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Introduction of 3 m cable anchors as face support in 'timber-less stopes' at Dishaba Mine

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Dishaba Mine, part of the Anglo American Platinum Division, is situated in the Bushveld Igneous Complex some 100 km north of Rustenburg, South Africa. Platinum group metals are mineralized in tabular orebodies, which are conventionally mined in an underhand stoping configuration at a stoping height of 1.2 m to 1.8 m. Several years ago the mine personnel faced a twofold rock engineering challenge: large falls of ground in the face and back areas of mining panels, and a shortage of timber support in the market at that stage. The support being used at this time comprised cable anchors, mine timber support, and grout packs. The support had to be increased dramatically in an attempt to stabilize the face area and thereby stop falls of ground. Given the timber shortage prevalent at the time, a non-timber support had to be part of the solution. Long cable anchors with a high capacity replaced timber and short tendon anchors. The use of grout packs was continued to support back areas at the stage where cable anchors progressed past their yield capacity. The number of falls of ground in face areas and lost panels were dramatically reduced. This 'timber-less' support system has now been accepted and introduced into virtually all the Merensky stopes at Dishaba Mine. Phenomenal results were obtained after the introduction of the support system. Direct support costs were also slightly lower than the original anchor-timber system.

Introduction

Mine background and general information

Dishaba Mine, part of the Anglo American Platinum Division, is situated approximately 100 km north of Rustenburg, between the towns of Thabazimbi and Northam (see Figure 1). The mine is situated within the Bushveld Igneous Complex (BIC) where the tabular platinum group metal (PGM) reefs are being mined.

The two economically payable reefs that are being mined are the Merensky and UG2 reefs. The overlying Merensky reef, considered the primary reef, has been extracted before the UG2 and has subsequently advanced far ahead of the UG2 (see Figure 2).

The Merensky reef is comprised of a pegmatoidal feldspathic pyroxinite and harzburgite with an average uniaxial compressive stress (UCS) of approximately 130 MPa. The reef varies in thickness from zero to approximately two metres, and is terminated by a very prominent chromitite layer, which does not exceed 20 mm.

The immediate hanging wall of the Merensky Reef is poikilitic pyroxinite, with an average UCS of approximately 150 MPa, and varies in thickness from zero to four metres.

The footwall of the Merensky reef consists of an approximately six-metre thick layer of brittle poikilitic anorthosite with an average UCS of approximately 190 MPa.

Mining of the Merensky Reef makes use of an underhand conventional scattered breast mining layout (see Figure 3). Regular stability pillars left at intervals complement geological losses to ensure stability on a regional scale. Mining production panels are around 30 m long on dip. Local stability is achieved using strike crush pillars with dimensions of 3 m by 3 m that are cut at the top of each panel. Regular pillar-holings with dimensions of 3 m are cut between the pillars to allow for ventilation through the panels. These pillars were originally designed as intact pillars but at approximately 200 m depth they start yielding. Yielding and eventually crushing takes place closer to the face as the depth of the workings increases. This is desirable as the residual strength of the crush pillars is what is required to stabilize the panels

Prior to mining with 1.5 m cable anchors, panels were supported with a combination of timber support (pencil sticks), typically spaced 1.5 m × 1.8 m, 1 m long tendons installed after each blast, and grout packs to stabilize the back area. Support designs were done using the design charts of Bieniawski[1] to estimate the rock mass modulus from the rock mass rating. The design charts of Beer and Meek[2] were then used to estimate the height of hangingwall that had to be supported.

The panel length of 30 m provided panels that could be mined at reasonable productivity with reasonably practical support.

Project background

The frequency of large falls of ground (FOGs) increased progressively with mining at deeper levels. An intensive drive to obtain a good database of such FOGs, especially with their heights (thickness) determined, was launched. A reliable record of such FOGs was compiled towards the end

