize element size. Moreover, it is found that the outer boundary should be more than 35 times of the opening width.

Roadway excavation in cut and fill underground mining plays an important role for the life of mines. The instability of roadway disrupts the productivities of mines. Pongkor is one of the oldest mine that still operating in Indonesia, and the support system standard of Pongkor has been established very well and becomes guidance for other underground mines in Indonesia. Pongkor underground gold mine roadway is located in footwall side.
Without support, yield zone of Pongkor underground gold mine’s roadway is about 1.3m to 1.7m. Two types of support systems are applied in Pongkor underground gold mine i.e. H-Beam and Shotcrete. Stability of Pongkor underground gold mine’s roadway increases after the roadway is supported. By using H-beam, yield zone surrounding roadway opening decreases 0.4m to 0.6m. Similar results also obtained after the roadway of Pongkor underground gold mine is supported by 10cm thickness of shotcrete. The yield zone decreases about 0.3m to 0.6m. Liner supports are relatively stable with a few failures occurred only at the corner of roadway. Moreover, rock bolt support has no tension failure. Therefore, it can be said that the standard support system is capable for Pongkor underground gold mine.

As one of established underground mines, Pongkor underground mine become a guidance to other underground mines which has strong similarity in ore type sulphidation like Cibaliung underground gold mine. However, due to differences on ore grade and lateral in-situ stress ratio, the adoption of Pongkor underground mine method should be firstly evaluated before applied in Cibaliung underground mine.
CHAPTER 3

APPLICATION OF PONGKOR’S SUPPORT SYSTEM ON CIBALIUNG UNDERGROUND MINE

In Chapter 2, study on determination of Pongkor underground mine’s support system has been discussed. Because the Pongkor underground mine’s support system standard is used at Cibaliung underground mine, similarities and differences conditions of Pongkor and Cibaliung underground mining has also been discussed. In regards to differences on shape of roadways, position of roadways relative to ore, and lateral in-situ stress ratio, thus application of Pongkor underground mine’s support system on Cibaliung underground mine should be evaluated.

3.1 INFLUENCE OF SHAPE OF UNDERGROUND OPENINGS ON STRESS DISTRIBUTION AROUND OPENING

The stability of roadways is affected by its shape, size of opening, in-situ stress and geological condition. Some underground mines develop the roadway in horseshoe shape. The horseshoe shape provides a wide flat floor for equipment and also provides a pleasant working platform. Hoek and Brown (1980) explained influence of excavation shape to induced stress. The excavation with corners has high compressive stress concentration in these locations. The effect of excavation shape that the highest potential of instability in underground excavation is located in corner of opening. Hence there is a tendency to increase the radius of curvature in the design of underground excavations, to avoid overstressing of the rock mass.

In this research, horseshoe shape of roadway is analyzed. Condition of this shape is compared with the rectangular shape. Pongkor underground gold mine’s properties are used as parameters on this numerical simulation. This study is useful for Cibaliung mine because it develops roadway with horseshoe shape. Initial model of horseshoe shape of Pongkor roadway can be seen in Figure 3.1.
Based on the result as given in Figure 3.2, yield zone at the roof of horseshoe shape is about 0.7m, whereas at the other sides are about 1.6m. Compared with the rectangular shape, the yield zone area of horseshoe shape at the roof is smaller than that yield zone of rectangular. At other locations the failure conditions are similar for both shapes.

In order to lucid the effect of support system on horseshoe shape on the stress distribution, the horseshoe shape of this study is installed of support system. Application of standard support system is similar with the condition of rectangular shape of roadway of Pongkor underground gold mine. It consists of two type support
requirement as class IV in Table 2.5. The result analyses of standard support system of Pongkor for horseshoe shape are given in Figure 3.3 for H-beam main support requirement and Figure 3.4 for shotcrete main support requirement.

![Image](image.png)

**Figure 3.3** Failure of horseshoe shape roadway excavation supported by H-Beam (spacing 0.6m) with rock bolt, spacing 1m, 1.8m in length, and mesh

When the horseshoe shape of roadway is supported by H-beam appropriate as Class IV, the stability increased significantly. Yield zones surrounding the roadway excavation decrease. In general there is no yield zone occurs at the roof. A few of yield zone can be found at the both of sidewall about 0.2m. The yield zone is occurred at the floor of roadway about 0.5cm.

Relatively similar result is given in Figure 3.4 that roadway with horseshoe shape is supported by shotcrete. The yield zone also decreased significantly, at the roof that there is no yield zone occurred. The yield zone at the sidewall near stope is about 0.5m, at the sidewall far stope is about 0.3m and yield zone at the floor about 0.6m. Based on these conditions, the roadway of Pongkor that simulated by horseshoe shape and supported by both H-beam and shotcrete can be said stable. This analysis gives better result of stability at the roof and relatively similar result at other sides compared with the standard rectangular shape of Pongkor roadway.

In order to show the role of support system as well as the difference of H-beam system and shotcrete system on horseshoe shape opening is resumed in Table 3.1. It
is shown at Table 3.1 that the support system could improve the yield zone which also means improvement of the stability of the openings.

![Figure 3.4 Failure of horseshoe shape of Pongkor roadway supported by shotcrete 10cm thickness with rock bolt, space 1m, 1.8m in length and mesh](image)

<table>
<thead>
<tr>
<th>Yield Zone (m)</th>
<th>Support system</th>
<th>Without</th>
<th>H-Beam system</th>
<th>Shotcrete system</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Roof</td>
<td>0.680</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td></td>
<td>Sidewall:</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>Far stope</td>
<td>1.581</td>
<td>0.225</td>
<td>0.382</td>
</tr>
<tr>
<td></td>
<td>Near stope</td>
<td>1.697</td>
<td>0.228</td>
<td>0.582</td>
</tr>
<tr>
<td></td>
<td>Floor</td>
<td>1.561</td>
<td>0.504</td>
<td>0.622</td>
</tr>
</tbody>
</table>

Regarding the simulation of horseshoe shape opening, as well as condition of the H-beam liner, condition of shotcrete liner is relatively stable. Meanwhile, liner shotcrete yield occur at the corner of floor. Moreover, tension failure of rock bolt is not occurred however a few of shear failure of rock bolt could be found. Based on above analysis, the horseshoe shape gives an advantages that failure zone is lower than that failure zone of rectangular shape. It is because the excavation with corner will produce overstress of the rock mass and subsequently causing instability around the corner.
3.2 INFLUENCE OF LATERAL IN-SITU STRESS RATIO

In order to understand the influence of in-situ stress regime, Pongkor underground mine is simulated by lateral in-situ stress ratio of ‘k’=2. The numerical analysis is done based on condition of lateral in-situ stress ratio of ‘k’=1 which has been discussed in subchapter 2.4. The result of numerical analysis shows that the failure zone of ‘k’=2 is higher than ‘k’=1. The failure zone of lateral in-situ stress ratio of ‘k’=2 can be seen at Figure 3.5. While failure zone of lateral in-situ stress of ‘k’=1 has been displayed at Figure 2.5 and Figure 2.6. Figure 3.5 shows that the failure zone at the roof and floor and sidewall near stope of H-beam is significantly smaller than that of shotcrete. While, the failure zone at sidewall far stope of H-beam is slightly smaller than that of shotcrete. High failure zone at roof, floor and sidewall far stope is caused by high horizontal stress. Meanwhile, increasing failure zone at sidewall near stope is more influenced by induced stress of stope. It indicates H-beam is more resists to horizontal stress compared with shotcrete particularly H-beam at the roof. It is in line with beam theory. According to beam theory the web of H-beam resists shear force, while flanges of H-beam resist most of the bending moment experienced by the beam. For lateral stress ratio of ‘k’=2, H-beam is better than shotcrete because H-beam has stronger resistance to shear force and bending moment compared with shotcrete. Figure 3.6 shows displacement vector of H-beam is higher than that of shotcrete. However, the failure zone of shotcrete is larger than that of H-beam.

Based on above discussion, we should give more attention to failure zone especially at roof, floor and sidewall near stope if lateral stress ratio increase or/and the support system is shotcrete.
3.3 INSTALLATION OF PONGKOR UNDERGROUND MINE’S SUPPORT SYSTEM ON CIBALIUNG UNDERGROUND MINE

3.3.1 Installation Support System at Hanging Wall

Based on geological condition, the host rock of Cibaliung has correlation with the host rock of Pongkor (Marcoux and Milesi, 1994). The ore of both mines are classified as low epithermal sulphidation type and to excavate the ore Cibaliung underground mine applies overhand cut and fill mining method. In Cibaliung
underground mine roadway is developed as main access for mine workers, materials and equipment transport. This method is similar with Pongkor underground mining method. The roadway of Cibaliung is developed as ramp down and the locations are not only in footwall but also in hanging wall side. Moreover, the rock mass condition of roadway in hanging wall of Cibaliung mine based on RMR classification has similar condition with the rock mass of Pongkor. The minimum RMR of Cibaliung is 30. Initial condition of roadway of Cibaliung underground mine that located in hanging wall side is given in Figure 3.7. The width of roadway is 4.2m and height is 4.8m. Distance between roadway and stope is 40m. Ratio between horizontal to vertical in-situ stress \((k)\) is 2.

Rock mass condition influences stability of underground mine and also to determine the support requirements. Rock Mass Rating (RMR) classification is widely used for early support requirement analysis. The summaries of RMR classification of hanging wall and footwall rock in Cibaliung Underground Mine is given in Table 3.2 and 3.3 respectively. Rock mass strength of Cibaliung is calculated by using Rocklab software, a same method with Pongkor. The rock mass strength properties of Cibaliung are given in Table 3.4. Without support system, the result of numerical analysis of roadway can be seen in Figure 3.8.

Table 3.2 The summaries of RMR classification for worst and best rock mass condition in hanging wall

<table>
<thead>
<tr>
<th>Parameters</th>
<th>Description</th>
<th>Rating</th>
<th>Description</th>
<th>Rating</th>
</tr>
</thead>
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<tr>
<td>Qualitative Condition</td>
<td>Worst</td>
<td></td>
<td>Best</td>
<td></td>
</tr>
<tr>
<td>(\sigma_0), MPa</td>
<td>24</td>
<td>2</td>
<td>24</td>
<td>2</td>
</tr>
<tr>
<td>RQD (%)</td>
<td>55</td>
<td>13</td>
<td>55</td>
<td>13</td>
</tr>
<tr>
<td>Spacing of discontinuous (cm)</td>
<td>4-10</td>
<td>8</td>
<td>4-10</td>
<td>8</td>
</tr>
<tr>
<td>Condition of discontinuous</td>
<td>Slickenslide to slightly rough surfaces, high weathered walls</td>
<td>10</td>
<td>Slightly rough surfaces</td>
<td>19</td>
</tr>
<tr>
<td>Groundwater</td>
<td>Wet to damp</td>
<td>9</td>
<td>Dry</td>
<td>13</td>
</tr>
<tr>
<td>Effect discontinuous</td>
<td>Unfavourable</td>
<td>-12</td>
<td>Unfavourable</td>
<td>-7</td>
</tr>
<tr>
<td>Total RMR</td>
<td></td>
<td>30</td>
<td></td>
<td>48</td>
</tr>
<tr>
<td>Classification</td>
<td></td>
<td>Poor</td>
<td></td>
<td>Fair</td>
</tr>
</tbody>
</table>
Table 3. 3 The summaries of RMR classification for worst and best rock mass condition in footwall

<table>
<thead>
<tr>
<th>Parameters</th>
<th>Description</th>
<th>Rating</th>
<th>Description</th>
<th>Rating</th>
</tr>
</thead>
<tbody>
<tr>
<td>Qualitative Condition</td>
<td>Worst</td>
<td></td>
<td>Best</td>
<td></td>
</tr>
<tr>
<td>(\sigma_c), MPa</td>
<td>51</td>
<td>7</td>
<td>51</td>
<td>7</td>
</tr>
<tr>
<td>RQD (%)</td>
<td>60</td>
<td>13</td>
<td>65</td>
<td>13</td>
</tr>
<tr>
<td>Spacing of discontinuous (cm)</td>
<td>20-50</td>
<td>9</td>
<td>20-60</td>
<td>10</td>
</tr>
<tr>
<td>Condition of discontinuous</td>
<td>Slightly rough surfaces</td>
<td>20</td>
<td>Slightly rough surfaces</td>
<td>20</td>
</tr>
<tr>
<td>Groundwater</td>
<td>Wet to damp</td>
<td>7</td>
<td>Damp to dry</td>
<td>13</td>
</tr>
<tr>
<td>Effect discontinuous</td>
<td>Unfavourable</td>
<td>-8</td>
<td>Favourable</td>
<td>-2</td>
</tr>
<tr>
<td>Total RMR</td>
<td></td>
<td>48</td>
<td>61</td>
<td></td>
</tr>
<tr>
<td>Classification</td>
<td></td>
<td>Fair</td>
<td>Good</td>
<td></td>
</tr>
</tbody>
</table>

Table 3. 4 Mechanical and physical rock mass properties

<table>
<thead>
<tr>
<th></th>
<th>(RMR)</th>
<th>(E) (MPa)</th>
<th>(\nu)</th>
<th>(c) (MPa) (\text{peak})</th>
<th>(\phi) ((\circ)) (\text{peak})</th>
<th>(\sigma_t) (MPa)</th>
<th>(\rho) (gr/cm(^3))</th>
<th>(c) (MPa) (\text{res.})</th>
<th>(\phi) ((\circ)) (\text{res})</th>
<th>(\omega) ((\circ))</th>
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</thead>
<tbody>
<tr>
<td>Breccia Smectite</td>
<td>30</td>
<td>1,161.7</td>
<td>0.25</td>
<td>1.054</td>
<td>30.8</td>
<td>0.003</td>
<td>2.5</td>
<td>0.5</td>
<td>15</td>
<td>3.63</td>
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<td></td>
<td>48</td>
<td>3,274.2</td>
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<td>1.430</td>
<td>36.3</td>
<td>0.013</td>
<td>2.5</td>
<td>0.7</td>
<td>18</td>
<td>3.63</td>
</tr>
<tr>
<td>Breccia Chlorite</td>
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<td>3.038</td>
<td>36.33</td>
<td>0.028</td>
<td>2.58</td>
<td>1.5</td>
<td>17.5</td>
<td>15.2</td>
</tr>
<tr>
<td></td>
<td>61</td>
<td>10,087.5</td>
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<td>3.630</td>
<td>40.25</td>
<td>0.074</td>
<td>2.58</td>
<td>1.8</td>
<td>20.1</td>
<td>15.2</td>
</tr>
<tr>
<td>Ore</td>
<td>67</td>
<td>10,223.9</td>
<td>0.35</td>
<td>5.06</td>
<td>39.35</td>
<td>0.086</td>
<td>2.7</td>
<td>2.5</td>
<td>19.1</td>
<td>13.2</td>
</tr>
</tbody>
</table>
Figure 3. 7 Initial model of hanging wall roadway of Cibaliung underground mine

Figure 3. 8 Failure and displacement of hanging wall roadway of Cibaliung underground mining with no support
Maximal displacement of hanging wall roadway without supported occurs at the floor 3.1-5.8cm, at the roof displacement around 3.0-5.0cm, and at the sidewall near stope displacement about 3.2cm whereas at the sidewall far stope the displacement about 3.1-3.3cm. Based on yield zone, hanging wall roadway of Cibaliung underground gold mine is more likely to failure at the floor and the roof because of the high lateral in-situ stress regime. The yield zone at the floor is more than 5.4m and at the roof is about 2.7m. However, the yield zones at the sidewall relatively small, at bottom of the sidewall yield zone are wide. At the sidewall near stope the yield zone is about 1.5m whereas at the sidewall far stope is about 1.4m. High failure zone at the roof and floor is caused by high horizontal stress. Based on this simulation, the hanging wall roadway of Cibaliung underground gold mine requires support to increase the stability.

