**Investigating the causes of stope instability at Golden Valley Mine**

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**ABSTRACT**

Ground stability issues are a major concern in underground mining excavations because if they are not properly addressed, they may cause loss of reserves, equipment and in worst cases, loss of life. This study is based on mining operations that are concentrated in a ground exposed to flooding with varying stope dimensions. Stope stability was assessed using the stability graph complemented by the equivalent linear over break slough (ELOS) stability approach. The stability graph, showed that the stopes which were in rock masses that had been exposed to flooding were highly unstable than in the ones which were not affected by flooding. The ELOS approach further showed that mining the previously flooded rock masses resulted in high over breaks in the stopes. Continuous implementation of the old support systems was no longer compatible with the state of the ground conditions post-flooding. Hence, a new support system with an acceptable factor of safety against failure in stopes was proposed.

**Keywords**: Ground, Stability, Flooding, Rock Mass, Factor of Safety

1. **INTRODUCTION**

The success of any open stope mining largely depends on the stability of un-reinforced stope walls and crowns as well as the stability of any exposed fill masses (Villaescusa, 2004). However, the complexity in the spatial distribution of rock-mass properties during site investigations result in the variability of physical and mechanical properties of the rock-mass and this often leads to poor design. Due to limited availability of geo-mechanical data during pre-feasibility and feasibility stages, most geotechnical design analysis is performed based on assumption and also on the use of discrete values of rock mass properties (Idris, et al., 2011). In such design, the result of such analysis can lead to an unstable stope that can cause accidents due to variation in the design parameters.

The need for this study was raised by Golden Valley Mine which is experiencing flooding at most of its mining levels. Prior to its flooding, mining operations at Golden Valley mine had progressed to Level 33. However, from the time the mine reopened to date, most of these levels have remained flooded. To sustain the mine until the levels are completely dewatered, the mine resorted to concentrate mining activities on the pre-mined stopes. This method would cause a significant increase in stope dimensions which would curb the mines financial problems. However, it did not take into consideration rock mass characterization, estimation of rock mass properties, identification of potential failure modes, appropriate stability analyses and other elements of the rock engineering design. Instead, the new panel lengths were solely dictated by the expected monthly production targets. This increase in stope dimensions was successfully implemented in all top levels but caused numerous ground stability problems from 21 Level.

The geological alterations brought about as a result of flooding has made the rock mass behavior in this area to be highly unpredictable. Rock mechanics problems are now being encountered and this is evidenced by both local and regional instabilities which has been occurring in these levels. Notably, falls of ground have been taking place resulting from blasting and failure of both the existing in-panel pillars and installed support. It was clearly observed that the production stopes were heavily fractured and were scaling thereby making them highly unstable.

The combinations of geological alterations in the host rock, inherent rock properties and induced stresses have created unfavorable geotechnical conditions resulting in instability of the rock mass surrounding production excavations. Sudden rock bursts, slabbing, collapse and ground movement continue to pose danger to the stability of ground. The mine has since recorded numerous injuries among workers as a result of the ground instability issues. It has become very difficult to meet the monthly production targets as stopes are temporarily closed for support reinforcement hence there is need to reduce these “stop and fix” scenarios. The mine has also lost a significant amount of money in compensating and also footing the medical bills for all the injuries that occurred and will continue losing its valuable revenue while meeting the above-mentioned avoidable expenses. It will also remain with a tainted reputation and a demotivated workforce due to low levels of safety in working areas if the problem of ground instability is not addressed with the immediate attention it deserves.

In order to solve these above-mentioned problems at Golden Valley Mine, this paper investigates the causes of instability at the mine with the primary aim of improving the ground condition by providing an alternative compatible support system. The rock mass classification approach of the Q-system was used to assess various ground conditions and to properly classify the rock in each stope. The stability chart was also used as an alternative method to evaluate the stability of all the four stopes. This would provide appropriate information about the ground conditions and support that is required to stabilize the roof prior to installation of support systems.

**1.1 Description of Golden Valley Mine**

Golden Valley Mine is located 20km west of Kadoma town in Mashonaland West province of Zimbabwe, as shown in Figure 1.



**Figure 1: Geographic Location of Golden Valley Mine (Google Maps, 2021)**

The shear zone hosting the Golden Valley Mine ore bodies is about 10-15m wide with a strike approximately 1700m, striking 10o north and dipping 35o west. The host rocks are massive and pillowed basalts and andesites which have been intruded by quartz porphyry sheets. The gold bearing reef is cut by an east–west trending normal fault zone (Pioneer fault) resulting in two discrete ore bodies, the south ore body dips at 27o-30o and the north ore body dips at 33o-37o.The northern ore body is characterized by reefs of smaller width and greater gold content whilst the southern side has reefs of wider width and lesser gold content. The Maida Vale fault, Golden Valley fault and other small terminal faults also cut the reefs at various points.