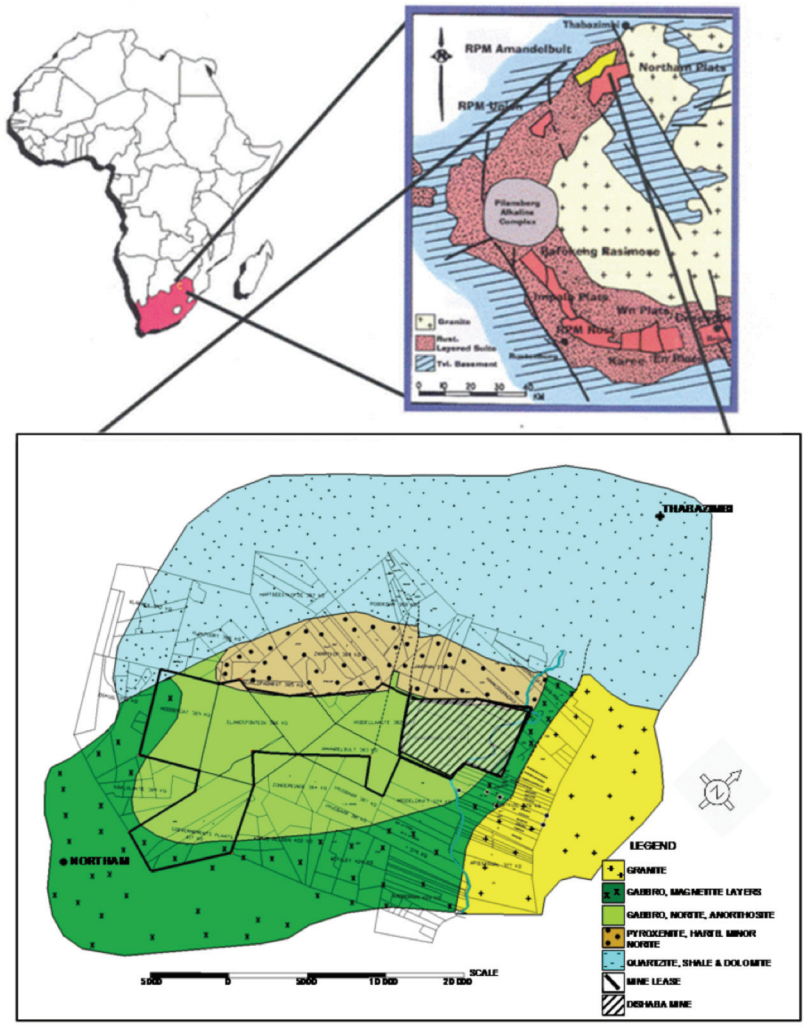

Figure 1. Dishaba locality map

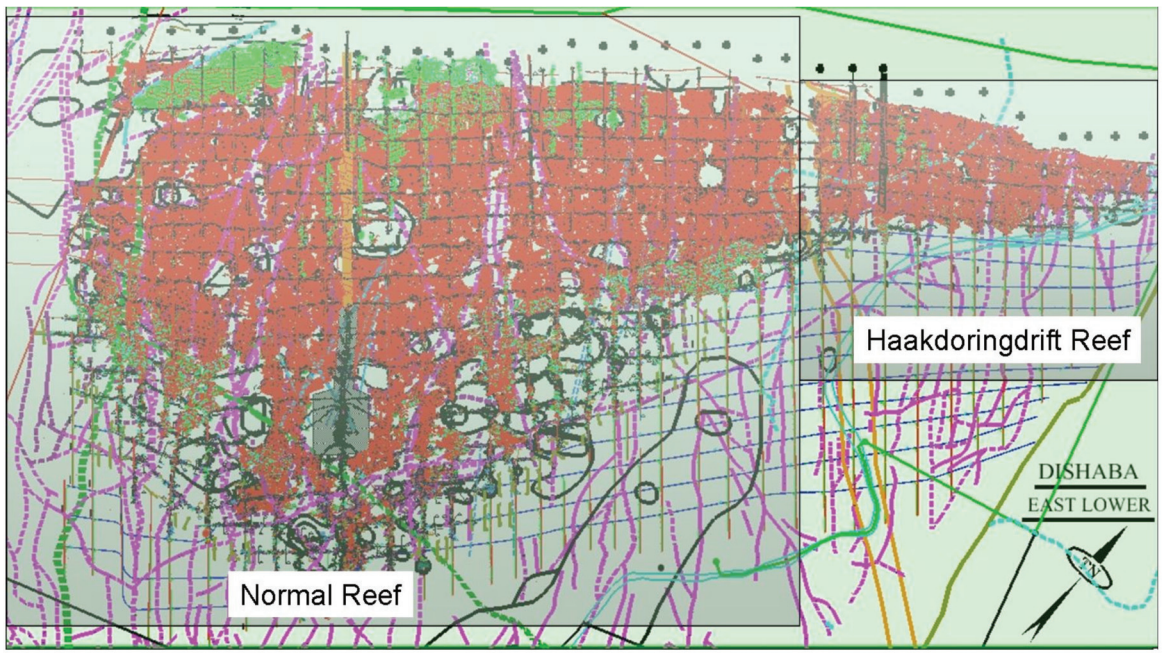

Figure 2. Extent of Merensky mining

of 2005. This database revealed that more than 95% of the falls were up to 3 m high. The rock engineers realized that a fallout height of up to 3 m had to be supported to stop uncontrolled FOGs from taking place. The support being utilized at that stage was 180 mm diameter, prestressed timber pencil props, spaced 1.5 m × 1.8 m apart (i.e. an average support resistance of 61.7 kN/m^2 when a factor of safety of 1.5 is applied). This is equivalent to a fallout height of approximately 2 m. At that stage 1 m long tendons were installed in the face area to stabilize the hangingwall between the last line of timber support and the face. RSS grout packs were utilized to stabilize the back areas. Neither the tendon nor the grout support types could be added to the support resistance because tendons were too short and the packs supported only the back areas. The problem presented in mining a flat dipping, tabular orebody is the low support resistance between the last line of timber support and the face. The historic fallout height in face areas was just less than 1.45 m. The 1 m long tendon support was not sufficient to stabilize the beam in the face area.

Subsequently the pencil stick spacing was reduced to 1 m × 1.8 m to increase the average support resistance to 93 kN/m^2, which is equivalent to a thickness of 3.2 m of hangingwall. The problem of maintaining a high support resistance in the face area, between the last line of pencil support and the face, remained. The 1 m long tendons in the face area were thereafter replaced with a 12.7 mm diameter 1.5 m long ('light weight') flexible cable anchor to support above the face area fallout height. This step, combined with re-raising panel faces when warranted, resulted in a large reduction in falls of grounds in face areas.

Grout packs were used to stabilize back areas as the closure in most back areas progressed to beyond the yielding capacity of the pencil sticks, i.e. between 150 and 200 mm. These also acted as breaker lines.

Towards the end of 2006 a critical shortage of mining timber was experienced countrywide and forecasts warned that the problem could worsen in 2007. This, together with the success that was achieved with lightweight flexible cable anchors, motivated the rock engineers to trial 'timber-less' panels. This support methodology was originally proposed for UG2 panels, due to the favourable lithology of the hangingwall.

In this support concept, the competent beam above is used to support the weak (hartzbertgite) layers below them, as shown in section in Figure 4. The 30 m beam, due to its width (thickness), is self-supporting between pillars. The previously used support design had a capacity to support only 1.5 m of hangingwall and would fail if the demand exceeded it. The timber-less project proved successful in the panels where it was applied, but was limited to a small number of Tumela UG2 panels (trial panels). Such a panel is shown in Figure 5. This support system was not readily accepted by production crews, most probably due the psychological effect of no visible support over a considerable area.

The rock engineers were then approached by the mine management to design a timber-less support system for Merensky stopes, i.e. in the same area where the fallout height was just less than 3 m and which was known for its extremely poor ground conditions.

Problem statement

'Design a stope support system for application in extremely poor ground occurring on Merensky stoping that addresses the 3 m fall out height requirements'. This paper focuses on the rock engineering investigations and designs that were carried out.