Figure 3. 9 Failure and displacement of hanging wall roadway of Cibaliung mine supported by H-beam (spacing 0.6m) and rock bolt 1.8m in length, spacing 1m and mesh
As mentioned above, minimum rock mass condition of Cibaliung (RMR=30) is relatively similar with Pongkor condition. Standard support Class IV of Pongkor underground gold mine is then applied to analysis Cibaliung underground gold mine’s roadway. Analysis result of H-beam supported is given in Figure 3.9. After supported, the stability of roadway increases however failure still occurred. The yield zone at side wall near stope is about 0.8m, at sidewall far stope is about 0.7m and the yield zone at the floor is about 2.9m. Even though the yield zone does not occur at the roof, some rock bolt failures occur as combination shear and tensile.

Another support requirement that consists of shotcrete as main material is analyzed similar with Pongkor underground gold mine standard support system (Class IV). The shotcrete thickness is 10cm, rock bolt 1.8m of length and space 1m are adopted. The result of this analysis is given in Figure 3.10. In contrast with H-beam supported, the yield zone still occurs about 1.7m at the roof. It indicates H-beam is more resists to horizontal stress compared with shotcrete particularly H-beam at the roof. The yield zone at sidewalk near stope is about 1.3m, and about 0.9m at sidewalk far stope. High yield zone still occurs at the floor of roadway is about 3.6m. Liner failure also occurred at the roof, bottom of both sidewalk and at the floor as well as some rock bolt. Displacement at the roof of hanging wall roadway is about 1.0-1.7cm. Displacement at the sidewalk near stope as well as far stope is about 1.9–2.8cm whereas displacement at the floor is about 1.2-4.3cm.

Displacement measurement has been conducted at the hanging wall roadway of Cibaliung underground mine that was supported by shotcrete 10cm thickness and rock bolt 1.8m in length with spacing 1m as same as Pongkor standard support system. Extensometer was used for measuring the displacement. Displacement measurement only conducted at the sidewalk since ventilation box is installed at the roof. The result is given in Figure 3.11. The displacement of roadway is about 55mm in 40 days and still tends to move. The displacement is inward direction to the roadway opening and reduces the width of roadway. According to the displacement measurement, the condition of the roadway was not stable. The result is similar with numerical modelling analysis which shows that the displacement in one side is about 28mm. It can be said that the numerical modelling analysis is applicable for this research.
From the result, when Cibaliung underground gold mine applies the support system based on the standard support of Pongkor underground gold mine, a failure in the hanging wall roadway of Cibaliung still occurs. The reason is because in-situ stress of Cibaliung underground gold mine is high which ratio between horizontal and vertical in-situ stress \((k)\) is 2. Thus it might give impact to failure at the roof and floor. Moreover, some failures also occur at rock bolt as shear and tensile. Therefore, the hanging wall roadway of Cibaliung underground mine still requires support system to improve the stability.

Figure 3. 10 Failure and displacement of hanging wall roadway of Cibaliung mine supported by shotcrete (10cm thickness) and rock bolt 1.8m in length, spacing 1m, and mesh
Displacement of sidewall at hanging wall roadway of Cibaliung mine supported by shotcrete (10cm thickness), rock bolt 1.8m in length with spacing 1m, and mesh.

3.3.2 Installation Support System of Roadway at Footwall

The roadway of Cibaliung underground mine in the footwall side is more stable than the roadway in hanging wall side. The worst rock mass condition of footwall of Cibaliung can be classified as good rock mass (RMR=48). According to Pongkor standard support system, footwall of Cibaliung underground mine can be supported by standard Class III that the support requirements consist of rock bolt spacing 1m, 1.8m in length with mesh and H-Beam (spacing 1-2m) or shotcrete (5cm thickness). Initial condition of footwall roadway of Cibaliung underground gold mine can be seen in Figure 3.12 and analysis result of the footwall roadway without support can be seen in Figure 3.13.
Figure 3. 12 Initial model of footwall roadway of Cibaliung underground mine

Figure 3. 13 Failure and displacement of footwall roadway of Cibaliung underground mining with no support
Condition of footwall roadway without support shows some failures occur at the roof and floor. Condition at the sidewall relatively stable even though small yield zone occurs at the bottom of sidewall. High potential to failure at the roof and floor is because of the high horizontal in-situ stress regime. Displacement at the roof is about 0.2-0.9cm, at the sidewall far stope is about 0.8-1.0cm, and displacement at the sidewall near stope is about 0.2cm whereas at the floor displacement is about 0.2-0.8cm.

Figure 3.14 shows the roadway supported by H-beam. The yield zone occurred at the both sidewalls and at the floor, whereas yield zone does not occur at the roof. The yield zone at the sidewall far stope is about 0.3m whereas at the sidewall near stope is about 0.4m and the yield zone at the floor is about 0.6m. Displacement at the roof is about 0.3-0.8cm, displacement at the sidewall far stope is about 0.6-0.9cm, and displacement at the sidewall near stope is about 0.2-0.3cm whereas displacement at the floor is about 0.5-0.6cm. Condition of footwall roadway that supported by H-beam is relatively stable. It is shown no failure at liner and only a few of failures occur at the rock bolt.

Numerical result of footwall roadway that supported by shotcrete is given in Figure 3.15. The result is relatively similar with the H-beam support requirement. There is no yield zone occurred at the roof. Small yield zone occurs at sidewall far stope about 0.7m and about 0.8m at the sidewall near stope. At the floor, yield zone is occurred about 1.1m. Small displacement is found surrounding the roadway. Displacement at the roof is about 0.3-0.8cm whereas displacement about 0.8-0.9cm, 0.2cm and about 0.3-0.6cm are occurred at the sidewall far stope, sidewall near stope and at the floor, respectively. Moreover, failure at the rock bolt and shotcrete liner surrounding the footwall roadway relatively not occurs. It is clear that Class III of standard support system Pongkor underground gold mine is appropriate to be applied at footwall roadway of Cibaliung underground gold mine.
Figure 3.14 Failure and displacement of footwall roadway of Cibaliung underground gold mine supported by H-beam (spacing 1.5m), and rock bolt 1.8m in length, spacing 1m, and mesh
Figure 3.15 Failure and displacement of footwall roadway of Cibaliung underground gold mine supported by shotcrete (5cm thickness), and rock bolt 1.8m in length, spacing 1m, and mesh

3.3.3 Discussion of Installation Results

Some similarities become reasons for Cibaliung underground mine to adopt the Pongkor’s support system. However, there are differences between the mines that influence the performance of support system. According to the results discussed above, it is found that support system of Pongkor underground mine is not sufficient to apply at hanging wall of roadway of Cibaliung underground mine.

Cibaliung underground mine develops roadway with horseshoe shape whereas Pongkor develops rectangular shape. Based on numerical analysis, the shape of horseshoe shape gives an advantage to reduce the failure zone especially at the roof compared to a rectangular shape.
Cibaliung underground mine has high lateral in-situ stress with ratio between lateral to vertical in-situ stress ‘k’=2, whereas Pongkor underground mine has in-situ stress ratio relatively similar (‘k’=1). This different condition gives high influence to the performance of support system. Based on analysis, if the ratio of in-situ stress of Pongkor is ‘k = 2’ as same as Cibaliung’s in-situ stress, high failures are occurred especially at the roof and floor, and the support system is not sufficient.

Cibaliung underground mine develops roadway at hanging wall side where the stress concentration is high whereas Pongkor develops roadway at footwall side. Moreover, condition of hanging wall of Cibaliung underground mine consists of weak rock. Based on the result, it is found that roadway of Cibaliung underground mine at footwall side is stable when the support system of Pongkor underground mine is applied. Meanwhile, high failures are occurred at the hanging wall of roadway of Cibaliung. Based on the results, Pongkor underground mine’s support system should be modified if we want to apply this support system on Cibaliung underground mine.

3.4 CONCLUSION

Cibaliung underground mine applies similar method with Pongkor. They have been applying cut and fill mining method and excavate the ore from bottom to top (overhand). Moreover, both of mines are operated under same company. Those become a reason for Cibaliung to apply similar support requirement. However several differences are occurred between Pongkor underground mine and Cibaliung underground mine.

Studies of horseshoe shape roadway give better result of roadway stability. By using H-beam support requirement, yield zone decreases significantly. There is no yield zone occur at the roof. However, yield zone at the sidewalls are about 0.2m and at the floor is about 0.5m. Similar results also occur for the roadway that supported by 10cm thickness of shotcrete. There is no yield zone at the roof. However, at other sides is about 0.4m to 0.6m. Based on these results standard support system of Pongkor underground gold mine is also capable to apply for horseshoe shape roadway.

In Pongkor underground gold mine, ratio between lateral stress to vertical stress is relatively similar (‘k’=1) and the support systems has been established to apply in Pongkor underground mine. However, based on analysis of influence of lateral
in-situ stress ratio, if the Pongkor underground mine has high lateral stress ‘k’=2, high failures are occurred especially at the roof and footwall sides.

After Cibaliung underground mine adopted Pongkor underground mine’s support requirement, the yield zone is still huge. Moreover, liner failures and bolts failures are occurred after the roadway supported. These incompatibilities of support designs occur because different condition between Cibaliung and Pongkor underground gold mines. In Cibaliung underground gold mine, ratio between lateral stress to vertical stress ‘k’ = 2 which mean lateral in-situ stress is higher than vertical in-situ stress and lead high failures at the roof and floor. Moreover, Cibaliung underground gold mine develops a roadway at the hanging wall side that stress concentrations occurred, and the hanging wall side of Cibaliung has poor rock mass properties.

Based on above analyses, standard support system of Pongkor cannot be said suitable to apply in Cibaliung underground mine. High failures that still occur in the roadway of Cibaliung underground mine become reasons for Cibaliung to develop more support requirements. Therefore, in the next chapter, modified support system of Pongkor underground mine is paramount when the support system is applied at hanging wall roadway of Cibaliung underground mine.
CHAPTER 4

HANGING WALL ROADWAY SUPPORT DESIGN OF CIBALIUNG UNDERGROUND MINE

In Chapter 3, support system of Cibaliung underground mine has been discussed based on Pongkor underground mine. Even though both mines Cibaliung and Pongkor have relationships on host rock and apply similar method of mining, but Cibaliung underground mine has high lateral in-situ stress, the roadway is developed in hanging wall side and consists of weak rock. Based on simulation in Chapter 3, the roadway of Cibaliung still potential to failure after applying the support system of Pongkor underground mine, consequently the standard support of Pongkor underground mine should be modified to apply in Cibaliung. In this chapter, empirical and numerical methods are studied to find an ideal support system.

4.1 MODEL CONSTRUCTION

To obtain good results, mesh type, element type, and element length were set to consider each models based on the capacities of the computer as explained in Chapter 2. The six-node triangle element with distance boundary 35 times the opening is applied to obtain the tangential stress. The boundary conditions were set as follows: the sides of the model were restrained perpendicular to each side, whereas the bottom of the model was restricted in the vertical direction, and the top surface was free.

The induced stresses are classified into three types, i.e. tangential stress, radial stress and shear stress. The tangential stress is the dominant stress that acts as the major principal stress at the boundary of excavation. It is known that the stability of roadways is influenced by in-situ stress condition, stope dimension, the distance from stope openings to the roadway and depth of opening from the surface. In the model of the study, the stope size is designed as 5m x 5m. In regards to the distance of roadway from the stope, the roadway is designed to be located at 5m, 10m, 20m, and 40m distance from the stope (see Figure 4.1).
In Cibaliung underground gold mine, the ore is excavated from under to upper direction. The cut and fill method of Cibaliung consists of three stages for one access. After the first excavation complete, concrete of 30cm thickness is constructed in the floor. Rock bolt and shotcrete are also installed in the sidewalls and continue with backfilling with waste material. The concrete of 30cm is only applied for the first slice of stope excavation, whereas the other slice the concrete is not applied anymore. However, shotcrete of 5cm thickness is applied before the backfilled. Figure 4.2 shows the stages of stope excavation and the backfill material property is given in Table 4.1.
Table 4.1 The backfilled material properties

<table>
<thead>
<tr>
<th>$\sigma_c$ (MPa)</th>
<th>$E$ (GPa)</th>
<th>$\nu$</th>
<th>$c$ (MPa)</th>
<th>$\phi (\ldots^\circ)$</th>
<th>$\rho$ (gr/cm$^3$)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.2</td>
<td>153.1</td>
<td>0.321</td>
<td>0.2</td>
<td>20.5</td>
<td>1.4</td>
</tr>
</tbody>
</table>

Figure 4.2 Stages of ore excavation

4.2 EMPIRICAL METHOD FOR HANGING WALL ROADWAY SUPPORT DESIGN

Instability of underground mine opening is more likely caused by induced stresses. When an underground mine opening is excavated in the rock, the stress field is locally disrupted and a new set of stress is induced in the rock surrounding the openings (Hoek et al., 1993). Some researchers raised the topic of induced stress distribution. Kirsch (1898) is the pioneer of this research. He used analytical solutions for quantifying induced stresses distribution in a stressed elastic plate containing a
circular hole. Hoek et al. (1993) noted that researchers such as Love (1927) and Muskhelishvili (1953) published solutions for excavations of various shapes in elastic plates. Other analytical solutions to solve the problem of stress concentration around underground opening were published by Savin (1968), Hoek and Brown (1980), Barton, et al., (1974), and Grimstad and Barton (1993). The researches summarized solutions for induced stresses and its application in rock mechanics, and also proposed additional tools for predicting the induced stress distribution by using stress chart.

When the tangential stress conditions around roadway opening and compressive strength of the rock are known, it is possible to predict rock damage and determine the support system. The purpose of this study is to design Cibaliung mine support requirements in roadway of cut and fill mining method based on hybrid numerical-empirical method.

