REHABILITATION OF THE DECLINE DUE TO SWELLING ROCK FORMATION AT HARMONY GOLD MINE TSHEPONG NORTH SHAFT

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SYNOPSIS

Tshepong North Shaft, one of Harmony's largest gold-producing mines in the Free State, is situated in the portion of the Witwatersrand Basin, East of Odendaalsrus and North of Welkom, South Africa. The gold-bearing ore is extracted from two separate reef horizons, namely the Basal and B-reefs of which 69% is planned from the Basal reef and the remaining 31% is from the B-reef. The B-reef is situated 140 m above the Basal, with current mining taking place at depths ranging between 1 493 and 2 290 m below the surface.

The Tshepong North main shaft was sunk to a depth of 2 148 m below the surface with 7 working levels of which 66 level is the bottom mining level. The access to the ore reserves below 66 level was gained through decline/chairlift excavations which were extended down to 75 level at a depth of 2 290 m. Squeezing-type rock formation was encountered between the 73 and 75 levels where extensive rehabilitation had to be done to ensure access to and from the 75 level remains open and stable.

This paper discusses the investigation and rehabilitation method that was employed to rehabilitate and maintain an operating decline. It also discusses constraints and challenges that had to be overcome.

INTRODUCTION

Harmony Gold Mining Co Ltd currently operates nine underground mines in South Africa of which five mines are currently operating in the Welkom area in the Free State. The main vertical shaft was sunk up to 66 level which was planned to be the bottom mining level. Feasibility studies were done to extend the minable ore reserves below 66 level of which the first was started in 2001. Access was gained by developing a 6 m wide x 4 m high decline with a 4 x 4 m chairlift excavation at an inclination of 12°, at an apparent dip direction through the strata.

The decline was developed using trackless mobile machinery which allowed for daily drilling and blasting operations to be planned. The initial decline system was developed and completed from 66 level up to 71 level in 2006 as part of the sub-66 decline project.

The success of the initial project prompted the investigation of a second feasibility study which was to extend the decline down to 75 level elevation. The sub-71 decline extension was initiated and continued without any incident until a massive fall of ground incident occurred a few meters below the 73 level station in 2009. **Refer to photo 1.**

The fall of ground in the decline was removed and extensively re-supported. The re-establishment of the decline excavation took a total of 9 months to complete. The decline remained stable following the initial rehabilitation. In 2014 it became evident that the RSJ sets that were installed below the decline rehabilitation brow started to deform. **Refer to photo 2**.

The deformation was initially monitored to establish the extent of the movement and it became evident in 2016 that the area would have to be rehabilitated as the support was no longer able to maintain the integrity of the excavation due to the extent of the rock deformation. The rehabilitation was started at the end of 2017 and will be discussed in detail in the paper.

Photo 1 Decline collapsed 2009

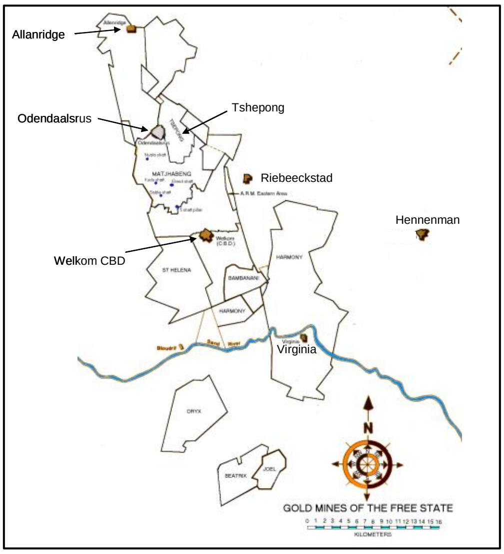
Photo 2 RSJ sets deforming

GEOLOGY

Location

Tshepong North Shaft is situated in the Free State portion of the Witwatersrand Basin, East of Odendaalsrus and north of Welkom. Figure 1 The Witwatersrand Basin is the main gold-bearing structure in South Africa. The rocks were deposited by sedimentation some 2 700 million years ago and the gold-bearing reef horizons are situated within the Upper Witwatersrand conglomerates. The rocks are made up of quartzite (deposited river sand), conglomerates (deposited river gravels), and shale (deposited mud). The conglomerates have pebbles of chert and quartz with a matrix of quartz gravel, silicates and various sulphides (mainly pyrite). The area underlain by accessible gold-bearing reefs in the Free State is about 50km long and 11km wide, stretching from Allanridge in the north to Virginia in the south.