The mine extracts ore deposits using sublevel open stoping which is an underground mining method used in competent ground conditions that requires minimum or no support. The method involves the creation of large open areas known as ’stopes’ and is used in large steep dipping ore bodies. An average stope height of 2.0m is normally maintained but it can be increased to 2.5m in cases where the footwall will be having high grade. Support installations follow mine standards based on the joint strike directions and the brows created due to mining and the presence of fault intrusions. Pillars, usually with dimensions of 4m×4m are left behind randomly to support the stopes. In cases where the stopes have advanced, the support system is improved by installing timber props in conjunction with mat packs.

1. **Stope Stability in Open Stope Mines**

A fundamental step in the design of any proposed underground excavation is the evaluation of both the natural stability and mechanical modes of instability in the design of the support system in order to minimize their failure (Stacey and Page, 1986). Instability in mining operations is mainly due to high stress to strength ratio conditions, structurally controlled mechanisms or a combination of induced stresses and geological structures.

Generally, geological structures in the rock govern the stability of near surface structures and the natural in situ stresses govern the stability of deep structures. However, at medium depths in weak rocks and at considerable depths in strong rocks the natural stress, which is usually altered by the mining excavation, can be the dominant problem.

* 1. **Factors that cause rock mass failure**

Failure in rocks occurs when the rock strength is overcome by the imposed stresses. The strength of a rock can be described as the extent to which a rock specimen can tolerate the process of stress redistribution before it fails. Therefore, factors that tend to increase the stresses in rocks increase the chances of failure. Increased pore pressure, cracking, swelling, decomposition of clay rock fills, creep under sustained loads, leaching, strain softening, weathering and cyclic loading are common processes that decrease the strength of rock mass which leads to failure in rocks. The other factors that lead to rock failure by affecting the rock mass strength are shown in Table 1.

**Table 1: Factors that cause failure in rock masses**

|  |  |  |
| --- | --- | --- |
| **Factor no.** | **Name of the parameters and properties**  | **Details**  |
| 1  | Geological discontinuities  | Faults, joints, bedding planes  |
| 2  | Water  | Ground-water, drainage pattern, rainfall, permeability, aquifer  |
| 3  | Strength of rock material  | Shear strength, compressive strength, tensile strength  |
| 4  | Geotechnical parameters  | Grain size, moisture content, etc.  |
| 5  | Dynamic forces  | Blasting, seismic activity  |
| 6  | Geometry of excavation  | Bench height and bench angle, concave or convex shape. |

As mentioned earlier the case study mine has most of its stopes submerged in water, thus review of the effects of water on stope stability has been done in this paper. The effect of water on the rock strength can be considered into two-fold. Firstly, it is ground water or aquifer below the surface that generates pore water pressure. Secondly, it is rainwater infiltration that seeps through the surface and flows along the geological discontinuities thereby generating water pressure. Both effects are however related to the surrounding precipitation levels, topography, nearby water masses and the geo-hydrological characteristics of the rock mass (Sjöberg, 1999). Water pressure acting within a discontinuity reduces the effective normal stress acting on the plane and this in turn reduces the shear strength of the discontinuity plane. Figure 2 illustrates how water typically affects the stability of rock masses in mining operations. The water applies horizontal and vertical pressure along the discontinuity plane. The uplift force U is also developed due to water at the surface between the block and its base. The water pressure increases linearly with depth down to the intersection of the sub-vertical plane with the base and linearly decreases from the intersection point to the lower edge of the block in contact with the surface where the water pressure is zero

 

**Figure 2: Effect of water on discontinuity planes**

In addition to the ground water which exists nearly everywhere beneath the earth’s surface, rainfall water also adds weight on the rock mass. Such water fills the pore spaces between the grains or fractures in the rock and can seep into discontinuities present in the rock mass, replacing the air in the pore space thus increasing the weight of the rock. This leads to an increase in the effective stress resulting in failure of the underground openings.

* 1. **Application of Rock Mass Classification in Open Stope Mining**

From the earlier discussions, it can be deduced that a rock mass is generally weaker and more deformable than its constituent rock material as the mass contains structural weakness planes such as joints and faults. The stability of an excavation in a jointed rock mass is therefore influenced by a number of factors including the confining stress of water. The best practical way in which the weakening effect in rock masses can be considered is by applying rock mass classification methods.

Lauffer (1958) came up with a system that links the rock types, to the stand-up time of any active unsupported span. The system explains how an increase in this span leads to a significant reduction in the time available for installing support (known as the stand-up time). In designing the support for hard rock excavations, it is prudent to assume that the stability of the rock mass surrounding the excavation is not time dependent. Pacher et al. (1974) modified Lauffer’s system such that the method now includes a number of techniques for safe tunneling in rock conditions where the stand-up time is limited before failure occurs.

The Norwegian Geotechnical Institute (NGI) also proposed an index (Q) for the determination of the quality of rock mass (Barton, 1974). The index relates its different values to the different types of permanent support by means of a schematic support chart. This means by calculating the Q-value, it is possible to determine the type and quantity of support previously applied to rock masses with similar qualities. The Q-value can be obtained from using Equation (1).

$Q=\frac{RQD}{Jn}×\frac{Jr}{Ja }×\frac{Jw}{SRF} $ (1)

* 1. **Stope Stability Analysis**

Bieniawski (1989), described ground control as a method used to minimise all risks associated with various forms of ground movement and inundation in underground mines within acceptable levels. Therefore, it is imperative that the potentially diverse range of ground characteristics around and within the mine are recognised and the mine planning and design techniques are well understood before any excavation is made in order to achieve safe and cost-effective ground control (Bieniawski, 1989).