Barton et al. (1974) proposed the Norwegian Geotechnical Institute (NGI) tunneling Q-index to investigate the rock mass, and this index was updated by Grimstad and Barton (1993). The Q-system was originally developed for rock masses classification with the aim of being a helpful tool for evaluating the requirement for support in underground tunnels. The Q-system is described in a chart namely the support estimation chart. The support estimation chart is constructed based on more than 1,000 cases of rock support performed in tunnels and caverns (Palmstrom and Broch, 2006). The support estimation chart (Figure 4.3) mainly consists of three parameters i.e. rock mass quality (Q-system), equivalent dimension ($D_e$), and the prediction result of support information including length and space of bolt, and shotcrete requirement.
Equivalent dimension \( (D_e) \) is an additional parameter on the NGI system when predict the support for an underground excavation. The \( D_e \) is influenced by dimension of underground opening that a function of span or height of excavation and Excavation Support Ratio (ESR). In application of support requirement prediction, the equivalent dimension \( (D_e) \) is plotted to support estimation chart together with the \( Q \)-value (Figure 4.15).

\[
D_e = \frac{\text{Span or height of excavation (m)}}{\text{ESR}} \quad (4.1)
\]

The ESR reflects construction practice which considers the balance of safety and support with an excavation. As permanent mine opening, the ESR value of Cibaliung roadway is 1.6 which in line with Barton et al. (1974) suggestion.

The superiority of the NGI empirical method compared with the other empirical method is the involvement of Stress Reduction Factors (SRF) parameter which
represents the effect of stress condition to the rock strength around underground openings. The detailed explanation of the NGI Q-index classification is given in this section. There are six parameters to calculate the NGI Q-index according to the following formula:

\[ Q = \frac{RQD}{J_n} \times J_r \times J_a \times J_w \times SRF \]  
(4.2)

where: RQD = Rock Quality Designation  
\( J_n \) = Joint set number  
\( J_r \) = Joint roughness number  
\( J_a \) = Joint alteration number  
\( J_w \) = Joint water reduction factor  
SRF = Stress Reduction Factor

The following section explains the six parameters of tunneling Q-index and the use of NGI classification to estimate the support requirements.

1. Rock Quality Designation (RQD)

Rock Quality Designation (RQD) is determined from drill core that introduced by Deere (1963). The RQD is defined as the percentage between the total lengths of all core pieces is more than 10cm in length over the total length of the core recovered. When no core is available but discontinuity traces are visible in surface exposures or exploration, Palmstrom (1982) suggested that the RQD is estimated from the number of discontinuities per unit volume. Parameter values of RQD can be seen in Appendix 1.

2. Joint set number \( (J_n) \)

\( J_n \) is the number of joint set in the rock mass. The values vary from 0.5 for a massive rock mass with no or few joints to 20 for a crushed or disaggregated rock. Parameter values for Joint set number can be seen in Appendix 2.

3. Joint roughness number \( (J_r) \)
$J_r$ represents the roughness of the structural features in the rock mass. The values vary from 0.5 for a slicken side and planar surfaces to 5 for non-persistent structures with spacing larger than 3m. Appendix 3 gives estimated values of Joint roughness number.

4. Joint alteration number ($J_a$)

$J_a$ represents the condition of the degree of alteration of the structures in the rock mass. The values vary from 0.75 for wall-wall contact in unaltered rock or for joints containing tightly healed, hard, non-softening and impermeable filling to 20 for structures with thick fillings of clay gouge. Appendix 4 presents estimated values of joint alteration number based on joint in-fill conditions.

5. Joint water reduction factor ($J_w$)

$J_w$ accounts for the destabilizing effect of high water pressures. The values range from 1.0 for dry excavations to 0.05 for excavations with excessive inflow and pressure. Estimated values of Joint water reduction factor are presented in Appendix 5.

6. Stress Reduction Factor (SRF)

SRF is a relation between stress and rock strength of the rock mass around an underground opening. The SRF can be determined for four different stress situations as shown in Appendix 6. Grimstad and Barton (1993) improved the SRF rating in the Q-system. The updated Q-system has changed the maximum SRF value from 20 to 400 (Table 4.2). In case of the rock mass is heavily jointed and under high stresses, a squeezing effect is more likely to occur than spalling, and Table 4.2c should be used instead of 4.5b (NGI, 2013).

Based on Eq. (4.2), the rock mass quality Q can be considered by function of only three parameters:

1. Block size \(\left(\frac{RQD}{J_n}\right)\)
2. Inter-block shear strength \(\left(\frac{J_r}{J_a}\right)\)
3. Active stress \(\left(\frac{J_w}{SRF}\right)\)

The block size and inter-block shear strength can be conducted by field investigation; however the active stress that depended on SRF is a complicated empirical factor.
Based on Appendix 6 that updated by Table 4.2, the SRF can be described from the function of maximum tangential stress ($\sigma_\theta$) and uniaxial compressive strength ($\sigma_c$). For complex opening, the tangential stress as explained before can be obtained from numerical analysis.

The application of Q-system develops over time. Some researchers and engineers have been applying Q-system for underground mine. In 2007, Peck and Lee investigated the correlation between installed ground supports with Q-system. The investigation was done for 59 selected sites for 15 mine sites. They summarized that there is a reasonable correlation between actual ground performance and $Q$ values. Based on this background, the Q-system is applied in this study to assess the stability of openings and select ground support requirements.

One of the advantages of Q-system compared to RMR is the Q-system considered the stress distribution (SRF) for predicting the failure and support requirement as a parameter. It is difficult to obtain the induced stress directly from the field therefore the SRF in the Q-system becomes redundant (Hutchinson and Diederics, 1996). The SRF can be set to 1.0 and in most cases, the joint water reduction factor ($J_w$) can be set to 1.0 especially where the excavation is dry. Hence, the Q-system by setting the SRF and $J_w$ factor to 1.0 is called modified Tunneling Quality index, $Q'$. Application of the $Q'$ is proposed by Mathews et al. (1980) and followed by Potvin (1988) as Stability Number and Graph for open stope dimensioning tool for many mining operations.

### 4.2.1 Tangential Stress

As shown in Figure 4.4, the influence of induced stress from the stope for in-situ stress ratio equals 1:1 and 1:2 are not clearly seen in tangential stress of roadway at roof since there is a big difference of the in-situ stress' influence and induced stresses from stope's influence. Tangential stress direction of roadway at roof is relative horizontal. While, the induced stress from stope at roadway roof is vertical. Thus, they are perpendicular each other. The induced stress of stope weakens the tangential stress at the roof. Subsequently, its influence is decreasing when the distance of stope and roadway increases. The tangential stress at roof is higher than that of the sidewall due to the horizontal in-situ stress was increasing the tangential stress.
Table 4.2 Classification of SRF (after Barton et al., 1974 and Grimstad and Barton, 1993)

<table>
<thead>
<tr>
<th>Classification</th>
<th>$\sigma_c/\sigma_1$</th>
<th>$\sigma_{\theta}/\sigma_c$</th>
<th>SRF</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>b. Competent rock, rock stress problems</strong></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>H. Low stress, near surface</td>
<td>$&gt;200$</td>
<td>$&lt;0.01$</td>
<td>2.5</td>
</tr>
<tr>
<td>J. Medium stress</td>
<td>$200-10$</td>
<td>$0.01-0.3$</td>
<td>1</td>
</tr>
<tr>
<td>K. High stress, very tight structure (Usually favourable to stability, may be unfavourable to wall stability)</td>
<td>$10-5$</td>
<td>$0.3-0.4$</td>
<td>0.5-2</td>
</tr>
<tr>
<td>L. Moderate slabbing after $&gt;1$ hour in massive rock</td>
<td>$5-3$</td>
<td>$0.5-0.65$</td>
<td>5-50</td>
</tr>
<tr>
<td>M. Slabbing and rockburst after minutes in massive rock</td>
<td>$3-2$</td>
<td>$0.65-1.0$</td>
<td>50-200</td>
</tr>
<tr>
<td>N. Heavy rockburst (strain burst) and immediate dynamic deformation in massive rock</td>
<td>&lt;2</td>
<td>$&gt;1.0$</td>
<td>200-400</td>
</tr>
<tr>
<td><strong>c. Squeezing rock: plastic flow of incompetent rock under the influence of high rock pressure</strong></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>O. Mild squeezing rock pressure</td>
<td>$1-5$</td>
<td></td>
<td>5-10</td>
</tr>
<tr>
<td>P. Heavy squeezing rock pressure</td>
<td>$&gt;5$</td>
<td></td>
<td>10-20</td>
</tr>
<tr>
<td><strong>d. Swelling rock: chemical swelling activity depending on presence of water</strong></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>R. Mild swelling rock pressure</td>
<td></td>
<td></td>
<td>5-10</td>
</tr>
<tr>
<td>S. Heavy swelling rock pressure</td>
<td></td>
<td></td>
<td>10-15</td>
</tr>
</tbody>
</table>

The influence of induced stresses from stope becomes clearer when the in-situ stress weakens the tangential stresses in the roof side. Such a condition happens in the in-situ stress ratio equals 1:0.5 and 1:0.33 when the vertical stress is stronger so that the tangential stress at roof, which is relatively in horizontal direction, is weaken. It can be clearly seen from one of those charts on Figure 4.4 that the induced stresses of stope is influencing the induced stresses of roadway roof and the influence becomes weaker over the distance between stope and roadways. It is found that the influence of induced stress of stope can be seen when the distance between roadway and stope is less than 20m.

The tangential stress in near-stope and far-stope sidewall tends to decrease with the increasing distance of stope to roadway. The effect of induced stresses from
the stopes to the tangential stresses in near-stope and far-stope side becomes clearer with decreasing the distance of opening to stope obviously when the distance between roadway and stope is less than 20m.

The influence of in-situ stress regime on tangential stress also can be seen in Figure 4.4. It is seen that the tangential stress at the roof increases with the lateral stress increasing. It is happened because the direction of horizontal in-situ stress parallels to the tangential stress’ direction. Contrarily occurs when the vertical in-situ stress which is perpendicular to the tangential stress’ direction increases as shown in Figure 4.4 (a). These charts indicate that the amount of tangential stresses at roof increase with increasing the lateral in-situ stress. The influence of in-situ regime on tangential stress at sidewalls can be seen in the Figure 4.4 (b) and 4.4 (c). It is seen that tangential stresses at the sidewalls decrease with increasing the lateral in-situ stress.

As mentioned before that tangential stress influences the stability of roadway. It means the tangential stress also might influence the support system requirement. Comparison tangential stress at every location surrounding opening also can be analyzed by above procedures of simulations. For example, when in-situ stress regime of lateral stress is higher than that of vertical stress, for example equals 1:2, tangential stress that acts at the roof will be highest than tangential stress that acts at other location. Therefore, at these kind conditions more concern should be given at the roof side. Meanwhile, for the in-situ stress ratio equals 1:0.5 and 1:0.33, the tangential stress at near-stope and far-stope sidewall is higher than that of roof then more concern should be given at the sidewall.

4.2.2 Support System Requirement in Roadway of Underground Gold Mine

In order to determine the roadway support requirement of Cibaliung, the condition of rock mass surrounding the roadway needs to be known. The summaries of RMR classification of hanging wall and footwall rock in Cibaliung Underground Mine is given in Chapter 2 Table 3.2 and Table 3.3, respectively. Meanwhile, Table 4.3 shows the summary of basic Q-system (Q value without SRF parameter, Q') for the same rock mass condition that evaluated by using the RMR rock mass classification.

As mentioned in Chapter 1 that the in-situ stress ratio between vertical stress and horizontal stress of Cibaliung is similar 1 : 2, the chart of normalized tangential
stress for in-situ stress ratio $\sigma_v : \sigma_h = 1 : 2$ is taken from Figure 4.4 (a). Since the application is carried out for roadway that is located 40m from stope, the maximum tangential stress in the roof of roadway which is $3.14\sigma_h$. By applying the horizontal stress that is known in Chapter 1 ($\sigma_h = 9.2$ MPa), the maximum tangential stress in hanging wall roadway is equal to 28.90 MPa.

When the maximum tangential stress in hanging wall roadway is equal to 28.90 MPa and the uniaxial compressive strength of intact rock sample ($\sigma_c$) is equal to 24 MPa, thus the ratio of $\sigma^\theta/\sigma_c$ is equal to 1.20. Table 4.2 predicts that mild squeezing might be occurred in the hanging wall roadway and the table suggests a SRF value of 10.

The Q' value that Q-value without $J_w$ and SRF for weak rock mass in hanging wall roadway from Table 3.2 (RMR 30) is 2.04 (see Table 4.6). For the SRF value of 10, the Q value of the rock mass would be 0.204 that is categorized as very poor quality of rock mass. The Q value was used to determine the support system requirement. However, two additional parameters should be determined i.e. Excavation Support Ratio (ESR) and equivalent dimension ($D_e$). As permanent opening, we should consider for the safety of the roadway. Because of roadway is a main access, the ESR value was taken 1.6 as suggested by the Barton et al. (1974). Since the height of roadways is 4.8 m, by Eq. 4.1, the equivalent dimension ($D_e$) is calculated and gave a value of $D_e$ is equal to 3.0. The Q value of 0.204 and $D_e$ value of 3.0 were plotted to support estimation chart.
Figure 4.4 Chart of normalized tangential stresses around the hanging wall roadway for different stress regime. (a) at the roof, (b) at the sidewall near stope and (c) at the sidewall far stope.
Table 4.3 Rock mass classification in Cibaliung underground mine based on index Q

<table>
<thead>
<tr>
<th>Location</th>
<th>Qualitative Condition</th>
<th>RQD</th>
<th>Jn</th>
<th>Jr</th>
<th>Ja</th>
<th>Jw</th>
<th>Q’</th>
</tr>
</thead>
<tbody>
<tr>
<td>Hanging wall</td>
<td>worst</td>
<td>55</td>
<td>9</td>
<td>2</td>
<td>6</td>
<td>1</td>
<td>2.04</td>
</tr>
<tr>
<td></td>
<td>best</td>
<td>55</td>
<td>9</td>
<td>3</td>
<td>3</td>
<td>1</td>
<td>6.11</td>
</tr>
<tr>
<td>Footwall</td>
<td>worst</td>
<td>60</td>
<td>9</td>
<td>3</td>
<td>3</td>
<td>0.66</td>
<td>6.67</td>
</tr>
<tr>
<td></td>
<td>best</td>
<td>65</td>
<td>6</td>
<td>3</td>
<td>3</td>
<td>1</td>
<td>10.83</td>
</tr>
</tbody>
</table>

The support requirements for hanging wall roadway are 9 pieces of rock bolts 1.8m in length with space of 1.4m and fiber reinforced shotcrete 12 cm. Support requirement at the floor of roadway is supported by the standard of Cibaliung underground gold mine with shotcrete 20cm thickness. Figure 4.5 shows the result of numerical analysis of support requirement by using Phase² software. Stability of roadway after supported increase compared with the roadway without support, as seen on Figure 3.8. The yield zone at the roof is about 1.4m, at the sidewall far stope about 1.1m and at the sidewall near stope about 1.3m. Yield zone at the floor of hanging wall roadway about 3.4m. Displacements also decrease, at the roof displacement about 0.8-1.6cm, at the sidewall near stope about 1.9-2.9cm, at the sidewall far stope about 1.8-2.9cm and at the floor about 1.1-3.6cm. Even though the failure decrease compared with Pongkor supporting system (see Figure 4.6), liner failure still occurred at the roof, floor and bottom part of sidewalls. Moreover, tension and shear failure also occurred at rock bolt support. Therefore, hanging wall roadway of Cibaliung underground mine needs another support design which consider with failure zone around the roadway.
Figure 4. 5 Failure and displacement of hanging wall roadway of Cibaliung underground gold mine supported by shotcrete 12cm thickness, rock bolt 1.8 m length, 1.4m space

Figure 4. 6 Comparison failure of hanging wall roadway of Cibaliung underground gold mine between supported by Pongkor system (left) and Q-system (right)
4.3 NUMERICAL METHOD FOR HANGING WALL ROADWAY SUPPORT DESIGN

In Chapter 2, support design of Cibaliung underground gold mine was analyzed based on Pongkor underground gold mine standard support system. The support requirement consists of shotcrete of 10cm thickness or H-beam space of 0.6m. In previous Section 4.2, support design of Cibaliung underground gold mine based on rock empirical method was studied and estimated shotcrete 12cm thickness, however some failure still occurred. Yield zone surrounding roadway generally is more than 1m, tension and shear failure of rock bolts also occurred. Moreover, liner failures at the roof and bottom part of sidewall were found from the modelling.