Project requirements

- Accumulate geotechnical information of the project area
- Confirm the height of the hangingwall that had to be

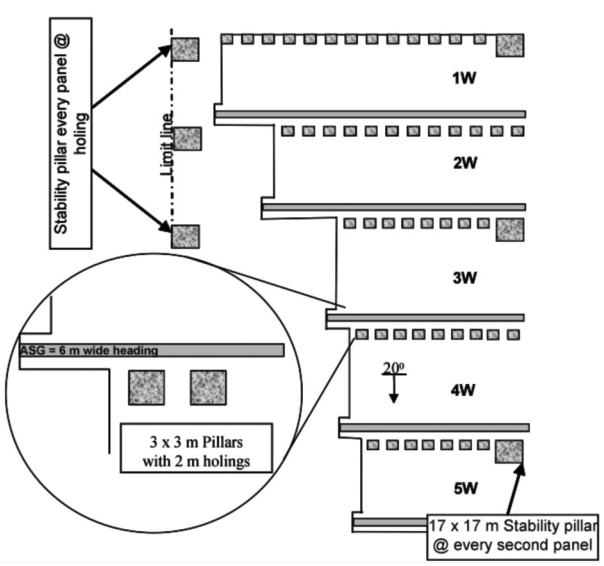

Figure 3. Typical Merensky breast mining layout at Dishaba

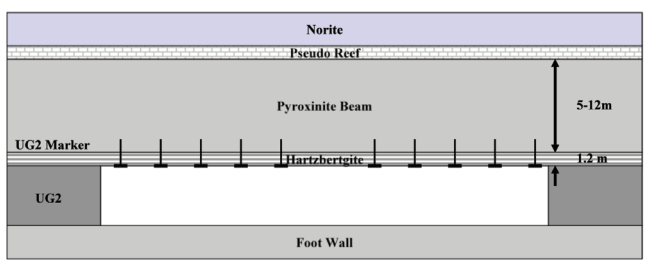

Figure 4. UG2 timber-less concept

Figure 5. A 'timber-less' UG2 panel

supported
- Calculate the required support resistance and compile a support standard
- Compile a support nomogram
- Evaluate the support with known joint orientations and the use of the J-Block numerical model
- Monitor and evaluate performance
- Scope the roll-out plan.

Methodology

The required support resistance was quantified. A support design was done. The support design was evaluated with the J-Block numerical model. The support effectiveness was monitored and evaluated in a small trial. A project was then compiled. This project included carrying out risk assessments, compiling installation procedures, establishing capital requirements, procurement of materials and equipment, and training of personnel. The support methodology was then rolled out to other sections.

Results

Geotechnical information of the hangingwall that had to be supported

The orientation of 'normal' joint sets in the Merensky reef areas of Dishaba 2 Shaft are shown in Figure 6. The principle joint set, 'J1', strikes sub-parallel to the north-south direction with the secondary set close to perpendicular to the first set. Both joint sets are steep, typically 90° ± 10°.

The joint sets, shown in Figure 6, are not the cause of falls of ground per se, but normally form one of the boundaries of such falls. However, alterations of both joint sets, such as a lower dip angle, opposite dip angles and infilling and water, often cause falls of ground as shown in Figure 7. The shown fall of ground originated from J1 combined with a low angled joint parallel to J1. The length of the fall is orientated parallel to the face and fell out after the last blast. The height of the fall of ground is approximately 2 m at its apex. Randomly orientated joints and domes are often intersected and also contribute to large falls of ground taking place.

Variations of J1 with a strike close to parallel to the strike gullies (ASG) of the east panels have caused large falls of ground striking sub-parallel to these gullies and often result in total abandonment of the gully and panel.

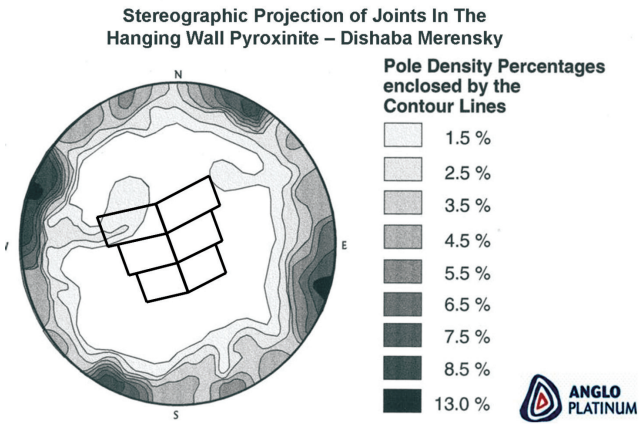

Figure 6. 'Normal' joint sets at Dishaba No. 2 shaft

In addition to the above, a combination of domed or flat dipping structures with J1 or J2 is often intersected resulting in further falls of ground. (Figure 7.)

Logging of geotechnical hangingwall bore holes of Merensky stopes (Figure 8) and exposed sections of the hangingwall of the Merensky Reef (Figure 9) indicated that the rock mass rating (Bieniawski[1] RMR) of the stope hangingwall varied between a lower Class IV to Class V conditions.

Figure 9 is a combination of two slides that were taken in off-reef gullies, facing the down-dip direction. The slide on the left hand side represents a section through a stope hangingwall and the one on the right shows the hangingwall above a re-raise to a depth of more than 3 m. It is obvious from both these slides that a combination of flat dipping

Figure 7. View down-dip of a fall of ground from an altered J1

Figure 8. Core from a geotechnical hole that was drilled into the hangingwall of a Merensky stope

Figure 9. Sections into the hangingwall of the Merensky Reef

and steep dipping joints are present in the Merensky Reef hangingwall, that, in many areas, could be described as a broken-up mass of rock.

What was also apparent in the left-hand picture of Figure 9 was the amount of dilation (opening of joints) that took place inside the hangingwall of the reef and the thick serpentinized infill that is present in some joints.

Panel and support design

As previously mentioned, panel spans are kept at 30 m due to the more favourable productivity that can be achieved when compared to shorter spans.

To use the Beer and Meek[2] Voussoir arch graphs to estimate the support demand, the 'rock mass modulus' (RMM) had to be estimated. The Bieniawsky[1] relationship, as shown in Figure 10, was used for this purpose. Rock with an RMR between 10 and 30 will have an RMM between 2 and 5 GPa. These values were used to estimate the support demand using a similar relationship with the NT Beam program as the relationship established by Beer and Meek[2] as shown in Figure 11. This figure indicates that the hangingwall beam thickness that had to be supported was approximately 3 m, i.e. 90 kN/m^2. This figure correlated well with the demand established with the cumulative fall of ground frequency graph shown in Figure 12.

Tendon support therefore had to penetrate the hangingwall to a depth of 3 m and have a load bearing capacity equivalent to 3 m of hangingwall, i.e. 90 kN/m^2 as well as a factor of safety of 1.5.