Numerous studies have discussed various methods used to investigate the stability of stopes, these include empirical, numerical, statistical method as well as in-situ test and monitoring methods. The empirical method of analyzing stope stability was proposed by Mathew et al. (1981) which entails the use of a stability number (N) to define the bearing capacity of rock mass to resist failure and hydraulic radius (HR) to denote the geometry of slope face termed stability graph (Saadaari et al., 2020). The numerical approach makes use of computer codes such as FLAC model to assess the stability of open stopes (Idris, et al., 2011)..

The statistical method is used to determine the accuracy and probability of stability boundaries (Sharp, 2011). The statistical approach by Mawdesley (2004) used logarithm regression analysis to re-examine the stability of the mine. This approach improved the Mathews stability graph by increasing the number of project cases to 500 in order to quantify the uncertainties of the Mathews design tools (Zhang, et al., 2018). The graph is divided into three segments, that is, the stable area, unstable area and caving zone as shown in Figure 3. The N-value represents the stability number which denotes the competency of the rock while S-value represents shape factor which is also known as the hydraulic radius (HR) which accounts for the geometry of the surface (Zhang, et al., 2018). In order to improve the reliability of the graph, Potvin (1988) added some transition zones within it as shown on Figure 3.

 

**Figure 3: Improved Mathews stability graph by Potvin 1988 (Jang, 2014).**

Another important parameter in the evaluation of stope stability in underground excavations is the stope performance (ELOS). It is described as the linear over break or slough of a stope and is an alternative way of expressing the volumetric measurements of the rock over break. It can be calculated using Equation (2)

$ELOS (m) = \frac{Volume overbreak (m^{3})}{Surface area of stope (m^{2})}$ (2)

* 1. **Ground Support and Ground Reinforcement**

The terms ground support and ground reinforcement are often used interchangeably however, they refer to two different approaches to stabilising rock (Stillborg, 1994). Ground support is applied to the perimeter of the excavation in order to limit movement of the rock mass while ground reinforcement is installed beyond the perimeter of the excavation, deep into the rock. Pillar support system is the most common type of support being used especially where weak ground conditions are experienced. It is also the only natural type of support hence its proper design is of great importance. The design of pillars for support of mine openings requires the determination of two aspects:

* the pillar strengths; and
* the stresses acting on the pillars.

Once the pillar strength and pillar stress are known, the factor of safety (FOS) of the pillar can be determined as the ratio of the strength to stress. A FOS of unity is equivalent to a probability of failure of 50%. The choice of the FOS value to be used for the design of the pillars and layout depends on the function of the pillars. In most cases, ground support systems are complemented by ground reinforcement elements such as roof bolts, cable anchors in conjunction with wire mesh and straps.

1. **Materials and methodology**

A study of the mine design plan was carried out to determine the extent to which the various stope dimensions had been increased. Survey and geological data were used to analyse and correlate the nature of discontinuities to the falls of ground.

* 1. **Rock Mass Classification**

Reclassification of the mines rock mass was done in order to account for the changes brought about by water flooding. Reclassification methods listed below were used to determine the ground classes and ascertain the rock quality.

1. Physical Observations;
2. Rock Tunnelling Index; and
3. Stability number.
4. **Physical Observations**

This entails comparing of the properties of the different rock masses found in the stopes. To achieve this, four different stopes with the following conditions were used:

* Stope A – not affected by flooding but extended;
* Stope B – affected by flooding and extended;
* Stope C – affected by flooding but not extended; and
* Stope D – not affected by flooding and not extended.

Each of the stopes rock mass was visually assessed and samples were taken for critical laboratory analyses. Parameters such as the rock strength, joint roughness, joint infill, and fractures present were analyzed. The correlation of jointing to rock falls and to the nature of the surrounding rock mass was noted. This would then enable the prediction of the likelihood of rock falls in rock mass with similar state of jointing as those already observed.

1. **Rock Tunneling Index**

Joint parameters such as joint alteration, joint number and joint roughness were used together with the rock quality designation to determine the rock tunnelling quality index (Q). The ground classes were then assigned based on the range of Q-values obtained.

1. **Stability Number**

Adopting the same joint parameter values used to calculate Q, the modified Q value denoted by $Q'$ was determined. The calculated $Q'$ was used to determine the stability number using Equation (3).

$N' = Q' × A × B × C$ (3)

Where, $N'$ is the stability number

 $Q'$ is the modified rock tunnelling index;

 A is the rock stress factor;

 B is the joint orientation adjustment factor; and

 C is the gravity adjustment factor.

* 1. **Stope Measurements**

The length, width, height and advance for the different stopes were measured using a distometer. These dimensions were then analysed to see their effect on the stope stability using the hydraulic radius and the stability graph. The stability number, N, and the shape factor, S, (Hydraulic Radius) for each stope were plotted on a stability graph to determine the stability of the stopes.