Figure 1 - Location map of the Free



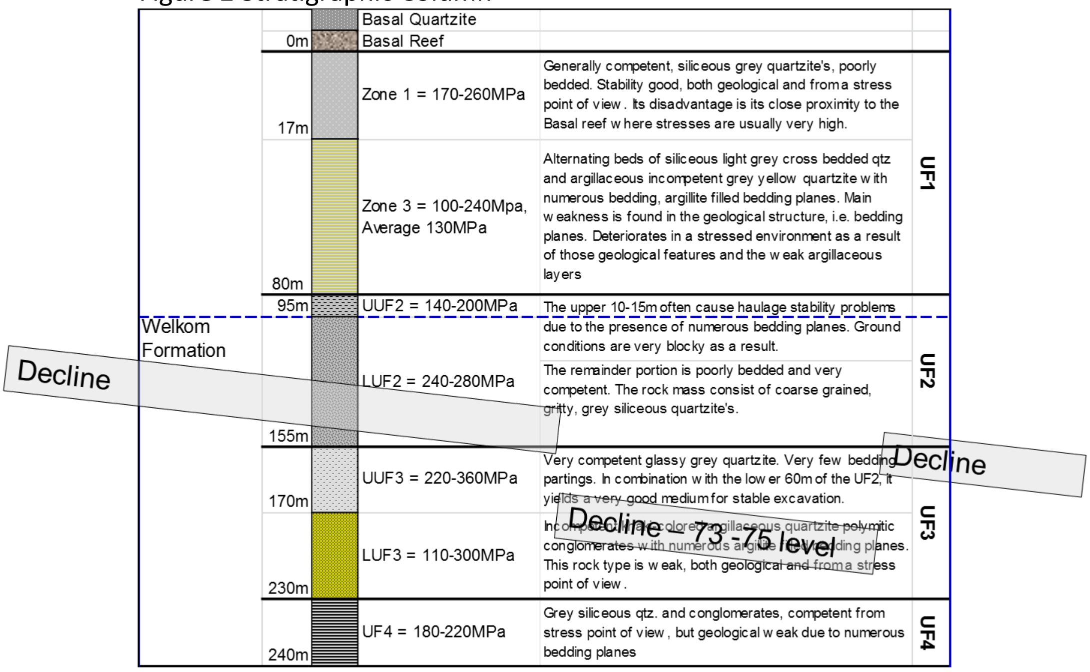
Ore Bodies Mined

Two reef horizons are mined namely the Basal and B-reef of which the Basal reef is the main orebody. The ore bodies dip between 5 and 38° with the average dip being 25°. Generally, the Basal reef strikes North-South but can vary somewhat in the North-Western area. The depth of mining ranges between 1 500 and 2200 m below the surface. The B-reef is situated some 140 m above the Basal reef and extraction is erratic due to the variation in grade distribution.

Footwall Formation

The Basal footwall formation is situated within the Welkom formation which is made up of the UF1, UUF2, LUF2, UUF3 and LUF3 formations. The footwall formation varies from very competent siliceous quartzite with Uniaxial Compressive Strength (UCS) ranging between 200 and 280 MPa to an argillaceous quartzite with a UCS of between 60 and 180 MPa.

Figure 2 Stratigraphic Column



The decline was initially planned to be developed through the competent LUF2 and UUF3 rock formation which is located between 95 to 172 m below the Basal reef horizon. The rock formation has a Uniaxial Compressive Strength varying between 220 and 280 MPa and has the following characteristics, very *competent glassy grey quartzite and very few bedding partings. In combination with the lower 60 m of the UF2, it yields a very good medium for stable excavation.*

The decline excavations remained in the competent LUF2-UUF3 up to just above 71 level in the chairlift and just above 73 level from where the Dolerite dyke was intersected. The footwall in which the declines were being developed was displaced upwards placing the decline in the weak LUF3 rock formation which is located between 172 and 202 m below the Basal reef horizon. The LUF3 rock formation has a UCS which varies between 110 and 300 MPa. Point load strength tests that were done yielded an average rock strength of 170 MPa.