In most mining operations, the underground support design is usually based on previous experience and evolves over a number of years. As mentioned in Section 4.2 that induced stress gives influence for excavation stability and consequences to the support system requirements.

In this section, another method for the requirement of support system is studied. The support system is analyzed based on numerical modeling. Induced stress concentration surrounding roadway opening is used to consider the rock bolt density. The aim of this study is to design more appropriate support requirements and increase the roadway stability.

4.3.1 Tangential Stress Concentration

In this study, around the roadway opening is divided into six zones to simplify the location of stress concentration as given in Figure 4.7. The result for tangential stress concentration is described in this section. The ratio of vertical in-situ stress and horizontal in-situ stress is 1:2 which mean the horizontal in-situ stress is higher than the vertical in-situ stress. In this research, the simulation consists of five stages. First stage is initial condition that the roadway and the stope are not opened. In the stage 2 the only roadway is opened. Stope is excavated from under to upper direction at the stage 3, stage 4 and stage 5. Tangential stress concentration in different distances between stope and roadway can be seen in Figure 4.8.
Figure 4. 7 Concentration of tangential stress zonation

Influence of stope on tangential stress can be seen from this result. At the first slice of stope excavation, high tangential stress occurs at the roof because the influence of lateral in-situ is higher than vertical stress. Tangential stress changes when the second slice of stope is excavated. Tangential stresses at the roof and floor decrease whereas tangential stresses at the sidewalls increase. Similar conditions are occurred when the third slice of stope is excavated. In this stage stress concentration at the sidewall near stope is high as well as at the roof near stope. Even stress concentration at the floor of roadway is low, stress concentration at the corners of floor are the highest. These conditions evidence that tangential stresses surrounding roadway influenced by stope activities. Direction of tangential stress at the roof and floor are relatively horizontal, whereas perpendicular with induced stress from stope. Therefore, when the stope is excavated, the induced stresses from stope reduce tangential stresses at the roof and floor. Whereas, direction of tangential stresses at the sidewalls is vertical and also parallel with direction of induced stress from stope. Therefore, the induced stresses from stope increase tangential stress at the sidewalls.

Based on the results at the final slice stope excavations, high stress concentrations are occurred at the roof (zone B and C) and at the sidewall near stope (zone D), and at the corner of floor. This condition should be considered to design support system due to failures risk.
Figure 4. Stress distributions surrounding hanging wall roadway. Distance between stope and roadway is 5m (a), 20m (b), and 40m (c)
Stress concentration at the roadway increase when the distance between stope and roadway decrease. When the distance is 20m, stress concentration is reflected by the change of tangential stress. High changes of tangential stresses are occurred at the roof, roof near stope, sidewall near stope and at the floor (zone ‘B’, ‘C’, ‘D’ and ‘F’), whereas at the sidewall far stope (zone ‘F’) the change of tangential stress relatively low. The condition becomes worst when the distance is 5m. Tangential stress of roadway is desultory that indicates the roadway is highly influenced by induced stress of stope. Moreover, based on this condition, high failure zone is occurred at roadway when the distance between stope and roadway is 5m. Based on stress concentration that occurs, prediction of support system at roadway is estimated.

4.3.2 Prediction of Support System at Hanging Wall Roadway

According to numerical analyses, stress concentration is influenced by stope excavations. Influence of stope excavations on stress concentrations in different distance of stope and roadway are summaries in Table 4.4. It is found that stage of stope activities also influence the stress concentration. In final stope excavation, high stress concentrations are occurred at the roof and sidewall near stope (zone “B, C and D”) whereas at the sidewall far stope and floor (zone of “E and F”) relatively similar and low, however at the corner of floor the tangential stress is the highest compare other locations. These conditions become a reason to design the space of bolts. The rock bolts are designed denser for high stress concentration than that of low stress concentration.

At the zone of “B, C and D”, the space of bolts designed to 0.5m, however at zone of “A and F” the space of bolts designed as 1.0m. Whereas, at the floor of roadway design of support system will explains particularly in the next section.
In addition to design the length of bolt, yield zone around opening is considered. Based on Subchapter 3.3.1 hanging wall roadway without support system, the yield zone at the roof is about 2.7m, at the floor more than 5m, at the bottom part of sidewall about 1.5m and 1.4m at near stope and far stope, respectively. Based on the yield zone, cable bolt of 4.0m in length is needed to support at the roof because of the length of rock bolt cannot reach the maximum yield zone. At the sidewall rock bolt 1.8m length are possible to apply at the upper part however at the bottom part of sidewall proposed to use 2.4m length of rock bolt. In this chapter, three types of liners are analyzed; those are 12cm thick shotcrete (as seen in Subchapter 4.2.2), 15cm thick shotcrete and H-beam. Property of cable bolt is given in Table 4.5.

Table 4.5 Cable bolt material properties

<table>
<thead>
<tr>
<th>Bore hole Diameter (mm)</th>
<th>Cable diameter (mm)</th>
<th>Cable modulus (MPa)</th>
<th>Cable strength (MN)</th>
<th>Angle of internal friction of grout (deg.)</th>
</tr>
</thead>
<tbody>
<tr>
<td>48</td>
<td>19</td>
<td>200,000</td>
<td>0.1</td>
<td>25</td>
</tr>
</tbody>
</table>
The result of roadway supported by shotcrete 12cm thickness is given in Figure 4.9. The yield zone at the roof decrease from 2.7m before supported to be about 1.0m after supported. At the bottom part of sidewall near stope the yield zone is about 1.3m and at the sidewall far stope the yield zone about 1.1m. Tension failure of bolts occurs at the roof and bottom part of sidewall. Liner failure also occurs at the roof, bottom part of sidewall and at the floor of the roadway. Displacements decrease to be 1.5cm at the roof, 2.1cm, 2.7cm and 3.6cm at the sidewall near stope, sidewall far stope and at the floor, respectively.

Figure 4.9 Failure and displacement of hanging wall roadway supported by 12cm thick shotcrete with various bolt lengths
Figure 4.10 shows the result of the roadway supported by shotcrete 15cm thickness. The yield zone at the roof is about 0.9m, at the sidewall near stope is about 1.3m and at the sidewall far stope is about 0.8m. The yield zone decreases compared the roadway supported by 12cm thick shotcrete however the bolts and liners failure still occur at the roof, bottom part of sidewall and floor of the roadway. Displacements around the roadway observed below. At the roof displacement is about 1.3cm, whereas 1.9cm, 2.4cm and 3.0cm at the sidewall near stope, sidewall far stope and at the floor, respectively.

Figure 4.10 Failure and displacement of hanging wall roadway supported by 15cm thick shotcrete with various bolt lengths

The result of roadway that supported by H-beam is given in Figure 4.11. Yield zone at the crown of roof decreases significantly. At the sidewall yield zone also decrease that about 0.6m at the sidewall near stope and about 0.5m at the sidewall far stope. Displacement at the roof is about 0.8cm, whereas at the sidewall near stope,
sidewall far stope and at the floor are 1.5cm, 1.7cm and 2.3cm, respectively. The bolt failures occur, a few and only as shear failure. Moreover, the liner failure is only occurred at the floor of the roadway that supported by shotcrete.

Figure 4.11 Failure and displacement of hanging wall roadway supported by H-beam with various bolt lengths

To install the rock bolts as proposed above are not simple because the different lengths of rock bolt needs different equipment. Hence, another support requirement is proposed and compared with above results. The method is similar with analysis above however the length of split set is changed to be uniform as 2.4m. Figure 4.12 to Figure 4.14 show the results of these supports.

According to Figure 4.12 the roadway was supported by shotcrete 12cm thickness and rock bolts 2.4m length, yield zone at the roof is about 1.0m and
maximum displacement is about 1.5cm. The yield zone at the sidewall near the stope is about 1.3m with displacement about 2.1cm, whereas at the sidewall far the stope the yield zone is about 1.2m and displacement about 2.7cm. The results are relatively similar with the previous supported by various bolts lengths.

Figure 4.12 Failure and displacement of hanging wall roadway supported by 12cm thick shotcrete, with rock bolt 2.4m in length

Figure 4.13 shows the roadway supported by shotcrete 15cm thickness. The yield zone at the roof is about 0.9m with maximum displacement about 1.3cm. The yield zone at both of sidewalls is about 1.0m with maximum displacements are 2.0cm and 2.3cm at sidewalls near stope and far stope, respectively.
Figure 4.13 Failure and displacement of hanging wall roadway supported by 15cm thick shotcrete, with rock bolt 2.4m in length

When the roadway is supported by H-beam (see Figure 4.14), the yield zone at the roof not occurs with displacement is about 0.8cm. At the sidewall near stope the yield zone is about 0.5m and at the far stope is about 0.6m. Maximum displacement of the sidewall near stope and far stope are about 1.5cm and 1.6cm, respectively.
The result of roadway supported by various rock bolt lengths is given in Table 4.6, and the result of the roadway supported by rock bolt 2.4m is given in Table 4.7. It is shown that influence of length of cable bolt to stability on hanging wall roadway in Cibaliung underground mine is not significant. Weak rock properties of hanging wall is caused the cable bolt is not effective. Therefore, the stability of hanging wall roadway in Cibaliung underground mine is more influenced by liner types or shotcrete thickness.
Table 4.6 Roadway of Cibaliung supported by various bolts lengths of 1.8m and 2.4m, and cable bolt 4m in length)

<table>
<thead>
<tr>
<th>Yieldzone:</th>
<th>No support</th>
<th>Shotcrete 12cm</th>
<th>Shotcrete 15cm</th>
<th>H-beam</th>
</tr>
</thead>
<tbody>
<tr>
<td>- Roof</td>
<td>2.7m</td>
<td>1.0m</td>
<td>0.9m</td>
<td>-</td>
</tr>
<tr>
<td>- Sidewall-near stope</td>
<td>1.5m</td>
<td>1.3m</td>
<td>1.3m</td>
<td>0.6m</td>
</tr>
<tr>
<td>- Sidewall-far stope</td>
<td>1.4m</td>
<td>1.1m</td>
<td>0.8m</td>
<td>0.5m</td>
</tr>
<tr>
<td>- Floor</td>
<td>&gt;5m</td>
<td>3.6m</td>
<td>3.6m</td>
<td>2.7m</td>
</tr>
<tr>
<td>Displacement:</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>- Roof</td>
<td>5.0cm</td>
<td>0.7-1.5cm</td>
<td>0.4-1.3cm</td>
<td>0.2-0.8cm</td>
</tr>
<tr>
<td>- Sidewall-near stope</td>
<td>3.2cm</td>
<td>1.7-2.1cm</td>
<td>1.3-1.9cm</td>
<td>0.8-1.5cm</td>
</tr>
<tr>
<td>- Sidewall-far stope</td>
<td>3.3cm</td>
<td>1.8-2.7cm</td>
<td>1.6-2.4cm</td>
<td>1.2-1.7cm</td>
</tr>
<tr>
<td>- Floor</td>
<td>5.8cm</td>
<td>1.1-3.6cm</td>
<td>0.8-3.0cm</td>
<td>0.6-2.3cm</td>
</tr>
<tr>
<td>Bolt failure</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>Tension;roof Tension;roof</td>
<td>Shear;</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>Tension-shear;  Tension-shear;</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>sidewall     sidewall</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Liner failure</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>Roof and floor</td>
<td>Roof and floor</td>
<td>Floor</td>
<td></td>
</tr>
</tbody>
</table>

Table 4.7 Roadway of Cibaliung supported by uniform rock bolt length 2.4m

<table>
<thead>
<tr>
<th>Yieldzone:</th>
<th>Shotcrete 12cm</th>
<th>Shotcrete 15cm</th>
<th>H-beam</th>
</tr>
</thead>
<tbody>
<tr>
<td>- Roof</td>
<td>1.0m</td>
<td>0.9m</td>
<td>-</td>
</tr>
<tr>
<td>- Sidewall-near stope</td>
<td>1.3m</td>
<td>1.0m</td>
<td>0.5m</td>
</tr>
<tr>
<td>- Sidewall-far stope</td>
<td>1.2m</td>
<td>1.0m</td>
<td>0.6m</td>
</tr>
<tr>
<td>- Floor</td>
<td>3.6m</td>
<td>3.6m</td>
<td>2.7m</td>
</tr>
<tr>
<td>Displacement:</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>- Roof</td>
<td>0.5-1.5cm</td>
<td>0.4-1.3cm</td>
<td>0.2-0.8cm</td>
</tr>
<tr>
<td>- Sidewall-near stope</td>
<td>1.6-2.1cm</td>
<td>1.6-2.0cm</td>
<td>1.2-1.5cm</td>
</tr>
<tr>
<td>- Sidewall-far stope</td>
<td>1.8-2.7cm</td>
<td>1.6-2.3cm</td>
<td>1.2-1.6cm</td>
</tr>
<tr>
<td>- Floor</td>
<td>1.0-3.5cm</td>
<td>0.8-2.9cm</td>
<td>0.6-2.1cm</td>
</tr>
<tr>
<td>Bolt failure</td>
<td>Tension-shear; Tension-shear;</td>
<td>Shear;</td>
<td></td>
</tr>
<tr>
<td></td>
<td>roof-sidewall  roof-sidewall</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Liner failure</td>
<td>Roof, sidewall, floor</td>
<td>Roof, sidewall, floor</td>
<td>floor</td>
</tr>
</tbody>
</table>
Tangential stress concentration around roadway is influenced by high lateral stress and induced stress of stope activities. Design of rock bolt density for support system in this research is estimated based on the highest stress concentration. High stress concentration occurs at the roof, sidewall near stope and at the corner of floor whereas at the sidewall far stope stress concentration is low. In this research for high stress concentration rock density is designed as spacing 0.5m and for low stress concentration the rock density is designed as spacing 1.0m. Since stress concentration is influenced by stope activities, installation process of density of rock bolt for roadway underground mine can be applied into two schemes. First, installation of rock bolt can be done immediately after the roadway is excavated which considered to the highest stress concentration in the final stope excavations. And the second, rock bolts are installed in line with stope activities which considered to the highest stress concentration in every slice excavation as following the result in Table 4.4.