The advance per blast for each stope was measured on daily basis for a period of one month. The total of the daily advances divided by the number of days gave the average advance per day. The average daily advance was then extrapolated in order to calculate the expected daily tonnage for each stope. The difference between the expected daily tonnages and the average actual tonnages were used to determine the volume of over break for each of the stopes. Stope performance was also calculated using the volumes of over break and surface areas of the stopes. ELOS range of values suggested by Mathews (1981) were used to determine the stability of the stopes.

* 1. **Determination of Maximum Tolerable Unsupported Length and Support Method**

The average vertical and horizontal lengths between support holes was measured using a distometer. The maximum unsupported length was determined using the calculated Q-values and excavation support ratio ESR values suggested by Mathews (1981). The performance and effectiveness of the installed support elements were analysed with regards to stope heights and ground characteristics to ascertain whether the failures could be attributed to the support elements, the ground conditions or both. The analysis included studying the support systems that had failed and also that which were susceptible to failure. Investigation of the effectiveness of the available pillars was also done by measuring the pillar dimensions and calculating the pillar strength, stress and the factor of safety. Fall out heights of falls of ground were measured using a distometer and these were used to determine the average volume of rock above the roof requiring support. Alternative support elements were then determined basing on the support demand for each of the various stopes.

1. **Results and Discussion**

This section gives an overview of the research findings followed by the discussion of those findings. The findings are based on the field observations and documented geological, mining and survey technical reports.

* 1. **General Observations**

Discontinuity survey sheets showed that joint structures were dominant and persistent mostly in previously flooded and extended stopes and major rock falls were common in those stopes. This indicated that formation and extension of fractures during drilling and blasting operations was probably the major cause of the disintegration which resulted in stope instability. A correlation of the nature of joints to the falls of ground was done and the results are shown in Table 2. It was observed that the pervasive carbonate alteration of the host rock (greenstone) occurred during the flooding period and this made the host rock to be weaker than before, hence it was now fracturing more easily.

**Table 2: Correlation between joints and falls of ground**

|  |  |
| --- | --- |
| Jointing nature of rock mass | Falls of Ground |
| No joints to small minor cracks | Absent |
| Smooth hanging wall (shear plane) without visible joints | Absent |
| Presence of reef sub-parallel shear plane and 1 joint set or 2 joint sets and random joints | Tabular blocks |
| Presence of a narrow shear <3m wide or fault zone that is associated with steep and shallow joints without intersecting joints | Blocks |
| Presence of shallow dipping parallel joints with same or opposite dip direction (dip<45), striking parallel or sub parallel to panel advance direction | Wedge  |

A study of the mine plan showed that most of the stopes which were above 23-Level had been extended to almost double their initial size and expansion of stopes in the levels below 23-Level was ongoing. This expansion was however paying no particular attention to neither the adverse effects imposed by the clearly visible geological discontinuities nor the rock strength deterioration induced by flooding.

* 1. **Quantification of the Rock Mass Quality**

The rock masses in the four stopes were studied in order to quantify their quality and subsequently be appropriately classified. The geotechnical parameter conditions of the rock masses in the four stopes were observed and the results were record in Table 3.

**Table 3: Rock mass parameters**

|  |  |  |  |  |
| --- | --- | --- | --- | --- |
| PROPERTY | STOPE A | STOPE B | STOPE C | STOPE D |
| Strength | **Method of Assessment** | Hand held specimen breaks under more than one blow of a hammer. | Cannot be peeled with a knife. Hand held specimen can be broken with one firm blow of a hammer. | Hand held specimen breaks under more than one blow of a hammer. | Many hammer blows are required tobreak an intact specimen |
| **Approx. UCS** | 70 to 150 | ≤70 | 70 to 150 | >150 |
| Joint filling | Non-softening sandy particles | Softening clayminerals | Low-friction clay minerals | Clay-free disintegrated rock |
| Joint Roughness | Rough and irregular | Undulating and planar | Slicken-sided planar | Rough and irregular |
| Ground water condition | Dry Excavation | Damp | Damp | Dry Excavation |
| Joint Spacing | Very narrow(2- 6mm) | Narrow(6-20mm) | Very narrow(2-6 mm) | Tight |
| Joint Separation | 60-200mm | <60mm | 60-200mm | 200-600mm |
| Persistence of Joints | 60-200mm | ≥200mm | ˃50mm | 20-40mm |

* + 1. **Rock Quality based on RQD System**

The RQD values for the rock masses corresponding to each stope were calculated using Equation (4) and the results are shown in Table 4. The rock classes were assigned based on the standard RQD range of values suggested by Deere et al. (1967).

$RQD=\frac{\sum\_{}^{}Length of core pieces\geq 100mm}{Total Length of core run}×100$ (4)

**Table 4: RQD Results**

|  |  |  |
| --- | --- | --- |
| Stope | RQD Value | Rock Class |
| A | 66 | Fair |
| B | 20 | Very Poor |
| C | 52 | Fair |
| D | 88 | Good |

The RQD value for Stope A was moderately high with a percentage core recovery of 66% as shown in Table 4. Following Deere et al. (1967) classification, an RQD of 66% falls in the “fair class”, which suggests that the rock mass in stope A is partially fractured and fairly competent hence few rock falls are expected in the stope. The fractures present may be mainly due to the presence of induced stresses caused by blasting. Percentage core recovery in Stope B was 20% which shows that the rock mass was highly fractured thus, it is very poor and incompetent. Unlike the rock mass in Stope A which is only affected by blast induced stresses, the rock mass in Stope B is affected by both the blast induced stresses as well as the pore pressure caused by flooding. Thus, it is valid to have the rock mass in Stope B being weaker than that in Stope A.