Confirmation of the height of hangingwall that had to be supported

The FOG database that was compiled during 2005, shown in Figure 12, was used for this exercise. This database was accumulated before adjustments to the support standard were made and is therefore a good indication of the problem that had to be solved. This database also confirmed that a hangingwall height of approximately 3 m had to be supported.

Support resistance calculations

The chosen tendon support units were 15 mm diameter (250 kN ~ 25 tons) × 3 m long cable anchors, prestressed to 100 kN (~10 tons). Holes for these units had to be drilled to a depth of at least 3.1 m to ensure that an effective support depth of 3 m could be achieved. The length of the shell was ignored and effective support depth was considered to be up to the position of the shell. This was considered as adequate as none of the measurements and dimensions were precise. (e.g. the fall out height of the 'cumulative frequency graph' and dimensions obtained from the 'Voussoir arch graph').

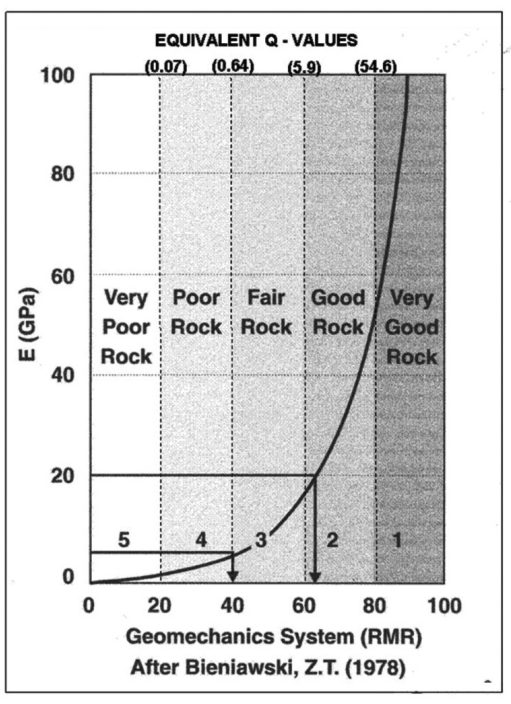

Figure 10. Estimation of the RMM using RMR values

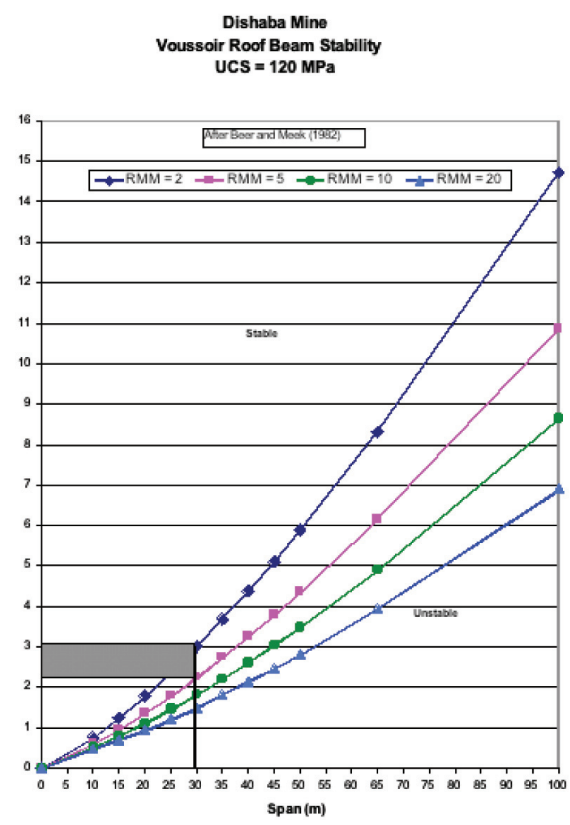

Figure 11. Voussoir roof beam stability

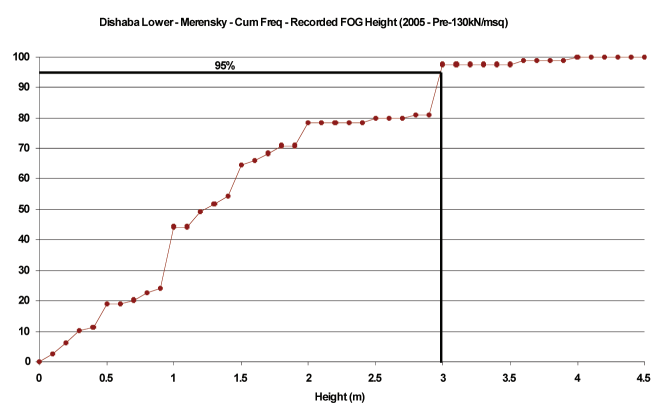

Figure 12. Cumulative frequency graph of 2005 fall of ground database

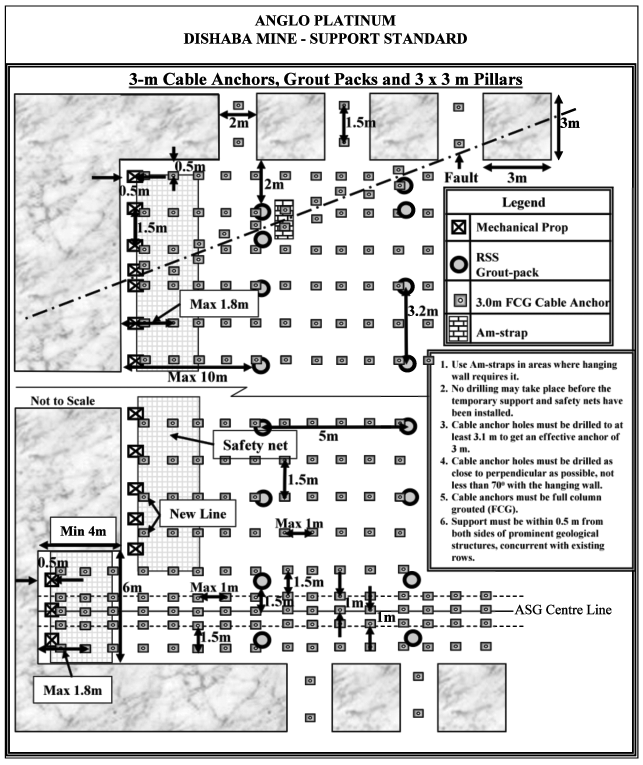

Figure 13. 3 m Cable anchor support layout

The capacity of the chosen support is 90 kN/m² if the length of the tendons is kept at 3 m, i.e. 3 m of hangingwall height due to the depth of penetration.

