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Alternative support systems for mechanized stopes

D.P. ROBERTS*, M.K.C. ROBERTS*, and A.J. JAGER*
**CSIR Division of Mining Technology*

The stoping of platinum orebodies in the Bushveld has in the past been accomplished by using in-stope pillars to ensure stability of the workings. In the future, the use of continuous mining machines to mine the platinum orebodies is probable. It is not envisaged that such machines will cut in-stope pillars. This paper describes the alternative support systems that will replace the pillars and the stope geometries that will be required for this type of mining. The paper describes how the support resistance provided by in-stope pillars in current mining was determined and then evaluates support systems that can substitute for these pillars. Conceptual layouts based on existing breast mining layouts are presented. The layouts consist of panels of up to 100 m face length between rigid strike stabilizing pillars. A number of design charts are then proposed that allow for the design of appropriate support systems.

Introduction

The stoping of platinum orebodies in the Bushveld has to a large extent been accomplished by using in-stope pillars to ensure stability of the workings. These pillars ensure that sufficient support resistance is applied to the hangingwall to preclude the tensile failure of the hangingwall known as a backbreak. In the future, the use of continuous mining machines to mine the platinum orebodies is possible. Such mining would require that a machine would mine in an uninterrupted path along a stoping face, which could be up to 100 m in length. This scenario does not allow that the cutting of in-stope pillars and alternative support systems would be required to substitute for the in-stope pillars. The objective of this paper is to identify such alternative support systems and to conceptualize mining geometries that would be compatible with a mining machine and associated stoping operations. In order to do this, the following research was undertaken:

- The determination of the support resistance requirement to ensure stability in Merensky and UG2 stopes in current crush pillar stopes
- The determination of methods of achieving the required support resistance, without pillars, for various lengths of face to a maximum of 100 metres
- Conceptualization of the type of mining layouts that will allow long faces to be mined without pillars.

The determination of the required support resistance for stopes

A number of pillar systems are currently used in the Bushveld platinum stopes and they can conveniently be classified into two types. The first are intact pillars. These are pillars that have not yet reached the stress state that would result in failure. The second are crush pillars, which are pillars that have been designed to fail shortly after being cut and after failure have a residual strength. Both these pillar systems are effective at preventing backbreak with the implication that they both provide at least the required support resistance to the stope hangingwall to prevent backbreak. The support resistance generated by an in-stope crush pillar system will be lower than that generated by an in-stope intact pillar system. If this support resistance could be determined then it could be used as a requirement for the support system replacing these pillars. In order to do this, the residual strength of crush pillars needed to be determined.

The determination of the residual strength of a crush pillar

Crush pillars are commonly cut so that the width to height ratio of these pillars is 2:1, allowing failure in a stable manner after about 4 millistrains of stope closure. They can be oriented on strike or dip and make up about 5–7% of the orebody being mined.

Back analysis of data from Randfontein Estates Gold Mine

The observations at Randfontein Estates Gold Mine were made in the late 1970s and early 1980s when stoping was taking place without crush pillars[1]. The stopes were supported with grout-pack systems, which provided insufficient support resistance to prevent backbreak. Extensometers and the monitoring of hangingwall excavations showed that the hangingwall failure extended up to 40 m into the hangingwall. In 1980 crush pillars were cut in all stopes. Closure stations and extensometers showed no inelastic movement in the hangingwall strata once the pillars were introduced. As the height of the hangingwall that collapsed during backbreaks was known, it was possible to calculate the minimum residual strength of the crush pillars[2]. This was determined to be 13 MPa. Considering the percentage of pillars compared to the percentage of mining allows the determination of the support resistance provided by the pillar support system. This was determined to be 1.1 MPa.

Back analysis of data from Northam Platinum Mine

The work at Northam is described by Roberts[3]. The intention of the work was to determine if the stope hangingwall was still prone to backbreak failure at a depth of 1400 m. Conventional pack and elongate support was used to support the stope. Instrumentation such as closure metres and extensometers were installed and the hangingwall was monitored with increasing mining spans. The extensometers showed substantial dilations between layers up to 28 m in the hangingwall. The magnitudes of these dilations could not be accounted for by the elastic response alone. The dilation and some of the closure component were therefore inelastic, which indicated that the hangingwall was becoming unstable and that collapse was inevitable with increasing mining spans. For this reason, all panels were then backfilled and stoping continued. Stress measurements in the backfill were then undertaken as the mining spans increased further.