The rock mass in the Stope C falls in the “fair” category thus, implying that it is moderately fractured. An RQD of 52% for the rock mass in Stope C suggests that it is slightly competent than that of Stope B which has a considerably lower RQD. However, the RQD in Stope C is lower than in Stope A which suggests that the rock mass in Stope C is more fractured than that in Stope A. The deterioration of rock mass quality in Stope C increased due to the weakness induced by the water pressure experienced during the flooding period. If one is to compare the three stopes, it can be deduced that the water pressure enhanced the deterioration of the rock mass quality than what is caused by blast induced stresses. Nonetheless, the combined effect of the two is evidently catastrophic. Stope D has the highest value of RQD of 88%, which shows that the rock mass in this stope has the least frequency of fractures as compared to the other three stopes. The fewer fractures in rock mass may be attributed to mild effects caused by mining operations which took place in this region prior the flooding era.

* + 1. **Rock Quality based on the Q-System**

The Rock Quality Index (Q) is dependent on three parameters that is the degree of jointing, joint friction and the active stress. The three parameters were calculated for each of the four stopes and the results are shown on Table 5. These parameters were then used to determine the Q-values which are also shown on Table 5.

**Table 5: Rock Quality Index parameters**

|  |  |  |  |  |
| --- | --- | --- | --- | --- |
| STOPE | Degree of Jointing | Joint Friction | Active Stress | Q-values |
| A | 11.00 | 1.500 | 0.4000 | 6.6 |
| B | 6.67 | 0.375 | 0.0667 | 0.17 |
| C | 17.33 | 0.375 | 0.1320 | 0.86 |
| D | 14.67 | 1.500 | 1.000 | 22 |

The extension of Stope A resulted in a slight increase in the stress reduction factor which in turn lowered the stope’s active stress. This led to the stress magnitude becoming slightly greater than the rock strength, hence the moderate Q value of 6.6. The low Q-value obtained in Stope B (approximately 0.17) is mainly due to the lower joint water reduction factor, Jw and a substantial SRF value thereby making its active stress extremely low. In addition, the softening clay joint filling in the undulating and planar joints observed on this rock mass implies some low joint friction which cause the rock mass to be of poor quality. To this point, it is logical to have Stope B being the most unstable of the four stopes in consideration.

In general stope stability increases if the stope is cut in a rock mass with a high degree of jointing, thus Stope C is expected to be relatively stable. However, by having a low active stress value due to the stope’s high SRF and the low Jw, the stability of the stope is significantly reduced. A low Q-value of 0.86 for Stope C indicates that a combination of low active stress and low joint friction, due to clayey minerals found in the slicken-sided planar joints, makes Stope C to be highly unstable despite the high degree jointing. A substantially high Q-value of 22 was obtained on the rock mass in Stope D. The absence of water, favourable stress conditions in the region and the presence of clay-free disintegrating rock joint infill in rough and irregular joints (high joint friction) aided in having the rock mass in Stope D to be of high quality. A stope cut in a rock mass of high quality is expected to be stable, thus Stope D should be highly stable.

* + 1. **Evaluation of the Stability Number (**$N'$**)**

The stability number was also used to assess the stability of the stopes. The assessment was dependent on the modified Q-value, the rock stress factor (A), the joint orientation adjustment factor (B) and the gravity adjustment factor (C). The modified Q-values were calculated using Equation 5 and the results are presented in Table 6. Parameter values for the stability factors A, B and C, shown on Table 6, were adopted for the calculation of $N' $using Equation (3)$.$

$Q'=\frac{RQD}{Jn}×\frac{Jr}{Ja}$ (5)

**Table 6: Modified Q-values**

|  |  |  |  |  |  |  |
| --- | --- | --- | --- | --- | --- | --- |
| Stope | Joint Orientation | B value | A value | C value | Modified Q values | $$N'$$ |
| A | 20 | 0.2 | 0.7 | 7 | 16.5 | 16.17 |
| B | 60 | 0.6 | 0.2 | 7 | 2.5 | 2.10 |
| C | 30 | 0.3 | 0.5 | 7 | 6.5 | 5.04 |
| D | 20 | 0.2 | 0.8 | 7 | 22 | 24.64 |

In general, failure occurs along critical joints which form a shallow angle with the free face. Therefore, basing the stability of Stope A on a B factor of 0.2 alone means that the stope was likely to fail. However, the rock mass in Stope A had high A factor which in turn caused the stope’s stability number $N'$ to be fairly high (approximately 16). Stope B’s stability number of 2, shows that the stope is the most unstable. Ideally, it should have been the most stable since the critical joints in the stope form the widest angles (≈600). However, the low A and $Q'$ values greatly decreased its stability number thereby causing it to be the most unstable stope.