Tendon support is most useful when it is installed close to the face after each blast. This also provides support for crews during all their activities, e.g. night shift crews.

The average advance for Merensky panels at Dishaba Mine was 0.9 m. The dip spacing of the support was kept at 1.5 m, the same as for all other previously used tendons due to this being an adequate frequency for the prevailing joint spacing. It also ensures an adequate overlap of the zone of influence as shown in Figure 28. The factor of safety for this support is 2 for a strike spacing of 0.9 m. The stipulated maximum strike spacing is 1 m, which would still give a FOS of 1.9. This value drops to 1.5, if the support capacity is downrated by 20% to 200 kN for possible poor installations, which is quite acceptable for any type of support. The support layout is schematically shown in Figure 13.

Grout pack support was left unchanged for back areas except for its support to face distance, which was brought closer (10 m) to ensure that packs are installed closer to the face.

Evaluation of the support with known joint orientations

The properties of joints that were mapped underground in panels and were used to do J-Block runs to test the probability of failure. J-Block is a computer aided program

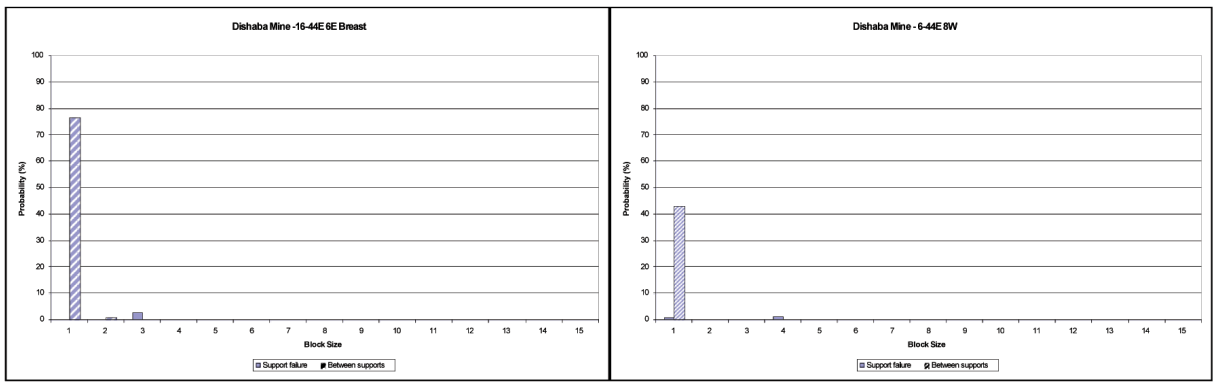

Figure 14. Probability of failure for 3 m anchors in 16-44E 6E and 8W

Table I
Data used for modelling 16-44E 6E

DATA :

Joint set	Dip	Dip Dir	Dip range	Ave Spacing	Min spacing	Max spacing	Ave Length	Min Length	Max Length
1	60.00	90.00	20.00	10.00	9.00	11.00	100.00	95.00	105.00
2	20.00	90.00	10.00	1.00	0.90	1.10	20.00	19.00	21.00
3	75.00	210.00	10.00	3.00	2.90	3.10	30.00	29.00	31.00
4	75.00	30.00	10.00	3.00	2.90	3.10	30.00	29.00	31.00
5	89.00	10.00	10.00	2.00	1.90	2.10	30.00	29.00	31.00
6	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
Excavation	20.00	120.00	1.00						

Support	Force (kN)	Length (m)	Width (m)	Angle (deg)
Point	200.00			
Line	200.00	1.00	-	30.00
Area	500.00	1.20	1.20	
Tendon	250.00	3.00		

Face direction	Face length m	Back length (m)
160.00	30.00	30.00

Rock Density : 3000.00 Seismic accelleration (g) : 0.00 (g)
Total area modelled = 2355895.00 m² Keyblocks generated = 1000
Percent tendons too short = 15.32%

Table II
Data used for modelling 16-44E 8W

DATA :

Joint set	Dip	Dip Dir	Dip range	Ave Spacing	Min spacing	Max spacing	Ave Length	Min Length	Max Length
1	60.00	90.00	20.00	2.00	1.90	2.10	60.00	59.00	61.00
2	55.00	300.00	20.00	2.00	1.90	2.10	60.00	59.00	61.00
3	50.00	120.00	20.00	6.00	5.90	6.10	60.00	59.00	61.00
4	60.00	120.00	20.00	10.00	9.50	10.50	100.00	99.00	101.00
5	20.00	70.00	5.00	4.00	3.90	4.10	10.00	9.50	10.50
6	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
Excavation	20.00	150.00	1.00						

Support	Force (kN)	Length (m)	Width (m)	Angle (deg)
Point	200.00			
Line	200.00	1.00	-	30.00
Area	500.00	1.20	1.20	
Tendon	250.00	3.00		

Face direction	Face length m	Back length (m)
20.00	30.00	30.00

Rock Density : 3000.00 Seismic accelleration (g) : 0.00 (g)
Total area modelled = 15634700.00 m² Keyblocks generated = 2338
Percent tendons too short = 10.53%

that uses joint information to generate possible wedges and evaluate the probability that these wedges will either fail the installed support or fall out between support units. Results from runs, as shown for a specific area in Figure 14, indicated a very low probability of failure.

A summary of the joints that were used in the J-Block analyses is shown in Tables I and II.

Monitor and evaluate performance from underground observations

A panel that has been supported with 3 m cable anchors and grout packs is shown in Figure 15.

Closure measurements

Previous measurements and observations of stope closures where timber support (pencil sticks) and short anchors were used for support indicated that a maximum closure of between 150 to 200 mm could be expected by the time that a raise line was completely mined out. This was the reason for the introduction of grout packs in the back areas of stopes a few years ago.

The maximum closure that was roughly measured in stopes where 3 m cable anchor support was introduced was approximately 10 mm. This indicated that closure that took place with 'conventional support' was mainly caused by separation of the joints in the hangingwall causing the hangingwall to dilate, i.e. mostly inelastic closure. This dilation was inhibited by the cable anchors.