Stope activities also influence failures surrounding roadway. Failure zone increases with stope excavation activities thus at the final excavation the highest failure zone is occurred. Length of cable bolt that designed based on the failure zone gives similar result with the uniform rock bolt of 2.4m in length. Weak rock of hanging wall and there is not strong rock around the roadway initiating cable bolt lose the strength. Therefore, uniform rock bolt length is recommended for support system at the roadway.

Distance between stope and roadway also influence the roadway stability. More close the roadway to stope more stress concentration occurs between both openings, and failures increase especially at the sidewall near stope. Therefore, when the distance between stope and roadway decreases, supplementary rock bolts are needed to improve at the sidewall near stope. Figure 4.15 shows influence of distance between stope and roadway on the roadway failure zone. Performance of support system still capable to apply when the roadway is located more than 20m from stope. Supplement support system is required when the distance of roadway is 10m from stope especially at the sidewall near stope. Influence of induced stress from stope increases and new failure could be occurred at the roadway. When distance between stope and roadway is 5m, overlapping failure zone occurs between stope and roadway. Therefore, it is not recommended to develop roadway at the distance of 5m from the stope.
Based on analysis of liner support system, H-beam gives better performance and after roadway is installed failure zone decrease. However, high failure zone at the floor still occurs. Therefore, in the next section support system at the floor is studied.

Figure 4.1 Influence of distance between stope and roadway on the roadway stability. Roadway supported by shotcrete 15cm (upper), and roadway supported by H-beam (under)

Since the support requirement influences mining cost, the price of support requirement should be estimated. Price of rock bolt with length of 2.4m is about 24USD/piece. In case that spacing of rock bolt is similar in row and column, the numbers of rock bolts become 57 pieces in every 1.2m length of peripheral roadway and the cost of rock bolt is 1,368USD estimated. The cost of shotcrete with 15cm thickness is about 133USD/m². The shotcrete cost of 1.2m length of peripheral roadway with area of 15.6m² is 2,080 USD. In a same area, when the Cibaliung underground mine applies H-beam, the cost of H-beam is 1,000 USD/set for length of 13m. It is predicted to apply at least 3 sets of H-beam in area of peripheral roadway of 15.6m² and the cost is 3,000 USD, approximately. Combination of support between shotcrete 15cm thickness and rock bolt 2.4m in length needs about 3,448 USD, while the cost of combination of H-beam and rock bolt 2.4m in length is 4,368 USD. In economic point of view, combination of shotcrete with 15cm thickness and rock bolt
2.4m in length is cheaper compared to combination of H-beam spacing 0.6m and rock bolt 2.4m in length.

4.4 FLOOR SUPPORT DESIGN

Floor stability plays an important role in the safety and operation of underground mines, both in maintaining the access for equipment transportation and in providing a foundation for the roof/pillar/floor system. Based on previous section simulation, failure evidence at the floor as well as at the roof because of the horizontal in-situ stress is higher than vertical stress. Stability at the roof condition is significantly changes after support applied however at the floor support requirement is still needed. Based on the shotcrete 20cm thickness that supported at the floor, wide yield zone still found hence other support requirement is needed.

![Figure 4.16 Shotcrete thickness for floor support](image)

To improve the floor stability, support system is designed by different shotcrete thickness as 30cm and 40cm. The results can be seen in Figure 4.16. High failures still occur at the floor due to high lateral in-situ stress. When the floor is supported by thick material such as shotcrete, the height of roadway will decrease and does not suitable for mine’s equipment. Grouting method and digging the hoist foundation pit can be designed for the floor support. In this study the floor excavation is designed by digging method as 20cm, 45cm and 75cm depth and filled the pit by concrete. The concrete properties simulated can be seen in Table 2.10. Sidewall and roof support system is designed based on the result from previous chapter that consists of rock bolt with
length 2.4m, space 0.5m at the roof and sidewall near stope and space 1.0m at the sidewall far stope. The liner supports consist of shotcrete 15cm thickness and H-beam.

Figure 4.17 shows the result of floor designed by 75cm digging and filled by concrete, and the roadway supported by shotcrete 15cm thickness. At the roof of roadway, yield zone about 1.0m with tension and shear failure of rock bolt occurred. Liner failure also observed at the roof. Yield zone at the floor after filled the pit by concrete does not occur directly at the floor of roadway. The yield zone occurs under the concrete support about 3.2m. Moreover, liner failure also does not occur at the floor. Based on Figure 4.18 that floor of roadway is designed by 75cm digging and filled by concrete, and other wall sides supported by H-beam, condition of floor is relatively similar with above supported. No liner failure occurred at the floor, and yield zone only occur under the concrete about 3.2m. At the roof of roadway relatively stable however yield zone about 0.8m occur at the sidewalls. Moreover, by this design no tension failure of rock bolt occurs.

Figure 4. 17 Floor excavation 75cm and filled by concrete, roadway supported by 15cm thick shotcrete
Another design is proposed for floor stability. The floor ground is designed by 45 cm digging and filled by concrete. The results can be seen in Figure 4.19 and Figure 4.20. Based on Figure 4.19 that the roadway supported by shotcrete 15 cm thickness, there is no yield zone occurred directly under the roadway. The yield zone occurred under the concrete filled. Other condition of surrounding roadway is relatively similar with 75 cm digging design. Yield zone at the roof is about 0.9 m, whereas at sidewalls are about 0.5 m. Rock bolts failure and line failure especially occurred at the roof of the roadway. The result of floor designed by 45 cm digging and filled by concrete, and roadway supported by H-beam can be seen in Figure 4.20. It shows that the result of floor is no yield zone occurred under the roadway but occurred under the concrete. Yield zone at the roof of roadway is stable, whereas at the sidewalls the yield zone is about 0.8 m and 0.7 m at near stope and far stope, respectively. Table 4.8 shows the result summaries of floor design.

In other case, based on numerical analysis result, when the floor is designed by 20 cm digging and filled by concrete, failure zone occurs at the floor of roadway. The result can be seen in Figure 4.21 and Figure 4.22. The displacement at the floor is about 1.6 cm to 4.1 cm. Based on the results to improve the stability at the floor of roadway can be designed by digging the floor at least 45 cm depth and filled by concrete.
Figure 4. 19 Floor excavation 45cm and filled by concrete, roadway supported by 15cm thick shotcrete

Figure 4. 20 Floor excavation 45m and filled by concrete, roadway supported by H-beam
Figure 4. 21 Floor excavation 20cm and filled by concrete, roadway supported by H-beam

Figure 4. 22 Floor excavation 20cm and filled by concrete, roadway supported by shotcrete 15cm thickness
<table>
<thead>
<tr>
<th></th>
<th>Shotcrete 15cm</th>
<th>H-beam</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Floor excavation 75cm</td>
<td>Floor excavation 45cm</td>
</tr>
<tr>
<td><strong>Yieldzone:</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td>- Roof</td>
<td>1.0m</td>
<td>0.9m</td>
</tr>
<tr>
<td>- Sidewall-near stope</td>
<td>0.5m</td>
<td>0.6m</td>
</tr>
<tr>
<td>- Sidewall-far stope</td>
<td>0.5m</td>
<td>0.5m</td>
</tr>
<tr>
<td>- Floor</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td><strong>Displacement:</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td>- Roof</td>
<td>0.4-1.4cm</td>
<td>0.4-1.4cm</td>
</tr>
<tr>
<td>- Sidewall-near stope</td>
<td>1.8-2.1cm</td>
<td>1.7-2.0cm</td>
</tr>
<tr>
<td>- Sidewall-far stope</td>
<td>1.7-2.2cm</td>
<td>1.7-2.2cm</td>
</tr>
<tr>
<td>- Floor</td>
<td>1.8-3.7cm</td>
<td>1.8-3.4cm</td>
</tr>
<tr>
<td><strong>Bolt failure</strong></td>
<td>tension and shear at the roof - sidewall</td>
<td>tension and shear at the roof - sidewall</td>
</tr>
<tr>
<td><strong>Liner failure</strong></td>
<td>Roof</td>
<td>Roof</td>
</tr>
</tbody>
</table>
4.5 CONCLUSION

In order to modify support system of Pongkor underground mine to apply in Cibaliung underground mine, empirical and numerical analysis are applied to find out appropriate support system.

Empirical method that considers induced stress is used to find support requirement for improving roadway of Cibaliung underground mine. Induced tangential stress is then combined with rock mass classification to estimate the support requirement. The support system requirements based on Q-support chart consist of fibre reinforced shotcrete 12 cm thickness and rock bolts 1.8m length with spaces of 1.4m. However, failures are still occurred because of high lateral in-situ stress of Cibaliung and the support system of Pongkor is not suitable to apply. Therefore, another method is needed to improve the roadway stability.

Based on numerical analysis, design of rock bolt density for support system is estimated based on the highest stress concentration. High stress concentration occurs at the roof, sidewall near stope and at the corner of floor whereas at the sidewall far stope stress concentration is low. In this research for high stress concentration rock bolt density is designed as spacing 0.5m and for low stress concentration the rock density is designed as spacing 1.0m. Length of cable bolt is designed based on the failure zone. However, in this research influence of length of bolt to reduce failure zone does not significant since roadway is located at hanging wall side consists of weak rock and the support is more influenced by liner type. Therefore, uniform cable bolt of 2.4m in length is applied at the roadway of Cibaliung underground mine. Based on numerical analysis H-beam gives better influence to reduce the failure zone.

Performance of support system of roadway is still capable to apply when the roadway is located at 20m from stope. However supplement support system is required when the distance of roadway is 10m from stope especially at the sidewall near stope. When distance between stope and roadway is 5m, overlapping failure zone occurs between stope and roadway. Therefore, it is not recommended to develop roadway at the distance of 5m from the stope.

High failure also occurs at the floor of roadway. To decrease the failures at the floor of roadway can be designed by digging the floor at least 45cm and filled by concrete.
In this chapter, support system of roadway where located at hanging wall side has been studied. However, since roadway is developed at the hanging wall, another opening like cross cut that connected the roadway to ore body should be developed at the same side. Under similar condition with roadway that developed at high lateral in-situ stress and weak rock, cross cut has high similar risk to failures. Therefore, in the next chapter support system at the cross cut needs to study.
CHAPTER 5

OPTIMAL CROSS CUT SUPPORT DESIGN OF CUT AND FILL MINING
METHOD AT CIBALIUNG UNDERGROUND MINE

5.1 INTRODUCTION

Cross cut is an important access that connected main roadway and the ore body in a cut and fill mining method. As mentioned in Chapter 2 that the Cibaliung underground mine develop roadway not only in footwall but also in hanging wall side. The problems associated with hanging wall openings at Cibaliung underground mine is the hanging wall consists of weak rock and consists of high lateral in-situ stress that sometimes induces opening collapse if it is not properly treated. In Cibaliung underground mine because the roadway is also constructed at hanging wall side, the cross cut access is also build in this side. Therefore, high risk of failure not only occurs at the roadway opening but also potential to happen at the cross cut access. Thus, in order to prevent failure on the openings of Cibaliung underground mine, study of support system at Cibaliung underground mine should be done not only for the roadway as discussed at Chapter 4, but also for cross cut access.

In this study, the optimal cross cut support design in a variety of ground conditions and mining conditions in Cibaliung gold mine is analyzed. In order to examine the support design, stability analysis is carried out by using a three dimensional finite difference method analysis program FLAC3D. FLAC3D contains an automatic 3D grid generator in which grids are created by manipulating and connecting predefined shapes. It incorporates the facility to model cross cut and stope that cannot modeled by Phase² software as two dimensional programs.

The model is designed as geological condition and cut and fill mining method that applied in Cibaliung underground gold mine. Some initial models are analyzed consist of: (1) influence of cross cut position at footwall and hanging wall; (2) influence of rock mass condition based on rock mass properties; (3) and influence of depth of mining stope. Figures 5.1 to 5.3 show initial model of cross cut design in Cibaliung underground gold mine by using FLAC3D.
Figure 5. 1 Initial model of cross cut in Cibaliung mine

Figure 5. 2 Model design of cross cut
The standard cross cut support system of Pongkor underground mine consists of 15cm thickness of shotcrete and 9 pieces of 2.4m length of rock bolts (see Figure 5.4). The properties of shotcrete and rock bolt are given in Table 2.4 and Table 2.5, respectively. The density of rock bolt is installed symmetrical because both of the sidewalls receive similar influence from stope. This is a little bit different with rock bolt installation at roadway which the density at sidewall near stope (less than 20m from the stope) is higher than that of at sidewall far stope (more than 20m from the stope) because the sidewall near stope besides experience the lateral in-situ stress also receives induced stress from stope. In the cross cut access, because the location is near to stope, the induced axial stress is dominant than that of induced transverse stress. Thus, the use of H-beam would not give significant difference with shotcrete. Due to the area near stope usually consists of many fracture zones, the Pongkor underground mine has chosen shotcrete instead of H-beam as their support system at cross cut access to diminish rock fall. In economic point of view and installation, the shotcrete is also better than that of H-beam. This Pongkor underground mine's support system at cross cut access is taken as initial condition in installation of support system at cross cut access of Cibaliung underground mine.

Figure 5.5 shows the results of cross cut condition after the stope is excavated for slice 1 to slice 3. It is shown that the failure zone which illustrated with green color progressively increases at the roof of cross cut access near roadway area (roadway intersection) to near stope area (stope intersection). Moreover, it is also found that the
failure zone at roof of cross cut near stope will increase when the slice move to upper level. This increasing failure zone is influenced by high induced stress around the stope.

Based on the numerical simulation, the size of failure zone is about 18m long times 10m height from stope intersection (see Figure 5.5). With this condition, it is almost impossible to retain the roof stability of cross cut access near stope of Cibaliung underground mine by adopting Pongkor underground mine’s support system at cross cut access which has length of rock bolt only 2.4m as shown in Figure 5.6. Therefore, this Pongkor underground mine’s support system should be modified to achieve safe condition at cross cut access near stope of Cibaliung underground mine. Based on the failures area, application of cable bolt can be proposed.

In this study, the modification of support system has been carried out by considering: (1) influence of lateral stress coefficient; (2) position of cross cut access in regards to ore; (3) influence of depth of cross cut access.

Figure 5. 4 Initial support design
Figure 5.5 Cross cut failure in hanging wall positions when first, second and final stope was excavated.
5.2 INFLUENCE OF LATERAL STRESS COEFFICIENT

Mining method and support system of Cibaliung is adopted from Pongkor mine. However, the in-situ stress regime between both mines indicated different. Hence, study of influence in-situ stress regime is needed. In order to understand the impact of lateral stress, study with ratio of lateral to vertical in-situ stress ($k = 1$) is numerically analyzed. The results of this numerical analysis are given in Figure 5.7 and Figure 5.8.