Although the stability number for Stope C is considerably low (approx. 5) due to the shallow angles between the critical joints in the stope, a fairly high rock stress factor of 0.5 makes the stope to be more stable when compared to Stope B**.** Stope D has the highest stability number of approximately 25, compared to the other stopes thus it is the most stable. This is mainly attributed to the stope’s high $Q'$ value of 22 and a high A factor of 0.8. These high $Q' $and A values reverses the adverse effect of the shallow angles between the stope’s critical joints. It is worth noting that the value of the gravity adjustment factor (C) was the same in all the stopes hence its effect in comparing the stability of the stopes was insignificant.

* 1. **Determination of the Hydraulic Radius**

The hydraulic radii for the stopes, shown in Table 7, were calculated using Equation (6). Stopes A and B have higher hydraulic radii compared to stopes C and D. This is mainly attributed to the fact that stopes A and B have been extended implying that they are larger than stopes B and C. Generally, the higher the hydraulic radius the higher the susceptibility of stope to instability, thus stope instability is expected to decreases from stopes A to D. The trend would be such that Stope A would be the most unstable, whereas Stope D would be the most stable.

$Hydraulic Radius= \frac{w × h}{2 ( w + h )}$ (6)

**Table 7: Stope Hydraulic Radius**

|  |  |  |  |  |  |
| --- | --- | --- | --- | --- | --- |
| Stope | Height | Width | Area | Perimeter  | Hydraulic radius |
| A | 41 | 38 | 1558 | 158 | 9.86 |
| B | 35 | 30 | 1050 | 130 | 8.08 |
| C | 24 | 15 | 360 | 78 | 4.62 |
| D | 18 | 11 | 198 | 58 | 3.41 |

* 1. **Determination of Stope Stability using the Stability Graph**

The calculated values of the stability number ($N'$) and shape factor, S, (hydraulic radius) for the stopes were used to ascertain the stability of the stopes using the stability graph in Figure 4. The effect of stope extension, flooding and a combination of the two were clearly brought out. Stope A and Stope C both fell in the potentially unstable zone. Although Stope A was not affected by water, its high hydraulic radius shows that it has been significantly extended and this caused some alterations in the stope’s stress distribution thereby making it potentially unstable. On the other end, the low hydraulic radius of Stope C indicates that the stope has not been extended, hence the stope has been made potentially unstable solely due to the increase in pore pressure caused by flooding. It can thus be deduced that both flooding and stope extension induce some instability in the rock mass.

Stope B lies in the potentially caving zone and the stope is highly unstable. Since the stope was extended after the flooding era, it can be concluded that the combined effect of pore pressure and mining induced stresses greatly affects the stability of the stope. Stope D falls in the stable zone because it has the highest stability number and the lowest hydraulic radius. Since the stope has neither been extended nor affected by flooding, the mining induced stress remained evenly distributed hence making it highly stable.



**Figure 4: Determination of Stope Stability on the Stability Graph**

* 1. **Analysis of Stope Performance (ELOS)**

ELOS values for the stopes were calculated using Equation (2). The data for daily advances, expected tonnage and over-break volumes which is required for the determination of ELOS values is presented in Table 8 below.

**Table 8: Stope performance (ELOS) data**

|  |  |  |  |  |  |
| --- | --- | --- | --- | --- | --- |
| Stope | Expected Volume | Actual Volume | Volume Over-break | Area | ELOS |
| A | 2555.12 | 3853.12 | 1298 | 1558 | 0.83 |
| B | 1837.5 | 4480.5 | 2643 | 1050 | 2.52 |
| C | 622.8 | 1202.8 | 580 | 360 | 1.61 |
| D | 306.9 | 402.9 | 96 | 198 | 0.48 |

Mathews(1981) came up with a standard system which puts the ELOS values into ranges corresponding to the likely type of stope. The ELOS for the Stope A (0.83) falls in the range of 0.75m - 2m where limited failure is expected from the unsupported stope. This is due to the considerably low volume of over break since the rock mass in this stope had not been affected by flooding. Hence, the stope was less susceptible to blast damage. Severe sloughing and huge failure volumes should be expected on Stope B if the stope walls are left unsupported. The ELOS value of 2.52 is mainly due to the high over break experienced as a result of the high fractured rock in the stope and wall collapse is highly possible. The high deterioration of the rock mass in Stope B can be attributed to a combination of blast induced damage and the weakening effect of water caused by flooding.

The high volume of over break experienced over a small stope face puts Stope C in the unstable zone. The quality of the rock mass in this stope was considerably deteriorated due to flooding. Thus, minor to moderate sloughing of the stope walls should be anticipated. An ELOS value of 0.49 for Stope D is classified under the stable zone. The ELOS value was the lowest because the volume of over break was small since the rock mass in this region was competent. Rock mass competence in this stope is possible since the stope had not been affected by both flooding and blasting disturbances. The stope walls can therefore be expected to be self-supporting.