Incident investigations

Since the adoption of the 'timber-less' mining support, two isolated incidents where total closure was greater than 10 mm were investigated. These will be discussed under the headings 'Case 1' and 'Case 2'. A large fall of ground that took place in a reasonably deep UG2 stope, which was also converted to 3 m cable anchor support, will be discussed as Case 3.

Case 1: This panel was part of one of the first stopes that was converted to 3 m cable anchor support. In this instance it was noted that the cable anchors were not properly tensioned over a large distance. This caused the wedges that are supposed to resist 'slip' over the tendon to slip as they have not been 'seated'. Closure in this stope was approximately 100 mm at the time of the investigation. It was obvious that the area was mostly being supported by the back area packs that were installed. A large fall of ground subsequently took in the face area of the panel. It is evident in Figure 16 that the load indicator did not extrude

Figure 15. Panel supported with 3 m cable anchors and grout packs

Figure 16. Case 1 FOG showing untensioned anchors

and that the anchor was not tensioned upon installation as shown in Figure 16.

The dimensions and orientation of this block was subsequently modeled with J-Block single block analyses. This analyses indicated that the block would not have fallen if the support were correctly installed and to standard as shown in Figure 17.

Case 2: Closure in this stope was approximately 100 mm at the time of the inspection. In this case virtually all the cable anchors were correctly installed. Evidence indicated that the hangingwall was moving as a single unit over a very large area (across a number of panels). In this case it could be deduced that the movement originated in a position above the anchoring points of the cables. The demand on the support also exceeded the capacity of the grout packs as closure continued to take place at a slow rate although mining in this stope has stopped. The total mined-out span in this case is also small (less than 60 m). Closure is therefore being driven by an 'unconventional' instability. Further work is required to understand the nature and cause of this regional instability.

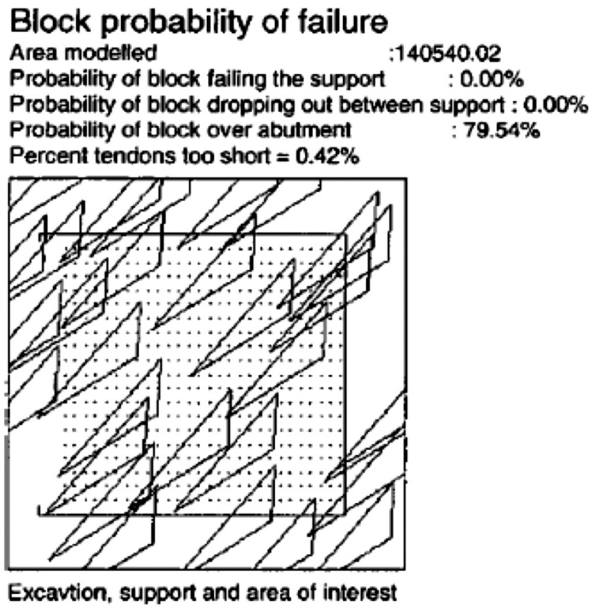

Figure 17. J-Block results for Case 1

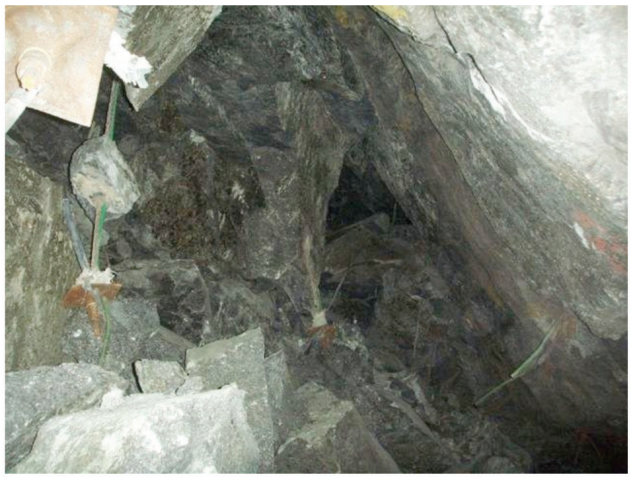

Figure 18. Case 3 FOG from UG2 hanging wall

Case 3: A large fall of ground occurred from the hanging wall of a UG2 stope as shown in Figure 18.

The fall of ground took place from the hangingwall of a UG2 stope. The UG2 Reef was not intended for this report but this fall of ground was included for completeness. In this case one of the boundaries was formed by a fault. The hangingwall adjacent to the fault was very serpentinized and blocky.

The J-Block single block analyses (Figure 19) again showed that if the support were installed correctly, the block would not have fallen.

Evidence from the underground investigation showed that the support was installed to a good quality in the area surrounding the fall. One can therefore assume that the support within the area of the fall was also installed to a good standard. The fall of ground therefore took place as a result of the extremely broken and serpentinized nature of the hangingwall. In this case the area adjacent to the fault (on its weak side) should have been identified and left as pillar.

Investigations of other smaller falls of ground that took place showed:

- That 3 m cable anchor support had to be correctly installed for it to be fully functional.
- Non-tensioning and installed at low angles were the main contributors to falls of ground with this support methodology.

The following have been implemented to prevent re-occurrences of FOGs of the above-mentioned nature:

- Support units are supplied with load and depth indicators that shows installers and auditors when support units have not been installed to the correct quality
- Support installers and supervisors are trained with an outcomes-based training syllabus on how to ensure that a good quality of support installation is achieved
- Safety-auditing personnel have been trained and made aware of the indicators and importance of the proper installation of these support units. Panels are immediately stopped if any substandard units are found until they have been replaced