Based on Figure 5.7 the failure zone around the face of stope is smaller when compared with failure zone at the same place for $k=2$ (see Figure 5.5). It means the influence of stope activities in ratio of lateral and vertical in-situ stress of $k'=1$ is not as significant as $k'=2$. It is clearly shown by Figure 5.8. This figure shows the failure zone at cross section “a-a” as attached at Figure 5.7. Compared with failure zone for $k'=2$ (see Figure 5.6), failure zone of $k'=1$ (see Figure 5.8) is smaller. The failure zone for $k'=1$ is about 2m in height from the roof. Based on this failure zone, the available Pongkor underground mine’s support system is still applicable to retain the roof from failure.
Figure 5.7 Cross cut failure in ratio lateral and vertical stress (‘k’) is 1 when first, second and final stope was excavated.
It is also seen in Figure 5.7 that the failure zone at roof of cross cut near stope (less than 20 m from the stope) increases when the slice move to upper level. However, failure zone at roof of cross cut far from stope (more than 20 m from the stope) is not strongly influenced by moving slice to upper level. This phenomena is in line with when ‘k’=2. It indicates that increasing failure zone at the roof of cross cut near stope is not significantly influenced by lateral in-situ stress but strongly influenced by induced stress.

Figure 5.8 Cross section “a’-a” view of cross cut with ‘k’=1 when final stope was excavated

5.3 FOOTWALL CROSS CUT FAILURES CONDITION

Cibaliung underground mine also develops openings in footwall side. In this section, failures condition at the footwall is studied. The length of cross cut is set to 40m as well as the hanging wall side and the lateral stress coefficient (k) is 2. Standard support system of footwall cross cut is similar with the support system of footwall roadway that consists of 5cm shotcrete and 1.8m length of rock bolt that installed 1m in space. The result of numerical analysis can be seen in Figure 5.9 and Figure 5.10.

Figure 5.9 shows that the stability of cross cut access is not influence by the stope. It is seen that the failure zone occurs only surrounding the stope. The failure zone increases with increasing of stope slice. The phenomenon of uninfluenced cross cut by stope is clearly shown by Figure 5.10. This figure shows the cross section “a’-a” which attached at Figure 5.9. This cross section’s figure shows the failure zone floating 4m over the roof of cross cut. Thus, it could be stated that the Pongkor underground
mine’s support system is suitable applied in footwall cross cut of Cibaliung underground mine.

Figure 5. 9 Cross cut failure at footwall when first, second and final stope was excavated
5.4 INFLUENCE OF DEPTH ON CROSS CUT STABILITY

When the stope excavation is developed and the depth increases, the influence of depth on cross cut stability needs to be analyzed in order to prevent the cross cut from failure. The analysis has been done by means of numerical analysis. Recently, Cibaliung underground mine has been excavating the ore at 100m depth. Therefore, all of numerical analysis which has been discussed in previous chapter is done for 100m depth. Considering the future mine plan of Cibaliung underground mine, the mining company plans to continue their activities in 150m depth. Therefore, this numerical analysis is done at 100m depth. Moreover, it has been discussed before that the most potential failure at cross cut occurred at the hanging wall side. Thus in this study, the numerical analysis has been done for cross cut access in the hanging wall. The results of numerical analysis are given in Figure 5.11 to Figure 5.13.

Figure 5.11 shows that failure zone near stope (less than 20 m from the stope) grows following the mining stages. However, the failure zone far from stope (more than
20 m from the stope) is few influenced by the stope. It is in good agreement with the previous results that has been discussed in subchapter 5.1 and subchapter 5.2. However, it looks like the failure zone at cross cut near stope of deeper cross cut access is wider than that of shallower cross cut access. It is seen in Figure 5.11 the failure zone is about 21m wide times 11m height from intersection between stope and cross cut. Whereas as mentioned in subchapter 5.1 which the depth of cross cut is 100m, the failure zone is about 18m wide times 10m height. The wider failure zone is more clearly in Figure 5.12. This figure shows the cross section “a’-a” that attached in Figure 5.11. The failure zone that showen at Figure 5.12 (cross cut location is 150m depth from the surface) is wider compared with the failure zone that showen at Figure 5.5 (cross cut location is 100m depth from the surface). It indicates that the overburden pressure also plays important role on growing the failure zone at cross cut near stope.

Considering the wider failure zone at deeper cross cut, the support system should be stronger. Thus, the Pongkor underground mine’s support system should be modified if the mine wants to keep the stability of cross cut when the cross cut moving to deeper area.
Figure 5. 11 Cross cut failure when depth is 150m at the first, second and final stope was excavated
5.5 SUPPORT DESIGN OF CROSS CUT MINE ACCESS

As explained in the previous section, the failure occurred at the cross cut in the hanging wall that consists of weak rock strength with high lateral in-situ stress. The failure zone occurred at the roof of cross cut because the influence of induced stress from the stope. According to previous discussion subchapter 5.1 to subchapter 5.4 the most failure zone at cross cut access is the cross cut where situated in the intersection between stope and cross cut to 20m distance from the intersection. For distance of cross cut more than 20m distance from the intersection, the available support system already suitable to prevent the cross cut failure. Thus in this study, the modification of support system is done at the cross cut where situated up to 20m distance from the intersection. Considering the sidewalls and floor of cross cut condition which already stable, the study is focused only on the roof. In this study, the modification is done for: (1) cable bolt length; (2) cable bolt density and; (3) combination of cable bolt and safety pillar.
The numerical simulation is done for Cibaliung underground mine conditions such as the lateral in-situ stress ratio equals 2, the cross cut located 100m in depth and the cross cut is located in hanging wall side.

5.5.1 Cross Cut Supported by Different Length of Cable Bolt

In this study, three different lengths of cable bolts at the roof such as 8m, 12m and 16m are numerically analyzed. The cable bolt is installed in accordance with the standard rock bolt which has spacing of cable bolt 1.5m distance. Different with rock bolt installation in roadway, the installation of cable bolt in this study is started from the middle of failure zone. From the middle of failure zone, the cable bolt is installed to the right side and left side. Based on the width of failure zone and cable bolt spacing, the number of bolt at the roof should be 5 pieces. Designs of cross cut are given in Figure 5.13.

![Figure 5.13 Length of cable bolt design of cross cut support system](image-url)
The results of numerical analysis for 8m, 12m and 16m in length of cable bolt are given in Figure 5.14. Based on these results, it is found that the cable bolt does not give strong influence on failure zone reduction. The influence of cable bolt length on failure zone reduction can be seen in Figure 5.15. Figure 5.15 illustrates the failure zone for different cable bolt length compared with standard support system at the roof of cross cut. The standard support system at the roof of cross cut as discussed in subchapter 5.1 consists of five pieces of 2.4m in length of rock bolt and spaced of 1.5m. The cable bolt will not effectively work when the anchored rock is weak. It is in line with the rock properties of this study.

Figure 5.14 Cross cut failure supported by different lengths of cable bolt (a) 8m, (b) 12m, and (c) 16m
Considering the length of cable bolt does not give significant improvement on cross cut stability, it is necessary to study about density of cable bolt with different length.

### 5.5.2 Cross Cut Supported by Different Cable Bolt Density and Length

Improvement of cross cut stability by different length of cable bolt has been numerically analyzed. The numerical result shows that in case of this study area, the length of cable bolt does not give significant improvement on cross cut stability. In this subchapter, the cable bolt density role on cross cut stability improvement is studied. In this study, three different lengths of cable bolt that used in subchapter 5.5.1 (8m, 12m and 16m in length) are installed in spacing of 1.5m, 1.0m and 0.5m. Similar with cable bolt installation at subchapter 5.5.1, the installation of cable bolt in this study is also conducted from the middle of roof. The spacing of 1.5m has been numerically analyzed at subchapter 5.5.1. Thus in this study, the numerical analysis is carried out only for spacing of 1.0m and 0.5m. Based on the failure zone as well as spacing of cable bolt, the number of cable bolt at the roof of cross cut should be 7 pieces and 9 pieces for spacing equal to 1.0m and 0.5m respectively. The cable bolt design of this study is descripted in Figure 5.16. This figure illustrates the axial section and transverse section.
Figure 5.16 Cable bolt design in various space

Figure 5.17 to Figure 5.19 show the failures of cross cut when it supported by different length of cable bolts. It is seen at Figure 5.17 to Figure 5.19, the failure zone slightly decreases with increasing the density of cable bolt for all length of cable bolt. This phenomenon is clearly described at Figure 5.20. The reduction of failure zone is found up to 25% by extending the length of cable bolt up to 16m and reducing the spacing up to 0.5m. However, the reduction of failure zone mostly occurred at cross cut where located around 11m to 20m distance from the stope (see failure zone in yellow box). Decreasing of spacing of 16m in length of cable bolt up to 0.5m does not effective to reduce the failure zone at cross cut where located less than 11m from stope. The ineffective of this method is more caused by high induced stress adjacent the stope than weak rock condition. Based on this result, it could be stated that the combination between high density and long cable bolt can solve the stability in weak rock condition but cannot solve the stability at weak rock location that has strong induced stress.

Considering the combination of length and density of cable bolt ineffective reduce the failure zone at location consisted of combination of weak rock and high
induced stress, it is necessary to find out a method which can overcome high induced stress and weak rock at the same time.

Figure 5.17 Cross cut supported by cable bolt with length 8m. (a) space 1.5m, (b) space 1.0m, and (c) space 0.5m
Figure 5. 18 Cross cut supported by cable bolt with length 12m. (a) space 1.5m, (b) space 1.0m, and (c) space 0.5m
Figure 5. 19 Cross cut supported by cable bolt with length 16m. (a) space 1.5m, (b) space 1.0m, and (c) space 0.5m
5.5.3 Cross Cut Supported by Cable Bolt through Ore Vein

Improvement of cross cut stability by combination of length and density of cable bolt has been numerically analyzed. The numerical result shows that the combination of 16m of length and 0.5m of spacing of cable bolt successfully reduce the failure zone up to 25% at weak rock condition. However, this method failed to overcome high induced stress effect on weak rock condition.

In order to solve this problem, cross cut supported by cable bolt through ore vein is studied. This study is carried out based on the best combination of length and density of cable bolt that has been discussed at subchapter 5.5.2. Based on Figure 5.20, the two effective combinations to reduce the failure zone are 16m in length of cable bolt that spaced of 0.5m and 16m in length of cable bolt that spaced of 1.0m.

In this study, to get strong anchored cable bolt, the cable bolt from roof of cross cut is attached to ore vein which has strong rock mass. This kind of ore vein is called safety pillar or sill pillar. The size of safety pillar is same with the size of stope i.e. 5m in height. Considering the cable bolt that attached to the safety pillar has maximum spacing of 1.0m, the maximum of cable bolts capacities are 4 pieces. In the cross cut, the cable bolt that connects to safety pillar should be designed attach at un-failure zone. According to subchapter 5.5.2, the limitation of failure zone at the cross cut is 11m distance from stope (see Figure 5.19). Based on the limitation of failure zone, the cable...
bolt which attached to the safety pillar is installed move away from the distance of 12m from the stope. The scheme of installation of cable bolt that attached between safety pillar and un-failure zone of cross cut is given at Figure 5.21.

Based on numerical analysis result that can be seen in Figure 5.22 for cable bolt spacing is 1.0m and Figure 5.23 for cable bolt spacing is 0.5m, it is found that this method can reduce the failure zone up to 40% and 30% respectively. This method can only decrease the size of failure zone from 18m to 9m in length and from 10m to 6m in height. This remaining failure zone is located below the cable bolt that was attached between safety pillar and non-failure zone of cross cut. This happens because the failure zone is not only affected by the high lateral stress condition, but also influenced by induced stress from the stope. From this result, it can be concluded that another method is needed to increase the stability of area around cross cut and stope. One way to do that is by increasing the stability of the stope itself. Therefore, in the next chapter, to reduce the failure zone in the intersection area of cross cut and stope, a new approach considering the stope condition is proposed.

Figure 5. 21 Scheme of cable bolt design that attached at the roof of cross cut and safety pillar (red line), and cable bolt that installed at roof of cross cut (black line)
Figure 5. 22 Cross cut stability in cable bolt through the vein. The cable bolt space is 1.0m

Figure 5. 23 Cross cut stability in cable bolt through the vein. The cable bolt space is 0.5m


5.6 CONCLUSION

Three designs of cable bolt have been simulated in order to improve cross cut stability. Design with different lengths of cable bolt to reduce failure zone at the roof of cross cut not significant. Improvement of cross cut stability by combination of length and density of cable bolt has been numerically analyzed. The numerical result shows that the combination of 16m of length and 0.5m of spacing of cable bolt successfully reduce the failure zone up to 25% at weak rock condition. However, this method failed to overcome high induced stress effect on weak rock condition.

In order to solve this problem, cross cut supported by cable bolt through ore vein is studied. This study is carried out based on the best combination of length and density of cable bolt. In this study, to get strong anchored cable bolt, the cable bolt from roof of cross cut is attached to ore vein which has strong rock mass. This kind of ore vein is called safety pillar or sill pillar. In the cross cut, the cable bolt that connects to safety pillar should be designed attached at un-failure zone. According to subchapter 5.5.2, the region of failure zone at the cross cut is 11m distance from stope. Based on the region of failure zone, the cable bolt which attached to the safety pillar is installed move away from the distance of 12m from the stope. Based on numerical analysis it is
found that this method can reduce the failure zone up to 40% and 30% for cable bolt’s spacing is 0.5m and 1.0m respectively. This method can decrease the size of failure zone up to 9m in long times 6m in height from intersection of stope and cross cut. This failure zone is located below the cable bolt that attached between safety pillar and un-failure zone of cross cut. It indicates that this method can overcome the induced stress that might generate the failure zone. Due to limitation of this method, unsolved failure zone adjacent stope can solely be solved from inside of the stope. Therefore, in the chapter 6, the support system at the stope needs to be studied.
CHAPTER 6

STOPE SUPPORT DESIGN OF CUT AND FILL MINING METHOD

Stope is the most important part of mining activities in regard to mine production. However as discussed in Chapter 5, some failure zone adjacent to stope should be treated from inside of stope. In other word, to retain stope from failure a support system should be installed in the stope. The unstable stope opening is shown in Figure 6.1. This Figure shows the failure zone adjacent stope before support system is installed. This figure is in good agreement with one of the conclusions of Chapter 5.

6.1 STOPE EXCAVATION STABILITY AND SUPPORT DESIGN

Pongkor underground mine applies support system at stope consists of shotcrete 5cm thickness with two pieces of 1.8m in length of rock bolt that spaced of 1.5m. This support system is used as initial support systems at stope of Cibaliung underground mine. When this support system was applied, the failure zone is still found adjacent of stope especially at hanging wall side and roof as shown in Figure 6.2. The failure zone at hanging wall and at the roof side is caused by dilution and confining stress and gravity. It is well known that a jointed of rock mass is significantly controlled by the confinement, e.g, a weak hanging wall or roof of stope will simply failure if the confinement is removed by process often referred to as unravelling or slabbing. Therefore, the Pongkor underground mine’s support system should be modified to retain the rock failure in the stope of Cibaliung underground mine.