The monitoring exercise resulted in the determination of the support resistance required to stabilize the stope. This can be determined by considering the dead weight load of the hangingwall up to the highest horizontal discontinuity along which separation may occur. This horizon coincides with the Bastard Reef at around 30 m into the hangingwall. Assuming a rockmass density of 3300 kg/m^3, this gives a support resistance requirement of approximately 1 MPa Further, the measurements in the backfill indicate a similar support resistance. When the backfill was installed, the extensometers in the hangingwall continued to dilate until the backfill had strained to about 12% and reached a vertical stress of about 1.1 MPa; see Figure 1. At this point hangingwall deformation became elastic and the hangingwall stabilized. This indicates that a support resistance of 1.1 MPa is sufficient to stabilize the stope hangingwall.

Stress measurements at RPM Frank 2 shaft

The direct determination of the residual strength of crush pillars has long been an objective of rock engineers. Due to the difficulty of measuring the strength of the failed material constituting the pillars, this has not been previously achieved. A novel approach was therefore attempted. This was to measure the stress in the intact rock above and below the crush pillars. The stress measurements were therefore attempted under the pillar by drilling into the gully sidewall and into the hangingwall above the pillar, see Figure 2. These measurements would translate into the residual pillar strength.

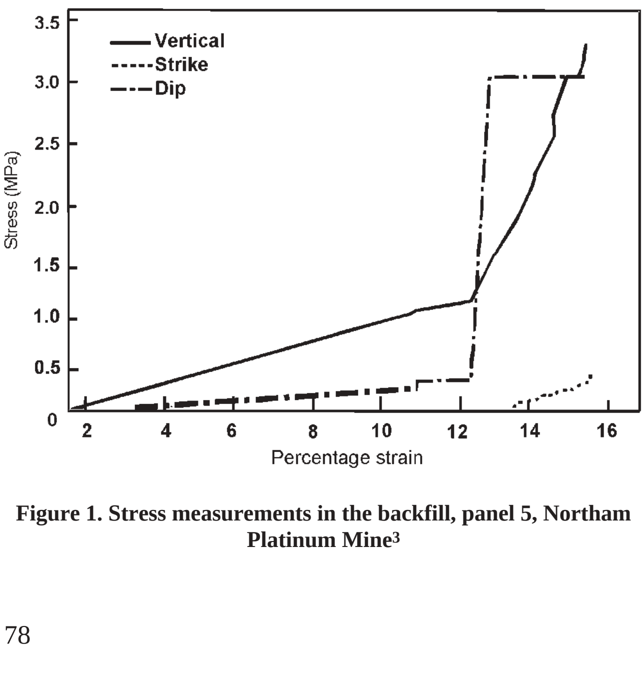

Figure 1. Stress measurements in the backfill, panel 5, Northam Platinum Mine[3]

All measurements were undertaken where pillars were more than 10 m from the face in order to be sure that the pillars had failed and that the stress measurements would reflect the residual strength of the pillars. Stope plans (Figure 3) indicated no nearby abutments or stabilizing pillars. The two pillars from which reliable readings were obtained are indicated.

Measurements were conducted at two sites where doorstopper stress cells were installed from the adjacent strike gully. Three doorstoppers were installed in each borehole. The reliability of the readings was checked by examining the degree of fracturing within the borehole cores. The experimental error was determined by calculating the difference between stress components, which should theoretically be equal.

The results from the stress measurements show a vertical stress that varies between 7.5 MPa and 25 MPa. A weighted average was obtained by adjusting the contribution of each measured value based on the error calculated for that value. Using this method, a weighted average of 18.9 MPa is