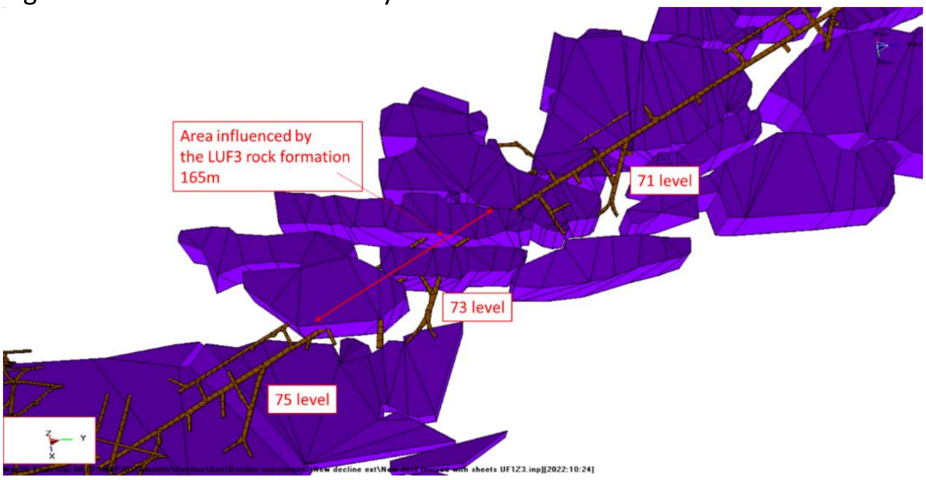
The LUF3 is characterized as follows; *Incompetent khaki-coloured argillaceous quartzite polymictic conglomerates with numerous argillite-filled bedding planes. This rock type is weak, both geologically and from a stress point of view.*

The decline excavations remained in the LUF3 rock formation up to the 75 TMM workshop where displacement along a fault put the decline back into the competent UUF3 rock formation. **Figure 3**

Table 1 Decline area affected by the weaker LUF3 formation

Rock formation	Below reef	Rock Strength (UCS)	Characteristic
UF2	80 – 95 m	UUF2 = 140-200 MPa	The upper 10-15m often causes haulage stability problems due to the presence of numerous bedding planes. Ground conditions are very blocky as a result.
	95 -155 m	LUF2 = 240-280 MPa	The remaining portion is poorly bedded and very competent. The rock mass consists of coarse-grained, gritty, grey siliceous quartzites.
UF3	155 -172 m	UUF3 = 220-360 MPa	Very competent glassy grey quartzite. Very few bedding partings. In combination with the lower 60m of the UF2, it yields a very good medium for stable excavation.
	172 – 202 m	LUF3 = 110-300 MPa	Incompetent khaki-coloured argillaceous quartzite polymictic conglomerates with numerous argillite-filled bedding planes. This rock type is weak, both geologically and from a stress point of view.

Figure 3 Decline area affected by the weaker LUF3 formation



Historic Support Design

The MAP3D numerical modelling software package was used to calculate the major principal stress and Rock wall Condition Factor (RCF) along the decline.

The following parameters were used in the Numerical Model to calculate the RCF value.

UUF2 : UCS: 140-200 MPa

LUF2 – UUF3 : UCS: 220-280 MPa

LUF3 : UCS: 110-280 MPa (AVG 170 MPa)

Young Modules (E) : 70 GPa

Poisons Ratio : 0.2

Failure F factor : 0.7 (Brentley; 2001) (Used in RCF calculations)

Rock wall condition factor

The Rockwall Condition Factor is a criterion used in hard rock mines for the control of tunnel conditions and the recommendation of suitable support. (Ryder and Jager 2002)

$$R.C.F. = \frac{(3\sigma_1 - \sigma_3)}{F\,\sigma_c} \qquad (1)$$

- $\sigma 1$ and $\sigma 3$ are the maximum and minimum field stress components in the plane of the excavation cross-section and F is a factor that represents the downgrading of σ_c, the uniaxial compressive strength, for the rock mass condition. In a highly discontinuous rock mass, it is recommended that F is approximately 0.5 and in large excavations (> 6 x 6 m) F may be further downgraded by 10 %. The formulation of the RCF represents a comparison of the maximum induced tangential stress of an excavation to the estimated rock mass strength. Wiseman (1979) used this criterion to examine the implementation of support systems in South African Witwatersrand gold mine tunnels.