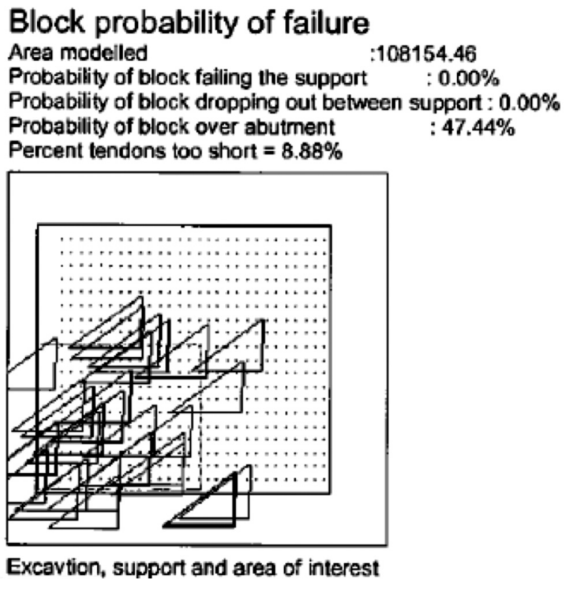

Figure 19. J-Block results for Case 3

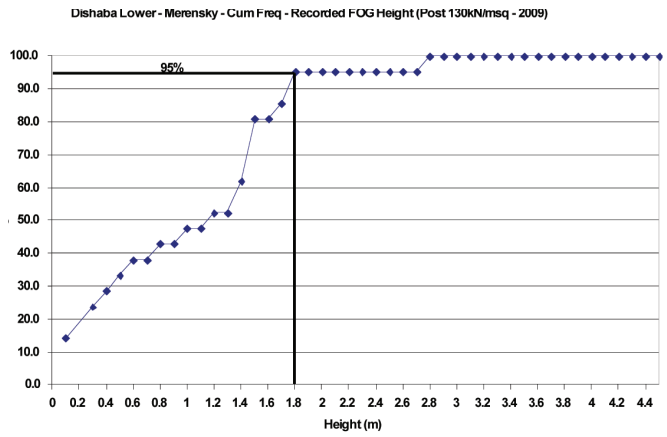

Figure 20. Cumulative frequency graph after increasing the support resistance to 130 kN/m² and introduction of 3 m cable anchors

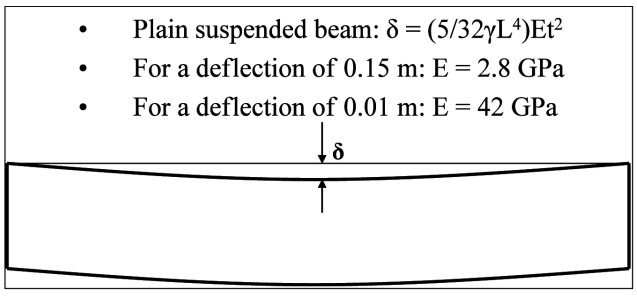

Figure 21. Equations for a plain suspended beam

- Ad hoc audits by safety and rock engineering personnel are conducted to ensure that the support units are installed to the required standards. Panels are stopped if any sub-standard units are found until they are replaced.

Falls of ground database after implementation of cable anchors

It is obvious from Figure 20 that the sizes of falls of grounds have been greatly reduced after upgrading the support resistance to more than 130 kN/m² and the subsequent introduction of 3 m cable anchors.

Quantification of cable support action

The large reduction in stope closure after the introduction of 3 m cable anchor support was not expected. This can only be explained with 'beam equations', i.e. deflection of a plain suspended beam as shown in Figure 21. If one considers a 3 m beam with a length of 30 m with a total deflection of 0.15 m (as was observed underground with pencil sticks that also had the capacity to support 3 m of hangingwall) then the modulus of the beam is back calculated as 2.8 GPa, which correlates well with the Bieniawski[1] and Beer and Meek[2] graphs discussed earlier. Similarly, a beam deflecting only 0.01 m has a modulus of 42 GPa. One can therefore conclude that the cable anchor support reinforces the beam to such an extent that the modulus is theoretically increased to 42 GPa. A competent beam is thus created that is stable at the spans between face, pack support, and pillar abutments underground.

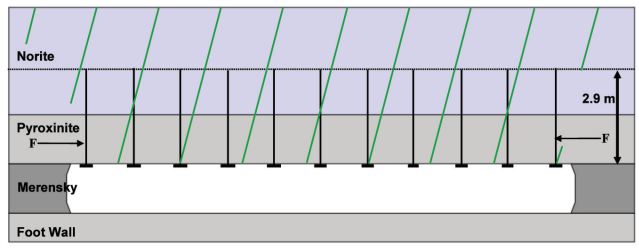

Figure 22. 3 m cable anchors with 750 joints in a 20 m panel

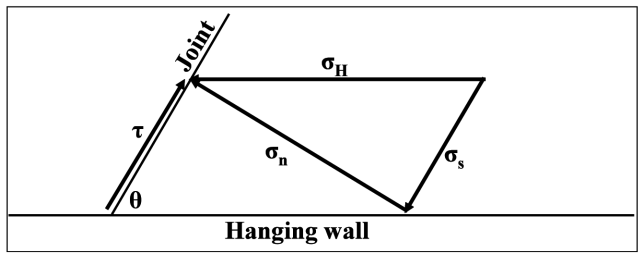

Figure 23. Vector components of horizontal stress influence on joint

Table III
Values of for various values of to ensure stability (the effects of cohesion and gravity excluded)

$\phi°$	Lowest value of $\theta°$ for stability
10°	80°
20°	70°
30°	60°
40°	50°
45°	45°

To further understand the 'mode of support' of 3 m cable anchors, a series of sketches depicting a section through a stope panel with various joint angles was created and is shown in Figures 22 to 27.

Every third joint is missed in the case of 750 joints as shown in Figure 22. These joints are therefore reliant on their cohesions and the horizontal stress in the hangingwall to keep them stable. To investigate this, one needs to consider the stress components on a joint as shown in Figure 23.

From Figure 23 consider the following: $\tau = \sigma_n \tan \varphi + c$. For stability $\tau = \sigma s$. If the influence of cohesion and gravity is excluded then $\tan \varphi = 1/\tan \theta$. For various values of the friction angle the stable joint angle is shown in Table III.

From the values in Table III it is apparent that if a joint has a low cohesion and is filled with thick enough material (say 3 mm) with a low friction angle, such as serpentinite combined with water, the stability of the hangingwall will become more dependent on cable anchors that penetrate joints as the effect of the horizontal stress is minimized.

From Figures 24 and 25 it is apparent that lower angled joints will be more readily penetrated by cable anchors.

The support effect of cable anchors on joints dipping in opposite directions is the same as previously discussed as shown in Figures 26 and 27.