The modification of this support system is mainly carried out based on guidance that developed for support system at roadway in hanging wall as has been discussed at Chapter 4. The difference of construction of support system at stope compared with roadway is the support system at stope is constructed separately based of the failure zone and stress concentration condition at each part of stope boundary. Meanwhile, because the properties of the rock adjacent to the roadway is same, the support system at the roadway has been constructed based on the failure zone and stress concentration condition at all boundary of the roadway. The stress concentration at boundary of stope for all slices is given in Figure 6.3.
Based on Figure 6.2 about the failure zone and Figure 6.3 about stress concentration, there is no correlation between failure zone and stress concentration because the rock type had sidewalls and roof as well as floor is different. The failure zone at hanging wall side is largest although the stress concentration at this side is lowest. In this study, the modification of support system is done based on failure zone instead of stress concentration.

Figure 6.1 Failure zone around stope

Figure 6.2 Initial conditions of stope stability, rock bolt length 1.8m, space 1.5m and shotcrete 5cm
Based on the failure zone adjacent to the stope, the length of rock bolt at the hanging wall side and roof of stope is modified to be 2.4m. Meanwhile the length of rock bolt at footwall side does not change as 1.8m. In order to reduce the failure zone surrounding the stope and get the stable state, the density of rock bolt is increased gradually from 2 pieces for each side to 4 pieces as shown in Figure 6.2, Figure 6.4 and Figure 6.5 respectively.

Figure 6.3 Stress concentration at the stope
Based on the numerical analysis, condition at the footwall side and at the roof of stope is stable after design with 3 pieces of rock bolts. However, failure zone still occurs especially at the hanging wall side. Thus, the density of rock bolt at hanging wall side is increased twice to achieve the stable condition (see Figure 6.6). From this discussion, it could be concluded the unsolved failure zone by cable bolt that attached between safety pillar or sill pillar and cross cut can be solved by increasing density of rock bolt.
6.2 SILL PILLAR THICKNESS ANALYSIS

As discussed at Chapter 5, sill pillar plays important role on the reduction of failure zone at the hanging wall near to stope. Sill pillar for Cibaliung underground mine also plays important role on mining method when the Cibaliung underground mine continue to excavate the ore at deeper location. This mining method technically required sill pillar to keep the stope stable.

Sill pillar is left as one stope is mined out toward an upper backfilled stope (Sjoberg, 1993). Figure 6.7 shows the sill pillar in stope of cut and fill mining. In some cases, the pillar can be low grade ore, not profitable to mine. However, pillar is used as support element, since geomechanical conditions seldom permit complete extraction of an orebody. Consequently, pillar design is important in optimizing mining operations. In term of revenue, improved pillar design can be measured. Oversized pillar represent potential ore losses, and undersized pillar may fail and lead to serious problem and loss of production from stope. However, since the quality of the ore to be mined in this mine is low, it may be mentioned that is economical to leave the sill pillar to keep stope stable rather than to excavate and filled it.
Considering the support system requirement at cross cut access and stope, study about stability of sill pillar is discussed in this chapter. In order to study the sill pillar, RQD of ore vein of Cibaliung underground mine is investigated. The RQD is found in the range of 40% to 75%. In engineering and safety point of view, the RQD which concerned in this study equals 40%.

In this study, the sill pillar thickness is evaluated trial and error. In the trial and error method, the sill pillar thickness is numerically analyzed gradually without sill pillar to 10m of sill pillar thickness. Thus in this study, the working stope is located under filled stope. The support method that installed at this numerical analysis is the available support system that consists of 2.4m in length of rock bolt. The number of rock bolt that installed at the roof of stope is 3 pieces as shown at Figure 6.6. The numerical analysis result of sill pillar under 100m depth is given at Figure 6.8.
According to Figure 6.8, if the mining process is conducted without sill pillar or even to 2.5m thick sill pillar, the failure will occur. This is because of the anchored rock is filled material which has low confining stress. On the other hand, when the 5m, 7.5m or 10m thick sill pillar is used, sill pillar is stable. Their displacement is almost similar as viewed at Figure 6.9.
In engineering point of view, 5m in thickness of sill pillar can be categorized safe condition when the sill pillar is located at 100m depth. However considering the safety that the failure zone is half of sill pillar thickness i.e 2.5m, it is better to consider thicker sill pillar e.g., 7.5m and 10m in thickness. However in economic point of view, 7.5m in thickness of sill pillar is better than 10m in thickness of sill pillar.

6.3 SUPPORT DESIGN OF SILL PILLAR

Considering failure zone and displacement surrounding sill pillar that shown at Figure 6.8 and Figure 6.9, the best recommendation thickness of sill pillar is 7.5m. The advantage of 7.5m in thickness of sill pillar compared with 5m in thickness of sill pillar is there is more space that can be used as anchored rock for stability purposes e.g., extent the cable bolt up to 7.5m.

In order to find out the best design of sill pillar, cable bolt with end anchored is compared with split set 2.4m in length. Moreover, some scenario is made in this study e.g., (1) extent the length of cable bolt; (2) increase the length and density of cable bolt; (3) inclined cable bolt installation. The study is simulated for the sill pillar at 100m depth.
6.3.1 Extent the Length of Cable Bolt

In this study, the influence of extended length of cable bolt that installed at roof of stope is numerically analyzed. The cable bolt is attached to recommended 7.5m in thickness of sill pillar. The cable bolt that installed has length of 2.5m, 5.0m and 7.5m. The cable bolts is installed at the roof of stope with spacing of 1.2m. It means, the optimum number of cable bolt that can be installed is 3 in regards to the width of stope is 5m. The results can be seen in Figure 6.10. It is seen at Figure 6.10 that installing cable bolt with 5m length does not give significant difference on failure zone reduction compared to installing cable bolt with 2.4m length. Although the failure zone of installed of 2.4m in length of cable bolt almost similar with installed of 5m in length of cable bolt, the shape of failure zone is different. The failure zone of installed of 5m in length of cable bolt is more inclined toward the hanging wall than that of 2.5m. It is because the installed of 5m in length of rock bolt does not effectively work since their rock bolt crossing into the hanging wall which consists of weak rock. Thus, the rock near to hanging wall is more fractured than that of near to footwall. The installed of 7.5m in length of cable bolt also shows similar shape with the installed of 5m in length of cable bolt because some parts of their rock bolt is also crossing into the hanging wall (weak rock). However, the installed of 7.5m in length of cable bolt success to reduce the failure zone. Based on this result, it could be concluded that the bonding of cable bolt in anchored rock is important to ensure that all of the components are in contact.
6.3.2 Increase Length and Density of Cable Bolt

In this study, the influence of increasing density of cable bolt that installed at roof of stope is numerically analyzed. Similar with the study that has been done at subchapter 6.3.1 about the influence of extending the length of cable bolt; this study is done at recommended 7.5m in thickness of sill pillar. In order to study the influence of increasing density of cable bolt, the number of cable bolt that installed at the roof of roof varies from 3, 5, 7 and 9 pieces. The length of cable bolt that used in this study is 7.5m.

The results of numerical analysis of the influence of density of cable bolt can be seen in Figure 6.11. It is seen at Figure 6.11 that the failure zone slightly decreases with increasing the density of cable bolt. However, the stress is still concentrated at the hanging wall and the shape of failure zone is inclined toward hanging wall. It indicates that although the increasing density can reduce the failure zone, but cannot reduce the failure zone located near to hanging wall due to bonding of cable bolt to hanging wall is not strong enough (low confinement stress).
6.3.3 Influence of Inclination to Cable Bolt Installation

In this study, four different scenarios of inclination are numerically simulated as 45°, 90°, 110° and 135° from horizontal plane. Based on numerical result, slight influence of inclination to reduce failures can be seen in Figure 6.12. When the cable bolt is installed at 110° inclination which means it can be anchored at the strong rock of ore vein, failures at the sill pillar is decreased than that other inclination. Moreover, the displacement when 110° inclination is also smaller than that displacement from other inclination (see Figure 6.13). This result in line with the concept that used to improve cross cut at subchapter 5.5.3.
Figure 6.12 Different inclination of cable bolt installation

Figure 6.13 Influence of inclination of rock bolt installation on displacement
6.4 SILL PILLAR UNDER DEEPER MINING ACTIVITIES

In addition, since the ore resources are predicted to occur at 150m depth, for future situation sill pillar thickness is also evaluated when the extraction level reaches 150m depth, and the result is given in Figure 6.14. Failure zone also increases when the depth of excavation is increasing. The failures occur at the top and bottom of sill pillar because high vertical stress that is increased by depth factor. Consequently, thicker sill pillar is needed. Based on the numerical analysis, the stope condition is in high risk to collapse when the stope is designed without sill pillar. Overlapping failure zone occurs when the thickness of sill pillar is less than 5m which means that the sill pillar unstable. High failures also occur at the top and bottom of sill pillar when the thickness is 7.5m. When the thickness of sill pillar is 10m, condition is more stable compared to sill pillar with 7.5m thickness. However, support requirement is still needed.

From above results of support design, when the sill pillar is at 150m depth, the support requirements are analyzed by numerical analysis for 10m thickness of sill pillar. The cable bolt is designed as 110° from horizontal with two different lengths, 7.5m and 10m, and the result can be seen in Figure 6.15.

Based on the numerical results, to reduce the failure at sill pillar, even though the cable bolt length is increased due to the increase of sill pillar thickness, failure still occurs because of the effect of vertical stress that increases with depth. Therefore, not only changing the thickness of sill pillar and the pattern of construction of the rock bolt, but also by other method, for example installation of standing support such as rigid arch and cribs or increasing the strength of backfilling materials are required in order to maintain of the stope when the extraction level becomes deeper. These analyses are important and can be done for the future research.
Figure 6.14 Sill pillar thickness at 150m depth (a) No sill pillar thickness; (b) 2.5m; (c) 5.0m; (d) 7.5m and (e) 10m

Figure 6.15 Sill pillar with 10m thickness at 150m depth supported by different bolts. (a) standard support system, (b) cable bolt with 7.5m in length, and (c) cable bolt with 10m in length
6.5 CONCLUSION

Stope is the most important part of mining activities in regard to mine production. However as discussed in Chapter 5, some failure zone adjacent to stope should be treated from inside of stope. In other word, to retain stope from failure a support system should be installed in the stope.

In the stope area, the position where high failure zone occurs is in hanging wall side. Since the surrounding stope consists of different rock type with different properties, the stress concentration around the stope is also different in each part. Consequently, it is difficult to apply stress concentration condition in determining the rock bolt support system at the stope. Therefore, the failure zone around stope opening is used to design the density and length of support system. At the footwall side and roof of stope where the rock condition is good, design of length and density of rock bolt is lower than that at hanging wall side.

Consequently, for Cibaliung to excavate ore at deeper levels, sill pillar is needed to increase the stability of stope. In engineering point of view, sill pillar with thickness of 5m can be categorized as safe condition. However, considering that failure occurs in half lower part of sill pillar, it is better to use thicker sill pillar e.g., 7.5m and 10m in thickness. However in economic point of view, 7.5m in thickness of sill pillar is better than 10m in thickness of sill pillar. Application of cable bolt to reduce failure zone gives a better result when the cable bolt is anchored to the ore vein. However, when the mining activities become deeper, other design of support requirement is needed and become important for the future research.
CHAPTER 7

CONCLUSIONS

The study was carried out in Cibaliung Gold mine which located at protected forest and categorized as marginal ore deposit. Cibaliung developed roadway not only at the footwall side, but also at the hanging wall side. At the current, support system in Cibaliung gold mine adopted support system from Pongkor gold mine.

Roadway excavation in cut and fill underground mining plays an important role for the life of mines. The instability of roadway disrupts the productivities of mines. Pongkor is one of the oldest mine that still operating in Indonesia, and the support system standard of Pongkor has been established very well and becomes guidance for other underground mines in Indonesia. Pongkor underground gold mine roadway is located in footwall side.

As one of establish underground mines, Pongkor underground mine become a guidance to other underground mines which has strong similarity in ore type sulphidation like Cibaliung underground gold mine. However, due to differences on ore grade and lateral in-situ stress ratio, the adoption of Pongkor underground mine method should be firstly evaluated before applied in Cibaliung underground mine.

Cibaliung underground mine develops roadway with horseshoe shape whereas Pongkor develops rectangular shape. Based on numerical analysis, the shape of horseshoe shape gives an advantage to reduce the failure zone especially at the roof better than the roadway as a rectangular shape.

Cibaliung underground mine has high lateral in-situ stress with ratio between lateral to horizontal in-situ stress ‘k’=2, whereas Pongkor underground mine has in-situ stress ratio relatively similar (‘k’=1). This different condition gives high influence to the performance of support system. Based on analysis, if the ratio of in-situ stress of Pongkor ‘k = 2’ as same as Cibaliung’s in-situ stress, high failures are occurred especially at the roof and floor, and the support system is not sufficient.

Cibaliung underground mine develops roadway at hanging wall side where the stress concentration is high whereas Pongkor develops roadway at footwall side.
Moreover, condition of hanging wall of Cibaliung underground mine consists of weak rock. Based on the result, it is found that roadway of Cibaliung underground mine at footwall side is stable when the support system of Pongkor underground mine is applied. Meanwhile, high failures are occurred at the hanging wall of roadway of Cibaliung. Based on the results, Pongkor underground mine’s support system should be modified if we want to apply this support system on Cibaliung underground mine.

In order to find an appropriate model to this research, mesh analysis was studied in this chapter. The results show that six-node triangle element type analysis with outer boundary of 35 times of the opening was found close to the Kirsch equation. Moreover, density of node 0.15 m/node was recommended to optimize element size.

Empirical method that considers with induced stress is used to find support requirement for improving roadway of Cibaliung underground mine. Induced tangential stress is then combined with rock mass classification to estimate the support requirement. The support system requirements based on Q-support chart consist of fibre reinforced shotcrete 12 cm thickness and rock bolts 1.8m length with spaces of 1.4m. However, failures are still occurred because of high lateral in-situ stress of Cibaliung and the support system of Pongkor is not suitable to apply. Therefore, another method is needed to improve the roadway stability based on numerical analysis.

Based on numerical analysis, design of rock bolt density for support system is estimated based on the highest stress concentration. High stress concentration occurs at the roof, sidewall near stope and at the corner of floor whereas at the sidewall far stope stress concentration is low. In this research for high stress concentration rock density is designed as spacing 0.5m and for low stress concentration the rock density is designed as spacing 1.0m. Length of cable bolt is designed based on the failure zone. However, in this research influence of length of bolt to reduce failure zone does not significant since roadway is located at hanging wall side consists of weak rock and the support is more influenced by liner type. Therefore, uniform cable bolt of 2.4m in length is applied at the roadway of Cibaliung underground mine. Based on numerical analysis H-beam gives better influence to reduce the failure zone.

Performance of support system of roadway is still capable to apply when the roadway is located at 20m from stope. However supplement support system is required.
when the distance of roadway is 10m from stope especially at the sidewall near stope. When distance between stope and roadway is 5m, overlapping failure zone occurs between stope and roadway. Therefore, it is not recommended to develop roadway at the distance of 5m from the stope. However, high failure also occurs at the floor of roadway. To decrease the failures at the floor grouting method and digging the hoist foundation pit can be designed for the floor support with depth is 45cm and filled by concrete.