- This allowed the development of an empirical relationship between the support systems and typical (3 x 3 m) mine tunnels in this specific geotechnical environment. In general, it was found that for RCF < 0.7 good conditions prevailed with minimum support (Appendix B); for 0.7 < RCF < 1.4 average conditions prevailed with typical support systems (Appendix B); and for RCF > 1.4 poor ground.

- The F value is assigned a value between 0.5 and 1. The higher the value the more competent the rock mass and vice versa. Five parameters were suggested by KR Brentley (2001) to be used to determine the F value which intern downgrades the UCS value for the rock mass. The following equation was suggested for tunnels with dimensions of 4 x 4 m or less; (refer to Appendix B)

$$F = (S + C + B + D + SP) \div 50. \qquad (2)$$

- The following equation was selected for use where the tunnel dimension is greater than 4 x 4 m, but less than 6 x 6 m.

$$F = [(S + C + B + D + S) \div 50]\ 0{,}90 \qquad (3)$$

- Where S defines the slabbing on the tunnel sidewalls, C the fractures created by the induced environment around the skin of the excavation, B the extent of barring used on the excavation, D is the drilling difficulty experienced along the tunnel and SP defines the existing support installed along the excavation to stabilize the sidewalls. (refer to Appendix A)

Table 2 represents the input values selected for the decline and calculated F value.

Tunnel Location	Slabbing	Cracks	Barring	Drilling	Support	F-Value	Adjust x 0.9
Decline	8	8	7	7	9	**0.78**	**0.7**
Average value determined for F							

Table 3 represents the described expected category of failure.

Category of Failure	Description of Conditions	F-Value	
		Excavations < 4 m x 4 m	Excavations > 4 m x 4 m < 6 m x 6 m
1	Very good conditions	1,0 – 0,96	0,90 – 0,84
2	Good conditions	0,81 – 0,95	0,73 – 0,83
3	Fair conditions	0,70 – 0,80	0,63 – 0,72
4	Poor conditions	0,59 – 0,69	0,53 – 0,62
5	Very poor conditions	0,50 – 0,58	0,45 – 0,52

Table 4 The following is a summary of the results obtained from the numerical modelling;

Level	Rock Formation	Depth below reef	UCS Rock strength MPa.	Stress levels - MPa		Increase in stress MPa	RCF values	
				Pre-mining	Current		Pre-mining	Current Peak
66-69	LUF2 - UF3	± 137 m	220	55	61	6	0.9	1
69-71	LUF2 - UF3	± 132 m	220	57	61	4	0.9	0.95
71-73	LUF3	± 180 m	220	59	61	2	0.99	1
73-74	LUF3	± 190 m	170	60	67	6	1.27	1.6
74-75	LUF2 – UUF3	± 143 m	220	61	64	3	1	1
>75	LUF2 – UUF3	± 149 m	220	62	66	4	1	1.1

Discussion of results

From the modelling results the following is evident;

- The calculated estimated maximum principal stresses and RCF were found to range between 64 and 70MPa and 1 and 1.2 respectively. The RCF value was based on the decline being situated in the competent LUF2 – UUF2 rock formations as determined in the earlier geological studies. It was expected that the larger portion of the decline would be overstoped with only a marginal influence from small fault loss pillars left intact.

- The intersection of the fault/dyke resulted in the decline being placed in the weaker LUF3. While the intersection of the structures did not result in a large change to the maximum principal stress, the weaker rock resulted in an increase in the expected RCF range to between 1.3 and 1.6 between 73 and 74 levels of the decline.

  The support designed for this area included the installation of primary support consisting of 2.3m grouted tendons followed by secondary support in the form of 4m grouted long anchors, additional grouted tendons and mesh and lace.

- Figures 4 and 5 show the unmined pillars that were created above the decline due to adverse geological conditions.