The zone of influence of 3 m cable anchors installed at 1.5 m spacings is quite considerable, as shown in Figure 28. This is a simplified model as the zone will reduce slightly

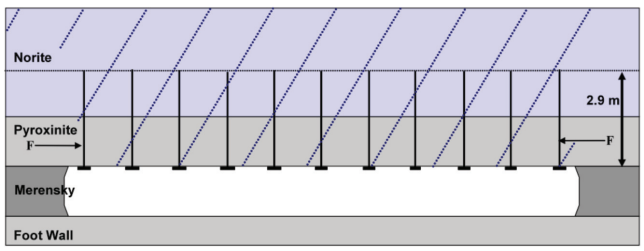

Figure 24. 3 m cable anchors with 60° joints in a 20 m panel

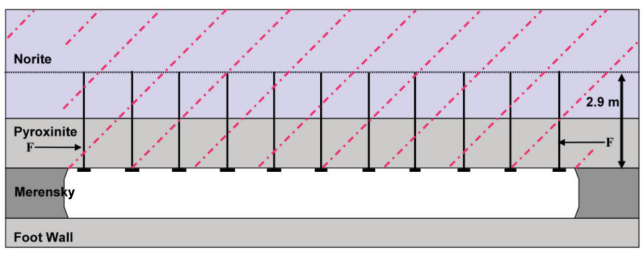

Figure 25. 3 m cable anchors with 45° joints in a 20 m panel

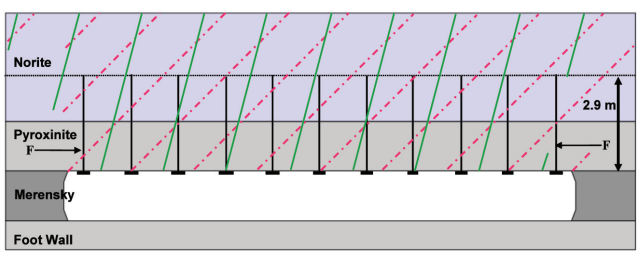

Figure 26. 3 m cable anchors with 45° and 75° joints in a 20 m panel

of the cost of a temporary or permanently lost panel.

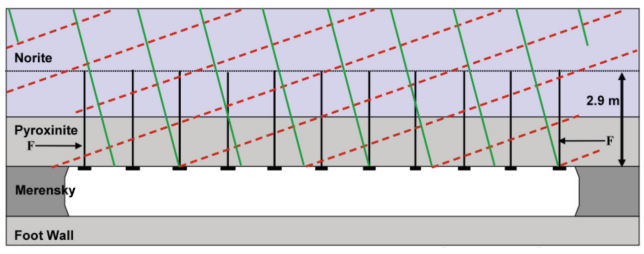

Figure 27. 3 m cable anchors with 45° and 75° joints in a 20 m panel dipping in opposite directions

Conclusions and recommendations

- 3 m, 250 kN cable anchors, prestressed to 100 kN are suited as sole support medium in a rock mass that has been severely weakened by joints.
- It is extremely important to install cable anchors to a good quality and standard otherwise they become ineffective.
- It is very important to ensure that cable anchors penetrate joints as the effect of horizontal clamping by the horizontal stress in the immediate hangingwall is minimal if the values of the friction angle and cohesion are low. These conditions are often found at Dishaba where joints have a wet serpentinized filling of more than 3 mm.
- It is very important to have staff trained to identify areas where tendon support will be insufficient to support the hangingwall and pillars have to be left.
- Further studies are necessary to determine the optimum cable anchor spacing for different joint spacings.
- The hangingwall above the 3 m zone of influence of the cable anchors must be inspected to determine the extent of joint separation.
- Extensometers must be installed to depth of 6 m to determine what separation is still occurring in the hangingwall above the anchors.

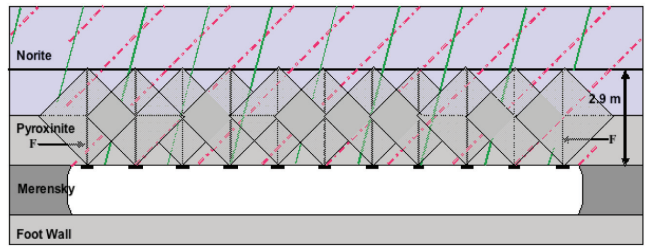

Figure 28. The zone of influence of 3 m cable anchors installed at 1.5 m spacings in a rock mass with an internal angle of friction of 45°

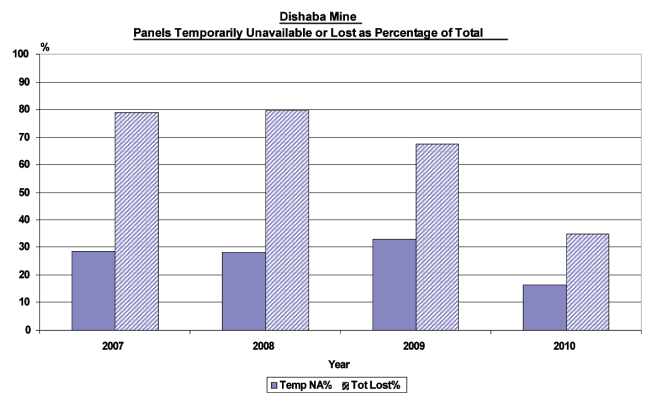

Figure 29. Panel 'unavailability' compared to total number of panels

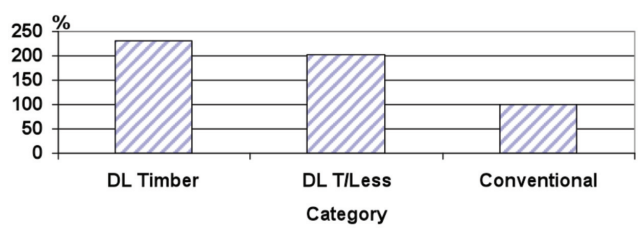

Figure 30. Support cost comparison (proportional)

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My mining career started in 1985 as Learner Official in Anglo American. I completed my mining degree in 1990, changed from production to Rock Engineering in 1994 in the Welkom goldfields area. I moved to Platinum mining in 1997 at Amandelbult which was at that stage owned by JCI but later taken over by Anglo American Platinum. Amandelbult was divided into two mines in 2009 of which one is Dishaba Mine where I am currently positioned as the Chief Rock Engineer.