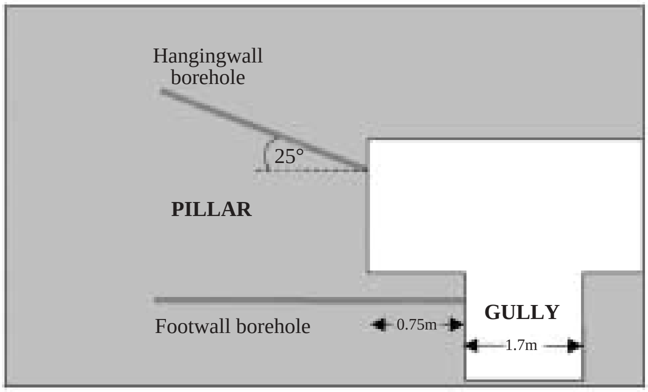

Figure 2. Position of the stress measurements relative to the stoping excavation

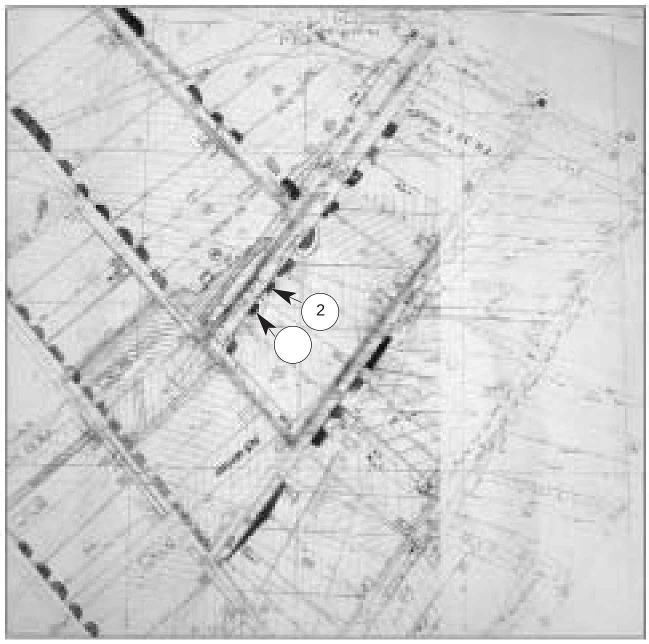

Figure 3. Stope plan of the area where measurements were conducted

obtained. The percentage extraction can be calculated from the mining plan and was determined to be 96%. The support resistance over the entire stope can then be calculated and is 0.76 MPa.

Consolidation of the three sets of data

Three different sources of data have given indications of the required support resistance that would be required to prevent backbreak in stopes. The support resistance of 1.1 MPa is estimated from the Randfontein Estates data. The Northam data indicates that a support resistance of between 1 and 1.1 MPa is required to stabilize the stopes. The ambitious stress measurement programme at Frank 2 shaft also proved successful and indicated that the residual pillar strengths were around 19 MPa, which translates to a total support resistance of 0.76 MPa.

This evidence indicates that a support resistance of 1.1 MPa would be a reasonable value for the purposes of designing support systems that would prevent hangingwall backbreak in Bushveld platinum stopes. For most Bushveld conditions, this represents an upper limit of the required support resistance. Under certain circumstances it could be envisaged that a lower support resistance could still stabilize the stoping excavations; however, the lower support resistance limit is not known. This upper limit translates to a support resistance of 1100 kN/m^2 or 110 $tons/m^2$. Conventional support units, with the possible exception of robust grout pack support systems, are simply incapable of generating this kind of support resistance at practical and economically viable support densities.

Methods of achieving the required support resistance without pillars

As stated above, hangingwall spans of up to 100 m need to be supported and stabilized. Extensive numerical modelling has been used to determine the stability of such a span and what support systems are appropriate.[4] The support options under consideration are standard quality hydraulic backfill or grout packs, where appropriate. Both of these systems have the advantage that the support material is transported via pipes into the stopes. Modelling was undertaken using the ELFEN finite/discrete element suite of programs. All models were conducted in 2D plane strain. Large-span models were generated with hangingwalls made up of discontinuous elastic blocks, representing typical generic platinum hangingwalls. The models were calibrated with simple sliding block models to ensure that the dynamic parameters in the system provided accurate results. The displacement histories of blocks at centre-span were used as a criterion for hangingwall stability. Different models were constructed to evaluate the performance of various backfill types and to establish the limits of grout packs.