Figure 4 Pillar over the decline.

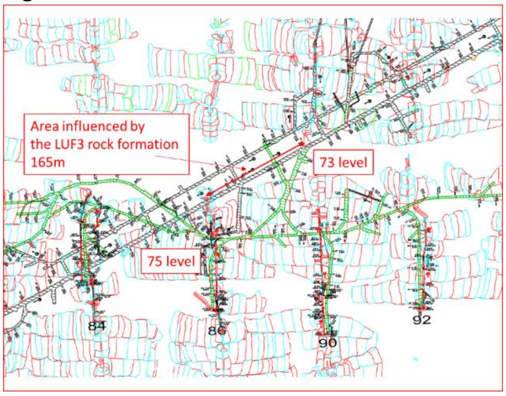
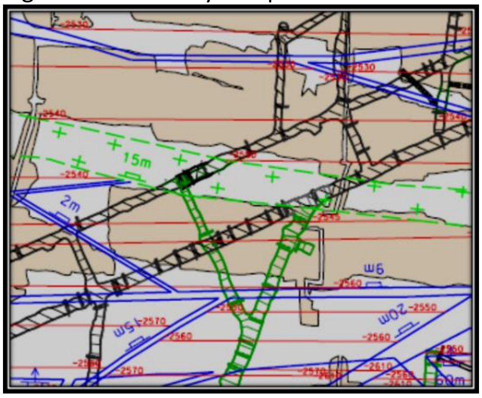


Figure 5 Dolerite dyke dips over the station and decline



Deformation encountered along the decline.

- A fall of ground resulted in additional support being installed in the decline below 73 level. The support included grouted tendons, mesh and lace, 4m grouted long anchors, 4-8mm of thin sprayed liner and 50mm of fibrecrete.

- The initial deformation was observed between the 71 and 73 level, as mining progressed over the decline excavations. It was recommended to install additional Rocprops in the middle, along the decline to ensure the stability of the decline excavation.

- Continuous deformation which was attributed to the orientation of the excavation to the strata dip combined with the weaker rockmass resulted in the ongoing rehabilitation of the decline which included:
    - Removal of the hangingwall mounted mono train from the decline hangingwall and suspending it onto footwall mounted RSJ sets.
    - The installation of RSJ sets from 69 level down to 73 level in the main decline.

- Deformation of the rock mass was evident on the RSJ sets that were installed in the decline. **Photo 4** The rate of deformation was only quantified in 2016 after sidewall closure pegs were installed. The rate of deformation was observed to be ±100 mm per year. The constant closure rate gradually increased over time which resulted in the RSJ sets deforming severely.

- Electronic extensometers were also installed along the decline in all the wide areas including station bull noses and chairlift landings to identify and record any rock deformation. Random FOG warning lights were also installed along the decline in areas where potential squeezing of the rock mass was expected.

- The extensometers at the 74-station bull nose recorded large deformations over 18 months as can be seen in graph 1.

Graph 1 Recorded movement 74 station

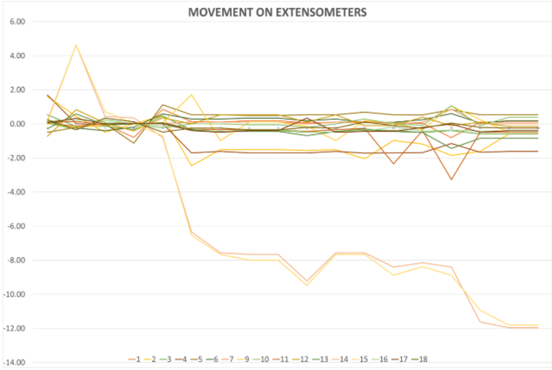


Photo 4 –cablelok slipping off the cable

- One of the concerns was the fact that some of the long anchor support units in the area of deformation showed signs of failure as the plates and cable lock barrel started to slip off the cables along the side and hanging wall. Photo 4. Sidewall fractures were measured to be at an average depth of 3.5 m which correlates with the sidewall primary support that showed no signs of failure or deformation as the units appear to be moving with the rock mass.

In conclusion, the above findings resulted in further investigation into the reason for the continued deformations being experienced to find a more permanent solution regarding the stability of the decline.