* 1. **Correlation between ELOS and Hydraulic Radius**

Figure 5 shows a plot of the hydraulic radius against the ELOS which has been used to determine the factor which influence the quality of the rock mass to a greater extent. Stopes A and B were expected to have the most over break since they had the largest values of the hydraulic radius. However, it can be observed that the over break in Stope C was greater than that in Stope A despite it having a small hydraulic radius. Therefore, it can be deduced that although stope extension plays a part in the over-break experienced in different stopes, it is not the main cause of the overall stope instability. The relationship suggests that over-break is highest in the stopes affected by flooding, that is in stopes B and C. This result confirms the supposition that over-break is largely controlled by pore pressure than it is by blast induced stresses.

**Figure 5: ELOS versus Hydraulic Radius**

* 1. **Determination of the Maximum Tolerable Unsupported Length**

After ascertaining the ground conditions, there was need to determine the maximum length that could be left unsupported for each stope. Stillborg (1994) came up with a formula to calculate the maximum tolerable unsupported length, Equation (7).

$Maximum unsupported length =2×ESR×Q^{0.4}$ (7)

Where, *ESR* is the excavation support ratio which is empirically deduced and *Q* is the Q-value deduced from the Q-system. Thus, in this case an ESR value of 1.6, assigned on permanent mine openings as proposed by Mathews (1981), was adopted in the calculation of the maximum unsupported length for the four stopes. The calculated values were then compared with the actual measured lengths of the unsupported spans in each stope and the data is tabulated in Table 9 below.

**Table 9: Maximum Tolerable Unsupported Length**

|  |  |  |
| --- | --- | --- |
| Stope | Actual Unsupported Span (m) | Maximum Tolerable Unsupported Length (m) |
| A | 3.9 | 6.8 |
| B | 3.9 | 1.6 |
| C | 3.9 | 3.0 |
| D | 3.9 | 11.0 |

From Table 9, it can be deduced that stopes A and D can be extended to approximately 6.8m and 11m respectively without requiring any support installation. Although, the standard unsupported span of 3.9m being used at the mine is lower than the maximum tolerable unsupported lengths for stopes A and D, it is greater than that which can be tolerated in stopes B and C. Therefore, if stopes B and C are left unsupported ground failure is likely to occur. This can be taken as a clear indication that the ground quality in stopes B and C has significantly deteriorated. Although, the ground in stopes B and C could be extended with no adverse consequences prior to flooding, it now requires some additional support at spans less than the standard mining span of 3.9m.

* 1. **Support Design**

The current support system being implemented at mine comprises of mat packs, natural in-situ pillars and timber props. The study showed that the stability of timberprops decreased with increasing height and length of the stopes. Timber props in Stope D were highly stable whereas those in Stopes A and B buckled at variouspanellengths. The stability of mat packs also decreased with increasing stope height. Daehnke et al. (2001) mentioned that it is commonly assumed that timber packs with height-to-width ratio exceeding 2:1 are unstable, particularly during dynamic closure.Hence, the use of mat packs in Stope A and B was no longer effective as the height-to-width ratio of 2:1 had been exceeded. Falls of ground were now prevalent in these stopes thus showing that the mat pack support systems had been rendered ineffective.

In-situ pillars of 4m×4m in dimension are left out as support in most of the stopes at the mine. In very bad ground, the pillar size is increased to be 5m×5m. Pillar spalling was now becoming the major problem with these natural in-situ pillars mainly due to excessive overburden stresses. Pillar overloading was due to the significantly increased stope dimensions whilst maintaining the same pillar dimensions. As a result, the recommended factor of safety of 1.6 on hard rock pillars was lowered beyond this point such that the pillars were now ineffective. Table 10 shows the average safety factors for the four stopes. The pillar strengths have been calculated with the design rock mass strength (D.R.M.S) assumed to be 37.5% of the UCS.

**Table 10: Factor of safety for the pillars in the various stopes**

|  |  |  |  |  |  |  |  |
| --- | --- | --- | --- | --- | --- | --- | --- |
| **Stopes** | **Average virgin stress (MPa)** | **Pillar dimensions** | **Effective pillar width (m)** | **Areal extraction (in panel) (%)** | **Pillar strength (MPa)** | **Pillar Stress (MPa)** | **Safety Factor** |
| **Pillar length (m)** | **Pillar width (m)** | **Height (m)** |
| A | 4.8 | 4 | 4 | 2.5 | 4 | 86.8 | 41.495 | 36.4 | 1.14 |
| B | 5.1 | 5 | 5 | 2.5 | 5 | 92.5 | 29.523 | 68.0 | 0.43 |
| C | 5.1 | 5 | 5 | 2.5 | 5 | 82.5 | 46.393 | 29.1 | 1.59 |
| D | 4.8 | 4 | 4 | 2.5 | 4 | 80.8 | 56.585 | 25.0 | 2.26 |
| $$Pillar Strength=D.R.M.S × {W^{0.5}}/{H^{0.75}}$$ |

To determine the support demand, the average thickness of rock requiring support should be determined. The average thickness was estimated by averaging the fall-out heights that had been recorded in each of the stopes and these are shown in Table 11. The support demand per square meter was calculated using Equation (8).