Backfill

Backfill is best suited to provide the 1.1 MPa support resistance requirement before excessive closure has occurred. The backfill modelled was representative of typical uncemented hydraulic fill and performed adequately, providing the required support resistance. It was only when, for experimental purposes the stiffness response of the fill was reduced to 10% of the original values, that the support offered by the backfill was insufficient. The determination that the support was inadequate was not made using the displacement stabilization criterion but was made by examination of the deformed numerical model geometry, which showed that separation had occurred along the top contact.

Grout packs

The numerical modelling was also used to indicate the limits of applicability of grout packs in providing the required support resistance. The most interesting observation was that the grout packs could support beam thicknesses equivalent to twice the expected dead weight load due to beam building within the discontinuous hangingwall represented by the model. It was noted that this decrease in loading could only be assumed for the modelled conditions, and that the introduction of rogue joints or other persistent features would again increase the load on the support system. It was concluded that simple dead weight loading calculated from beam thickness would represent the worst-case loading for grout packs and should be used for design purposes until further insights are gained. Industry representatives indicated that the strength of grout packs could be increased by increasing the quality of the constitutive grout and by increasing the diameter of the packs. Below, an attempt has been made to estimate the strengths of these larger (and stronger) packs and to incorporate these into a design chart indicating the spacing for various types of units for various heights of instability.

Conceptual layout designs for stopes

Proposed stope layout for stopes supported with backfill

The simplest and most obvious approach to designing a stope layout is to employ and amend strategies that are currently in operation. High percentage filling on a breast mining layout is employed at Northam Platinum. This strategy can be amended to stopes with 100 m long panels between stabilizing pillars on a breast mining layout. A typical stope is shown in Figure 4. The two controlling parameters are the panel length and the pillar width (a). The panel length is taken as 100 m. Pillar widths are designed such that these pillars do not fail or crush. Note that this layout assumes the existence of left- and right-handed

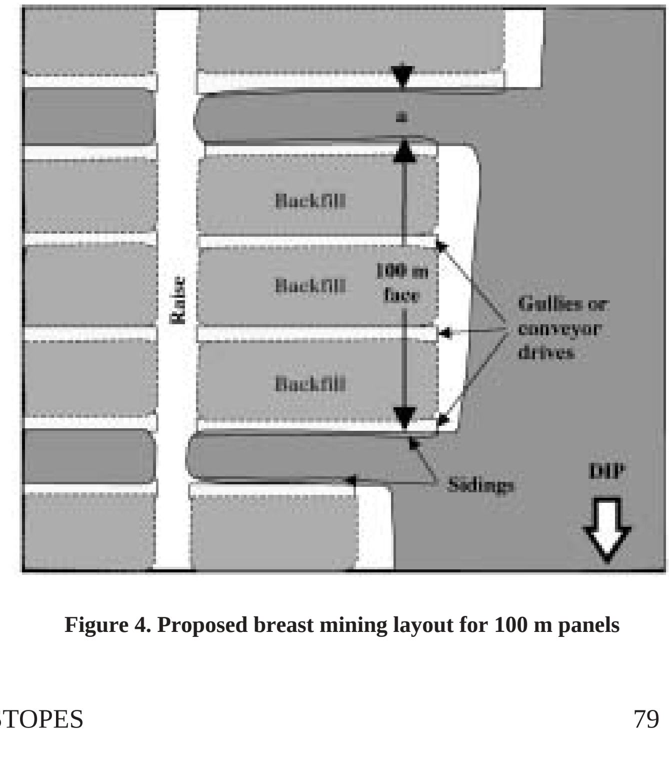

Figure 4. Proposed breast mining layout for 100 m panels

machines and that more than one machine is operational at any one time.

To accommodate rail or face-conveyor mounted rock-breaking machines the 100 m long face is kept straight and uninterrupted by leads and lags. This is considered both practical and feasible with respect to sound rock engineering practice except perhaps under very high stress conditions where severe dynamic rock mass failure along a straight face is possible.

At least one and possibly two strike drives or gullies will be required for access to the 100 m long face. This is considered particularly to ease backfilling. These drives or gullies would be in addition to the top and bottom gullies, which would require sidings. Leach, *et al.*[5] indicated that sidings of 2 m would be adequate to ensure gully stability. Some mechanized means of footwall-lifting the gullies some distance behind the face would be advantageous. The spoil could be packed in the sidings or against the top of the backfill on the down-dip side of the gully. The sidings could also be supported by some form of reinforced fillpacks.