High Deformation – Squeezing formations

- The use of the RCF to determine the required support requirements did not provide for the continued deformation being experienced in the decline. Dr DF Malan and FRP Basson (Nov 1998) publish a paper that dealt with the squeezing effects of the rockmass on tunnels at Hartebeesfontein Gold Mine. The squeezing mechanism at Hartebeesfontein mine was considered to be a combination of time dependant failure of the intact rock and the sliding along the bedding planes which is similar to that being experienced in the Tshepong decline. Closure rates of up to 50 cm per month were recorded in some of the excavations which resulted in severe support difficulties.

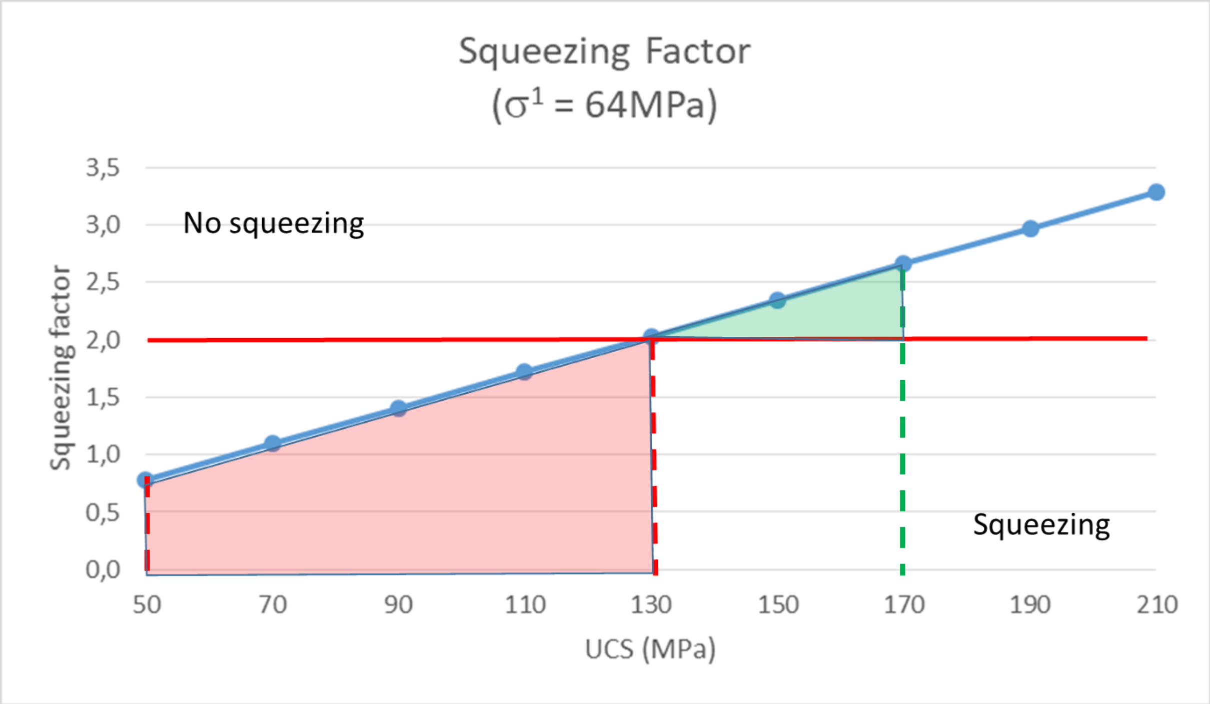
- Dr D.F. Malan and F.R.P Basson adapted the original squeezing formula $c = \sigma c/\sigma_v$ ***"Tunneling in squeezing ground, JC Chern March 1998"*** for shallow tunnels to be used in deep-level mines by substituting the σ_v with the Major Principal stress σ_1. Squeezing of the rock mass is expected where the $c < 2$.

$$c = \sigma c/\sigma_1$$

- The squeezing factor was calculated for the Tshepong decline to determine if the section of decline fell into the squeezing category and identify any other areas where potential squeezing may occur. The squeezing factor was calculated taking the following information into account:

UCS	LUF3:	: 110 – 280 MPa	
Point load strength	LUF3:	: 170 MPa	(Unaltered rock mass)
		: 50 MPa	(Altered rock mass)
Major Principal Stress	σ_1	: 67 MPa	

Graph 2 Squeezing graph C=2



- No deformation would have been anticipated assuming an average rock strength of 170 MPa. Graph 2 represents the rock strength vs the squeezing factor C=2. The mayor principle stress σ_1 of 64 MPa was also plotted on the graph, indicating when squeezing could be expected.