The problems associated with hanging wall openings at Cibaliung underground mine is the hanging wall consists of weak rock and consists of high lateral in-situ stress that sometimes induces opening collapse if is not properly treated. In Cibaliung underground mine because the roadway is also constructed at hanging wall side, the cross cut access is also build in this side. Therefore, high risk for failure does not only occur at the roadway opening but also potential to happen at the cross cut access. Based on the numerical simulation, the size of failure zone is about 18m long times 10m height from stope intersection. Therefore, this Pongkor underground mine’s support system should be modified to achieve safe condition at cross cut access near stope of Cibaliung underground mine. Based on the failures area, application of cable bolt can be proposed.

Three designs of cable bolt have been simulated in order to improve cross cut stability. Design with various lengths of cable bolt does not significantly reduce failure zone at the roof of cross cut. Improvement of cross cut stability by combination of length and density of cable bolt has been numerically analyzed. The numerical result shows that the combination of 16m of length and 0.5m of spacing of cable bolt successfully reduce the failure zone up to 25% at weak rock condition. However, this method failed to overcome high induced stress effect on weak rock condition.

In order to solve this problem, cross cut supported by cable bolt through ore vein design is studied. This study is carried out based on the best scenario of combination of length and density of cable bolt. In this study, to get strong anchored cable bolt, the cable bolt from roof of cross cut is attached to ore vein which has strong rock mass. This kind of ore vein is called safety pillar or sill pillar. In the cross cut, the cable bolt that connects to safety pillar should be designed attached at un-failure zone. According to subchapter 5.5.2, the area of failure zone at the cross cut is 11m distance from stope. Based on the area of failure zone, the cable bolt which attached to the
safety pillar is installed move away from the distance of 12m from the stope. Based on numerical analysis it is found that this method can reduce the failure zone up to 40% and 30% for cable bolt’s spacing is 0.5m and 1.0m respectively. However, it still leaves a failure zone below of the cable bolt that was attached between safety pillar and non-failure zone of cross cut. From this result, it can be concluded that another method is needed to increase the stability of area around cross cut and stope. Therefore, in Chapter 6, to reduce the failure zone in the intersection area of cross cut and stope, a new approach considering the stope condition is proposed.

High failure zone occurs in the hanging wall side of stope area. Considering the different rock type and properties, which results in different stress concentration condition around the stope, it is concluded that the design of rock bolt support system is based on the failure zone around stope opening. At the footwall side and roof of stope which are good condition, design of length and density of rock bolt is lower than that at hanging wall side.

Consequently, for Cibaliung to extract the ore at deeper levels, the stability of the stope needs to be considered. To increase the stability of the stope, sill pillar can be used. In this research, 5m thick sill pillar is considered safe in engineering point of view. However, considering that failure occurs in half lower part of sill pillar, thicker sill pillar is preferred. But, the thicker the sill pillar is, the less of ore that can be extracted from stope. Therefore, design of stope with 7.5m thick sill pillar is selected by considering engineering and economic point of view. However, when the mining activities become deeper, other design of support requirement is needed and becomes important for the future research.
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APPENDIX

Appendix 1 RQD classification

<table>
<thead>
<tr>
<th>DESCRIPTION</th>
<th>VALUE</th>
<th>NOTES</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>1. ROCK QUALITY DESIGNATION</strong></td>
<td><strong>RQD</strong></td>
<td></td>
</tr>
<tr>
<td>A. Very poor</td>
<td>0 – 25</td>
<td>1. Where RQD is reported or measured as ≤10 (including 0), a nominal value of 10 is used to evaluate Q.</td>
</tr>
<tr>
<td>B. Poor</td>
<td>25 - 50</td>
<td></td>
</tr>
<tr>
<td>C. Fair</td>
<td>50 - 75</td>
<td></td>
</tr>
<tr>
<td>D. Good</td>
<td>75 - 90</td>
<td>2. RQD intervals of 5, i.e. 100, 95, 90 etc. are sufficiently accurate.</td>
</tr>
<tr>
<td>E. E. Excellent</td>
<td>90 - 100</td>
<td></td>
</tr>
</tbody>
</table>

Appendix 2 Joint set number

<table>
<thead>
<tr>
<th>2. JOINT SET NUMBER</th>
<th>Jn</th>
<th>NOTES</th>
</tr>
</thead>
<tbody>
<tr>
<td>A. Massive, no or few joints</td>
<td>0.5 – 1.0</td>
<td>1. For intersection use (3.0 x Jn)</td>
</tr>
<tr>
<td>B. B. One joint set</td>
<td>2</td>
<td></td>
</tr>
<tr>
<td>C. C. One joint set plus random</td>
<td>3</td>
<td></td>
</tr>
<tr>
<td>D. D. Two joint sets</td>
<td>4</td>
<td></td>
</tr>
<tr>
<td>E. E. Two joint sets plus random</td>
<td>6</td>
<td></td>
</tr>
<tr>
<td>F. F. Three joint sets</td>
<td>9</td>
<td></td>
</tr>
<tr>
<td>G. G. Three joint sets plus random</td>
<td>12</td>
<td></td>
</tr>
<tr>
<td>H. H. Four or more joint sets, random, heavily jointed, 'sugar cube', etc.</td>
<td>15</td>
<td>2. For portals use (2.0 x Jn)</td>
</tr>
<tr>
<td>J. J. Crushed rock, earthlike</td>
<td>20</td>
<td></td>
</tr>
</tbody>
</table>
### Appendix 3 Joint roughness number

<table>
<thead>
<tr>
<th>3. JOINT ROUGHNESS NUMBER</th>
<th>Jr</th>
</tr>
</thead>
<tbody>
<tr>
<td>a. Rock wall contact</td>
<td></td>
</tr>
<tr>
<td>b. Rock wall contact before 10 cm shear</td>
<td></td>
</tr>
<tr>
<td>A. A. Discontinuous joints</td>
<td>4</td>
</tr>
<tr>
<td>B. B. Rough and irregular, undulating</td>
<td>3</td>
</tr>
<tr>
<td>C. C. Smooth undulating</td>
<td>2</td>
</tr>
<tr>
<td>D. D. Slickensided undulating</td>
<td>1.5</td>
</tr>
<tr>
<td>E. E. Rough or irregular, planar</td>
<td>1.5</td>
</tr>
<tr>
<td>F. F. Smooth, planar</td>
<td>1.0</td>
</tr>
<tr>
<td>G. G. Slickensided, planar</td>
<td>0.5</td>
</tr>
<tr>
<td>c. No rock wall contact when sheared</td>
<td></td>
</tr>
<tr>
<td>H. H. Zones containing clay minerals thick enough to prevent rock wall contact</td>
<td>1.0 (nominal)</td>
</tr>
<tr>
<td>J. J. Sandy, gravelly or crushed zone thick enough to prevent rock wall contact</td>
<td>1.0 (nominal)</td>
</tr>
</tbody>
</table>

1. Add 1.0 if the mean spacing of the relevant joint set is greater than 3 m

2. Jr = 0.5 can be used for planar, slickensided joints having lineations, provided that the lineations are oriented for minimum strength
Appendix 4 Joint alteration number

<table>
<thead>
<tr>
<th>4. JOINT ALTERATION NUMBER</th>
<th>$Ja$</th>
<th>$\phi_r$ degrees (approx.)</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>a. Rock wall contact</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td>A. Tightly healed, hard, non-softening, impermeable filling</td>
<td>0.75</td>
<td></td>
</tr>
<tr>
<td>B. Unaltered joint walls, surface staining only</td>
<td>1.0</td>
<td>25-35</td>
</tr>
<tr>
<td>C. Slightly altered joint walls, non-softening mineral coatings, sandy particles, clay-free disintegrated rock, etc.</td>
<td>2.0</td>
<td>25-30</td>
</tr>
<tr>
<td>D. Silty-, or sandy-clay coatings, small clay fraction (non-softening)</td>
<td>3.0</td>
<td>20-25</td>
</tr>
<tr>
<td>E. Softening or low-friction clay mineral coatings, e.e. kaolinite, mica. Also chlorite, talc, gypsum and graphite etc., and small quantities of swelling clays</td>
<td>4.0</td>
<td>8-16</td>
</tr>
<tr>
<td><strong>b. Rock wall contact before 10 cm shear</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td>F. Sandy particles, clay-free, disintegrating rock etc.</td>
<td>4.0</td>
<td>25 – 30</td>
</tr>
<tr>
<td>G. Strongly over-consolidated, non-softening clay mineral fillings (continuous &lt;5 mm thick)</td>
<td>6.0</td>
<td>16 – 24</td>
</tr>
<tr>
<td>H. Medium or low over-consolidation, softening clay mineral fillings (continuous &lt;5 mm thick)</td>
<td>8.0</td>
<td>12 – 16</td>
</tr>
<tr>
<td>J. Swelling clay fillings, i.e. montmorillonite (continuous &lt;5 mm thick). Values of $Ja$ depend on percent of swelling clay-size particles, and access to water</td>
<td>8.0</td>
<td>6 – 12</td>
</tr>
<tr>
<td><strong>c. No rock wall contact when sheared</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td>K, L, M. Zones or bands of disintegrated or crushed rock and clay (see G, H, J for description of clay condition)</td>
<td>6.0, 8.0 or 8.0 – 12.0</td>
<td>6 – 24</td>
</tr>
<tr>
<td>N. Zones or bands of silty- or sandy clay, small clay fraction (non-softening)</td>
<td>5.0</td>
<td></td>
</tr>
<tr>
<td>O, P, R. Thick, continuous zones or bands of clay (see G, H, J for description of clay condition)</td>
<td>10.0, 13.0 or 13.0 – 20.0</td>
<td>6 – 24</td>
</tr>
</tbody>
</table>

1. Values of $\phi_r$, the residual friction angle, are intended as an approximate guide to the mineralogical properties of the alteration products, if present.
Appendix 5 Joint water reduction

<table>
<thead>
<tr>
<th>4. JOINT WATER REDUCTION</th>
<th>Jw</th>
<th>Approx. water pressure (kgf/cm²)</th>
</tr>
</thead>
<tbody>
<tr>
<td>A. Dry excavations or minor inflow, i.e. &lt;5 l/m locally</td>
<td>1.0</td>
<td>&lt;1.0</td>
</tr>
<tr>
<td>B. Medium inflow or pressure, occasional outwash of joint fillings</td>
<td>0.66</td>
<td>1.0 – 2.5</td>
</tr>
<tr>
<td>C. Large inflow or high pressure in competent rock with unfilled joints</td>
<td>0.5</td>
<td>2.5 – 10.0</td>
</tr>
<tr>
<td>D. Large inflow or high pressure</td>
<td>0.33</td>
<td>2.5 – 10.0</td>
</tr>
<tr>
<td>E. Exceptionally high inflow or pressure at blasting, decaying with time</td>
<td>0.2 – 0.1</td>
<td>&gt;10.0</td>
</tr>
<tr>
<td>F. Exceptionally high inflow or pressure</td>
<td>0.1 – 0.05</td>
<td>&gt;10.0</td>
</tr>
</tbody>
</table>

1. Factors C to F are crude estimates, increase Jw if drainage installed
2. Special problems caused by ice formation are not considered
Appendix 6 Stress reduction factor

<table>
<thead>
<tr>
<th>5. STRESS REDUCTION FACTOR</th>
<th>SRF</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>a. Weakness zones intersecting excavation, which may cause loosening of rock mass when tunnel is excavated</strong></td>
<td></td>
</tr>
<tr>
<td>A. Multiple occurrences of weakness zones containing clay or chemically disintegrated rock, very loose surrounding rock any depth</td>
<td>10.0</td>
</tr>
<tr>
<td>B. Single weakness zones containing clay, or chemically disintegrated rock (depth of excavation ≤ 50 m)</td>
<td>5.0</td>
</tr>
<tr>
<td>C. Single weakness zones containing clay, or chemically disintegrated rock (depth of excavation &gt; 50 m)</td>
<td>2.5</td>
</tr>
<tr>
<td>D. Multiple shear zones in competent rock (clay free), loose surrounding rock (any depth)</td>
<td>7.5</td>
</tr>
<tr>
<td>E. Single shear zones in competent rock (clay free) (depth of excavation ≤ 50 m)</td>
<td>5.0</td>
</tr>
<tr>
<td>F. Single shear zones in competent rock (clay free) (depth of excavation &gt; 50 m)</td>
<td>2.5</td>
</tr>
<tr>
<td>G. Loose open joints, heavily jointed or 'sugar cube' etc. (any depth)</td>
<td>5.0</td>
</tr>
<tr>
<td><strong>b. Competent rock, rock stress problems</strong></td>
<td></td>
</tr>
<tr>
<td></td>
<td>$\frac{\sigma_c}{\sigma_1}$</td>
</tr>
<tr>
<td>H. Low stress, near surface</td>
<td>&gt; 200</td>
</tr>
<tr>
<td>J. Medium stress</td>
<td>200–10</td>
</tr>
<tr>
<td>K. High stress, very tight structure (Usually favourable to stability, may be unfavourable to wall stability)</td>
<td>10–5</td>
</tr>
<tr>
<td>L. Mild rock burst (massive rock)</td>
<td>5–2.5</td>
</tr>
<tr>
<td>M. Heavy rock burst (massive rock)</td>
<td>&lt; 2.5</td>
</tr>
<tr>
<td><strong>c. Squeezing rock, plastic flow of incompetent rock under the influence of high rock pressures</strong></td>
<td></td>
</tr>
<tr>
<td>N. Mild squeezing rock pressure</td>
<td>5–10</td>
</tr>
<tr>
<td>O. Heavy squeezing rock pressure</td>
<td>10–20</td>
</tr>
</tbody>
</table>

1. Reduce these values of SRF by 25–50% if the relevant shear zones only influence but do not intersect the excavation.

2. For strongly anisotropic stress field (if measured): when $5 \sigma_1 / \sigma_3 \leq 10$, reduce $\sigma_c$ and $\sigma_t$ to 0.8 $\sigma_c$ and 0.8 $\sigma_t$. When $\sigma_1 / \sigma_3 > 10$, reduce $\sigma_c$ and $\sigma_t$ to 0.6 $\sigma_c$ and 0.6 $\sigma_t$, where $\sigma_c$ = unconfined compression strength, $\sigma_t$ = tensile strength (point load), $\sigma_1$ and $\sigma_3$ = major and minor principal stresses.

3. Few case records available where depth of crown below surface is less than span width. Suggest...
<table>
<thead>
<tr>
<th>P.</th>
<th>Mild swelling rock pressure</th>
<th>5 – 10</th>
</tr>
</thead>
<tbody>
<tr>
<td>R.</td>
<td>Heavy swelling rock pressure</td>
<td>10 – 15</td>
</tr>
</tbody>
</table>

**d. Swelling rock, chemical swelling activity depending on presence of water**

SRF increase from 2.5 to 5 for such cases (see H)