The face orientation should be kept slightly underhand as illustrated to keep mining and backfill drainage water on the face. This water would need to be collected in a sump arrangement immediately above the bottom gully and piped out of the stope. This would keep the ore being transported out of the stope relatively dry.

Up- and down-dip layouts are considered less suitable because of the difficulties in backfilling on the horizontal and a loss in confinement of the fill during placement in the down-dip layout. For the up-dip layout on the UG2 in particular, fines loss into and under the backfill would be a problem as water jetting cannot be employed.

Transport of ore from the face will depend on the environment and the nature of the mining machine. If a high throughput is expected from multiple faces it may be feasible to install conveyor belts along the gullies and extend these installations as the face advances. This will be particularly applicable where the machine operates similarly to a coal mining longwall. A trackless system employing load haul dumpers and a centralized conveyor belt or orepass system is also feasible where conditions allow.

Required backfill characteristics.

The required backfill characteristics are those of typical uncemented hydraulic fill derived from Merensky tailings as shown in Figure 5.

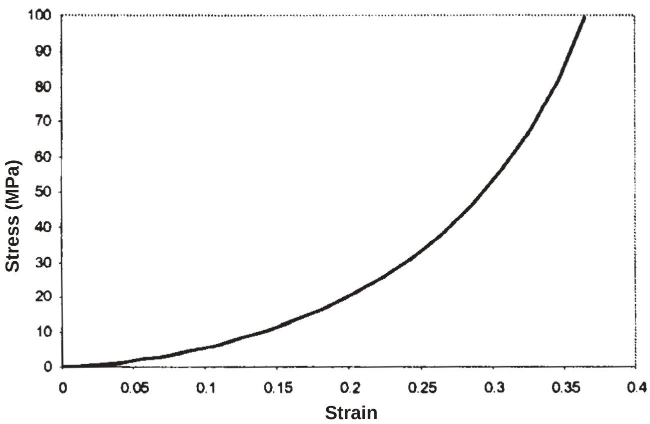

Figure 5. A suitable backfill stress-strain curve

Proposed layout for stopes supported with grout packs

The layout for grout-pack supported stopes would be similar to that of the backfilled stopes. Where backfill is indicated in Figure 4, the area will be supported with regularly spaced grout packs. The number of strike gullies will be determined by operational requirements and logistics. The spacing of these units will depend on the pack strength.

Previous modelling indicated that the loading on grout packs was approximately half the expected dead weight load; it must be emphasized that this observation is only applicable for the modelled geometry and loading conditions. The presence of a single low-angled discontinuity may result in the full dead weight loading being applied to the support units. The actual loading is also very much dependent on the k-ratio and contact conditions between blocks. A thorough sensitivity analysis, including modelling and *in situ* instrumentation would be required to accurately determine the loading on grout packs. Using information from such analyses and employing statistical techniques to analyse case studies would allow the determination of the probability of failure and hence safety factors. The worst case scenario is represented by dead weight loading.

The density of the grout pack units can be related to the dead weight capacity of the support by applying tributary area loading theory. If the grout pack unit strength is known then design charts for the selection of appropriate support units and spacing can easily be drawn up.

Required grout pack characteristics

The strength of grout pack units is variable across the industry. Consultation with persons in the industry[6] and manufacturers[7] indicated that both the diameter of the units and the strength of the constituent grout can be engineered to provide packs of various strengths.

Estimating the strength of grout packs requires some approximation of the relationship between pack diameter and strength. King and Jager[8] performed some tests on comparable packs of different size. Plots of strength vs. w/h ratio for these packs indicate, as expected, increasing strength with w/h ratio. Each of these curves only consists of two or three points; however, the curves must also pass though the origin. It is expected that the pack strengths will be governed by a power law, as the range of interest is from w/h ratio of 0.5 to 1.0. Applying power law curves gives a consistent exponent of around 0.6. To estimate the strength of a 1.5 m diameter grout pack, the strength of the 0.9 m grout pack (2.2 MN) is entered on the graph, shown as Figure 6, and another point corresponding to the strength of the 1.5 m pack is varied until an exponent close to 0.6 is obtained for the best-fit power-law curve. The data from King and Jager[8] and the projected strength curve for grout packs is presented in Figure 6. As indicated, a projected 1.5 m pack strength of 7.5 MN (4.2 MPa) gives an exponent of 0.5875.