$Support Demand/m^{2}= ρgh$ (8)

Where, ρ is the density of the rock (taken to be 2700kg/m3);

g is the acceleration due to gravity; and

h is the rock block thickness (average fall-out height)

**Table 11: Average fall-out height and support demand**

|  |  |  |  |  |
| --- | --- | --- | --- | --- |
| STOPE | FALL-OUT HEIGHTS (m) | Cumulative Fall-out height (m) | Average Fall-out Height (m) | Support Demand/m2(kN) |
| **1** | **2** | **3** | **4** | **5** |
| A | 1.56 | 0.97 | 1.65 | 1.11 | 1.34 | 6.63 | 1.33 | 35 |
| B | 3.20 | 3.41 | 3.52 | 2.90 | 3.95 | 17.0 | 3.40 | 90 |
| C | 2.88 | 3.20 | 2.90 | 2.78 | 2.43 | 14.2 | 2.84 | 75 |
| D | 0.41 | 0.32 | 0.43 | 0.47 | 0.36 | 1.99 | 0.40 | 11 |

Using the calculated rock support demand for the various stopes and the maximum load carrying capacities of different support elements on the market, the appropriate support elements to be used in each of the stopes were ascribed as shown on Table 12 below. The recommended support systems are not only based on the support demand but also the maximum tolerable area and the factor of safety for the varying ground conditions.

**Table 12: Recommended Support Systems**

|  |  |
| --- | --- |
| **Support Element Specifications** | **Stope Support Pattern** |
| **Support element** | **Load bearing capacity** | **Stope** | **Recommended Support Elements** |
| Timber Props | 50kN | **A** | - Timber props spaced at 4m apart- 4m×6m in-situ pillars |
| Mat packs | 80kN | **B** | - 20mm anchored rock studs spaced 2m apart or 16mm cone bolts spaced at 1m apart- 6m×8m in-situ pillars |
| Roof bolts (Shepherd's Crook cone bolts) | 16mm – 100kN | **C** | - 16mm rock studs spaced at 2m apart- 6m×8m in-situ pillars |
| Anchored rock stud | 20mm – 200kN16mm – 150kN13.5mm – 100kN | **D** | - timber props spaced at 4m apart- 4m×4m pillars |

1. **Conclusion and Recommendations**

The mine under review involved mining infrastructure which has been affected by flooding and in turn this has caused the rock mass quality to significantly deteriorate. Mining has been progressing well during the post-flooding era up until 21-Level where it is now very evident that the stopes below this level are now highly unstable and the support method is now incompatible with the new ground conditions. The research has been prompted with the aim of investigating the causes of stope instabilities at Golden Valley Mine and consequently, providing an alternative compatible support system.

It can be concluded that the pervasive carbonate alteration that occurred in the host rock during the flooding period has clearly weakened the rock mass. Development of numerous fractures upon a single blast is confirmation that the rock mass strength has substantially deteriorated. These fractures have a great adverse effect on the quality of the rock mass which ultimately cause stope instabilities. In addition to blast induced fractures, high pore pressure in the moderately altered joint walls in post flooded excavations has caused washing out of most of the joint fillings thereby inducing ground failure along these discontinuities. Falls of ground have now been so prevalent in the previously flooded stopes as was noticed in stopes B and C in which the maximum tolerable unsupported lengths have been significantly reduced.

It was also clearly seen that stope stability is adversely affected by the increase in stope dimensions. This can be attributed to the redistribution of stresses to the rock mass surrounding the stopes such that the in-panel pillars will be highly stressed thereby triggering stress induced ground failures. The stress induced failure would manifest as spalling or slabbing on the pillars and in some instances, it manifested in the form of small rock bursts. In addition, in order to release this stress, rock masses tend to fracture with ease and this leads to stope over-breaks. As a result, smaller excavations (those which have not been extended) such as stopes C and D are mostly likely to be more stable. Stability in stopes C and D has been caused by the low hydraulic radii which is contrary to the high hydraulic radii in stopes A and B. This is premised on the fact that the higher the hydraulic radius the higher the susceptibility of stope to instability. It was observed that over-break was highest in stopes which were affected by flooding, that is in stopes B and C. This shows that the invasion of water in mining excavations have greater influence on stope stability.

The support system in any excavation depends on the quality of the rock mass and the stopes support demand. Therefore, incompetent rock masses such as that in Stope B require some comprehensive support systems involving support elements which are closely spaced in order to counter major rock falls. The varying ground conditions entail different maximum tolerable unsupported lengths and support systems. Thus, the implementation of a support system should never be a one size fits all without considering the rock characteristics or else, the support system is rendered inefficient.

Rectangular pillars should be used instead of square pillars since they are highly capable of abating ground failure imposed by high horizontal stresses being incurred at the mine. The mine should implement 6m×8m pillars which have proved to have an acceptable factor of safety against failure. However, in areas where it is observed that the ground is excessively weak, the dimensions should be changed to suit such conditions. In addition, a ground monitoring program should be implemented in order to monitor the performance of the ground support systems. For this to be effective, there is need to consider installing support elements coupled with electronic sensors which give early warning signs in real time. This will allow the timeous evacuation of personnel and equipment from the danger zones.

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