- It was observed that the LUF3 contained weaker zones. It was also noted that, when exposed to water and the atmosphere, the rock seemed to weaken substantially. Samples were tested using calibrated pointload tests and the UCS varied somewhat between 50 and 170MPa. See photos 5 and 6.

Photo 5 Bulging sidewalls

Photo 6 Weathered rock mass

Squeezing of the rock mass was not anticipated at the average rock strength of 170 MPa. It became evident that due to the varying rockmass strength of the LUF3, continuous squeezing effects can be expected under the existing circumstance. Contributing factors include the influence of water and weathering due to humid conditions.

Revised Support Design

- The conventional support that was installed was not able to stabilise the decline due to the continuous squeezing often resulting in delays and expensive rehabilitation. This was attributed to the tendons not being able to anchor the rockmass due to the extent of the fracture zone and where the longer tendons remained anchored they failed either by snapping in tension or the locking barrels and nut were pulled off. Photo 4.

- To accommodate the continued squeezing in areas identified the method of support selected was to create an artificial tunnel by progressively moiling the excavation to the desired dimensions and installing specially designed steel arch sets. Any cavities between the sets and the rock are filled with aerated cement fill which can act as a cushion between the arch sets and the rock.

- The steel square arch set was tested and has the following characteristics;

    - AC1Y / 3, the maximum deflection was at the crown. The deflection with a load of 230 kN on the crown and 230 kN on each bend, was 95 mm. The permanent deflection at this point was 53 mm, which results in a permanent deformation of 55.8%. The main result of this test is that this arch can bear a load of 700 kN on its crown without collapsing.

Photo 7 Square arch set, AC1Y / 3

- The main advantage of the square arch sets was that it would allow the decline to continue operating, as the rehabilitation team would only have to expose 1.5 m of the deformed rock mass at one time. The sets could then be progressively installed as the rehabilitation progressed forward.

- The second advantage is that the sets would provide adequate support which would restrict the deformation of the rock mass. The sets would allow the decline team to constantly monitor and where necessary adjust the arch sets where deformation or point loading was evident.

Rehabilitation of the Decline

The rehabilitation of the decline occurred in the following sequence;

The rehabilitation of the decline started in November 2017 and was eventually completed in 2021. Several logistical constraints hampered the process as the decline was required to remain operational during the rehabilitation. Photos 8 and 9 give a perspective of the before and after rehabilitation process.

Photo 8 Deformation of Northern sidewall

Photo 9 Initial sets completed

- The project was initially costed at R14m but the final cost was calculated to be R19m. Time constraints and material costs were the main contributors to the escalated cost.

Conclusion

- The deformations encountered were not anticipated in the initial modelling and support design. This was largely due to the limited knowledge of the geology and rockmass in which the decline was developed.

- Although the expected increase in stress on the decline due to pillars left in situ was minimal, the weak and incompetent LUF3 resulted in high deformations at relatively low stress. The use of an average UCS in a variable rockmass such as the LUF3 is inadequate. A detailed rockmass classification should be considered.

- Monitoring of the decline is done using electronic extensometer measurements weekly as well as survey peg measurements taken monthly. Mechanical extensometers were also installed in the sidewalls of the chairlift excavation and are measured monthly.

- Quarterly decline inspections are done by a Rock Engineer to identify any areas where closure or point loading is present on the sets. The identified areas along the set are reopened and rehabilitated by removing the squeezing rock mass from the sets.