The effect of grout strength is much easier to determine. The UCS of the grout will relate directly to the strength of the pack. The finer points of the post-failure behaviour are not considered.

Simple tributary area calculations for the stress on each unit gives a relationship between the unit spacing and the dead weight capacity of the support. This exercise is performed for the 0.9 m and 1.5 m diameter grout packs. Two materials are considered: one corresponding to the modelled strength and another high-strength material twice

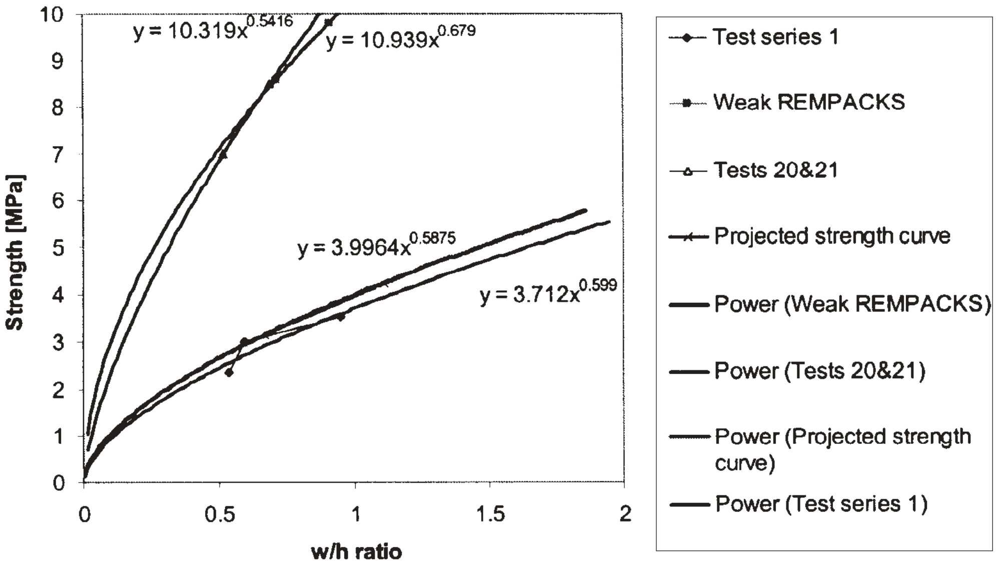

Figure 6. Grout pack strength as a function of w/h ratio for various grout pack types

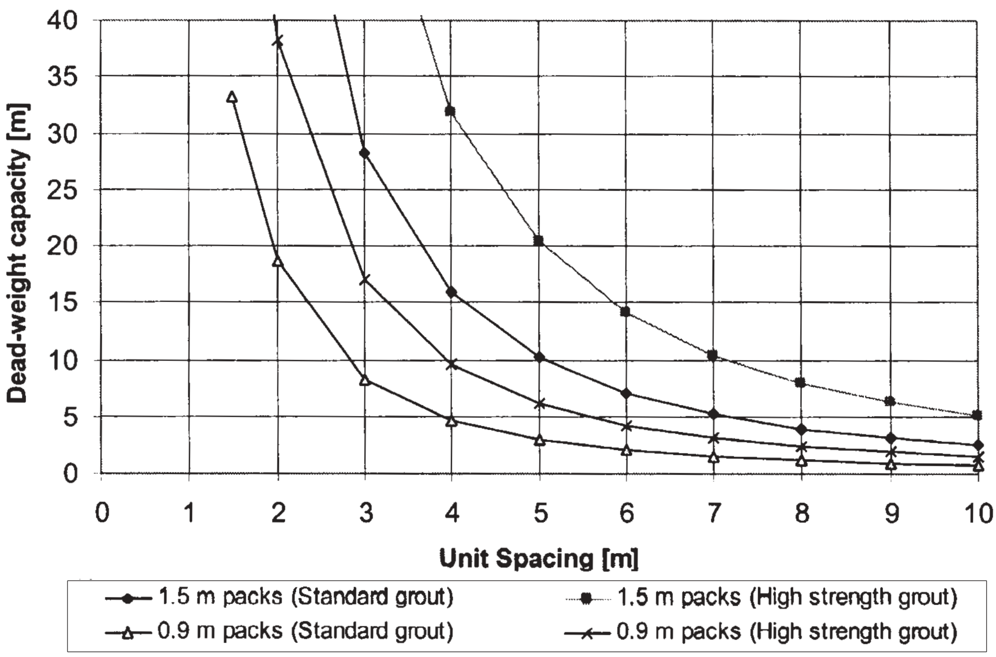

Figure 7. Dead weight capacity for various grout packs as a function of unit spacing