Appendix A

PARAMETER DESCRIPTION	RATING
i) Slabbing (S)	
1. No visible slabbing (scaling)	10
2. Slabbing (scaling) visible in places	9
3. Slabbing (scaling) visible - barred down in places	8
4. Evidence of fallen slabs on footwall	7
5. Extensive falls of rock evident on footwall	5
ii) Cracks (C)	
1. No visible cracks	10
2. Cracks begin to appear in sidewalls	9
3. Cracks begin to open up in sidewalls	8
4. Open cracks - rocks fall out randomly	7
5. Rock fallout unaided (gravity)	5
iii) Barring (B)	
1. Sounds solid when struck by a pinch bar or hammer	10
2. Sounds hollow in places when struck by a pinch bar or hammer	9
3. Generally sounds hollow in places when struck by a pinch bar or hammer	8
4. Slabs tend to buckle and fall out when struck by a pinch bar or hammer	7
5. Slabs fall out unaided (gravity)	5
iv) Drilling (D)	
1. Drilling of holes is not problematical	10
2. Drilling of holes is not problematical	9
3. Drilling of holes becomes problematical	8
4. Drilling of holes difficult or impossible without prior dressing down of loose rock	7
5. Drilling of holes impossible without dressing down of loose rock	5
v) Support (SP)	
1. Support installed exceeds requirements for present conditions	10
2. Support installed is adequate for present conditions	9
3. Support assists moderately in stabilizing rock walls	8
4. Support has little effect on stabilizing rock walls	7
5. No support	5

R.C.F.	PRIMARY SUPPORT	SECONDARY SUPPORT
0,7< R.C.F.< 1	Static stress: - 2.3 m Videx lacing bolts as per mine standard 6.10.1 or 1.8 m Resin bolts, with welded mesh. Installed on a 3:2 pattern) Std 6.10.3 - Support resistance: 70 – 90kN/m².	Static stress: - Spot support where necessary. Light mesh where ground conditions are inherently poor.
	Stress changes: - Spot support where necessary (Resin bolts, etc.)	Stress changes: - Mesh with lacing ropes through the primary support where Videx bolts had been installed.
1 < R.C.F.<1.4	Static stress: - 2.3m Videx lacing bolts as per mine standard 6.10.1 or 1.8m Resin bolts, with welded mesh. Installed on a 3:2 pattern) Std 6.10.3 - Support resistance: 70 – 90kN/m².	Static stress: - - Install drill grout mesh and lace on a two meter pattern with 2.3 m Lacing roped where Videx lacing bolts were installed as primary support, - Install additional 4 m fully grouted mechanical anchors installed on a 2 m pattern where so recommended.
	Stress changes: - 2.3 m Videx lacing bolts as per mine standard 6.10.1 or 1.8 m Resin bolts, with welded mesh. Installed on a 3:2 pattern) Std 6.10.3 - Support resistance: 70 – 90 kN/m².	Stress changes: - Install drill grout mesh and lace on a double two meter pattern with 2.3m Lacing slings where Videx lacing bolts were installed as primary support, or over the perment mesh that was installed with 1.8 m resin bolts. - Additional 4 m fully grouted mechanical anchors installed on a 2 m pattern where so recommended.
R.C.F. > 1.4	Static stress: - Install 5.6 mm welded mesh secured with 2.3 m Resins bolts on a 1 m pattern. Std 6.10.3 - Support resistance: 110 – 130kN/m².	Static stress: - Install additional 4 m fully grouted mechanical anchors installed on a 2 m pattern - 8 mm thick Thin Line Spay.
	Stress changes: - Install 5.6 mm welded mesh with 2.3 m resin bolts up to 0.5 m from the face. Std 6.10.3 - Additional 4 m fully grouted mechanical anchors must be installed on a 3:3 pattern in the hanging wall and on a 2:1 pattern in the sidewall not further than 4 m from the face. - If recommended 8 – 16mm TSL must be applied 10 m from the face. - Support resistance: 110 – 130kN/m².	Stress changes: - The area must be visited and modelled by the Rock Engineer to issue support recommendations or to recommend an alternative layout.

Appendix B

REFERENCES

Tshepong North Shaft (2013). *Mandatory Code of Practice to Combat Rockfall and Rock burst Accidents.*

DF Malan & F Basson (1998). *Ultra-deep mining: The increased potential for squeezing conditions.*

JC Chen (1998). *Tunneling in squeezing ground*

K. R. Brentley (2001). *Quantification of support requirements for tunnels subjected to changing field stresses at Harmony Gold Mine.*