as strong as the modelled grout. Design charts are presented in Figure 7 where spacing refers to centre-to-centre spacing of units in a square pattern.

The maximum expected dead weight load of 1.1 MPa corresponds to a height of 30 m of hangingwall. This situation will require a centre-to-centre spacing of 1.5 m for the weakest 0.9 m diameter grout packs and 4.1 m for the strongest 1.5 m diameter grout packs. In this case, the weaker 0.9 m diameter pack could not be practically used as the skin-to-skin spacing would be only 0.6 m. In cases where the maximum parting height is known, the rock engineer is presented with options to satisfy the support requirement. For example, for a 15 m discontinuity height, standard 0.9 m diameter packs can be employed on a 2.2 m spacing, or high-strength 1.5 m diameter packs can be spaced at 5.9 m. The stability of the hangingwall between support units is not considered in this analysis, but should be considered by rock engineering personnel. The support pattern for the grout packs can be tailored to the environment or operational requirements of the mine. The design chart above considers regularly spaced units, but can easily be amended for various geometries. Any pattern that satisfies the dead weight requirement will be acceptable. Future underground instrumentation and further work could allow a relaxation from using the worst case dead weight loading scenario for grout pack support design.

Conclusions

The estimation of support resistance that results in stable stopes was achieved through back analysis of failed panels and *in situ* instrumentation and measurement. It was found that the maximum total support resistance required to stabilize a stope is around 1.1 MPa. For most Bushveld conditions, this represents an upper limit of the required support resistance though the required support resistance may be lower under certain circumstances. This lower limit is not known. It is the belief of the researchers that this maximum value was obtained via a sound scientific process and is a reliable indicator of the support resistance that is required to prevent failure of the stope hangingwall known as backbreak.

The numerical modelling indicated that the backfill modelled had sufficient increase in the rate of stiffness with strain to provide the 1.1 MPa support resistance before excessive deformation of the hangingwall occurred. This backfill was representative of typical uncemented hydraulic fill derived from Merensky tailings.

A question which has remains open is the maximum tolerable displacement at which the required support resistance must be provided. It is believed that this value is site specific and is probably related to the quality of the rock mass. The Northam back analysis gives a guideline and shows that that the support resistance of 1.1 MPa was obtained at a backfill displacement of around 12% strain. Until further work is done, it is recommended that the backfill must provide a support resistance of 1.1 MPa at a maximum strain of 12%.

The numerical modelling was also used to indicate the limits of applicability of grout packs. The most interesting observation here was that the grout packs could support beam thicknesses equivalent to twice the expected dead weight load due to beam building within the discontinuous hangingwall. It was noted that this decrease in loading could only be assumed for the modelled conditions, and that the introduction of low-dipping joints or other persistent features would again increase the load on the support system. It was concluded that simple dead weight loading calculated from beam thickness would represent the worst case loading for grout packs and should be used for design purposes at present. Future underground instrumentation and further work could allow a relaxation from using the worst case dead weight loading scenario for grout pack support design.

Industry representatives indicated that the strength of grout packs could be increased by increasing the quality of the constitutive grout and by increasing the diameter of the packs. An attempt has been made to estimate the strengths of these larger (and stronger) packs and to incorporate these into a design chart indicating the spacing for various types of units for various heights of instability. Laboratory testing of these larger grout packs are to be undertaken in the future and this data will then supersede the strength estimates made in this paper.

This paper has derived design recommendations based on the best available knowledge. Optimization of the design parameters will require further research. These parameters include laboratory and *in situ* grout pack strength determination, insights into grout pack spacing and the determination of the minimum acceptable backfill quality that will achieve the required support resistance.

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