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The design of stable stope panels for near-surface and shallow mining operations

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Unplanned stope panel collapses occur on most near-surface and shallow mines in South Africa. Although these incidents often occur during blasting, they pose a major threat to the safety of underground workers and the economic extraction of orebodies. Most stope panel collapses occur due to excessive panel lengths that are not designed according to a systematic design procedure or methodology. Instead, panel lengths are often dictated by the equipment in use and by previous experience under similar conditions. Hence, a rock engineering design methodology for the design of stable stope panels between pillars is of vital importance for optimum safety and production in shallow mining operations.

Using the proposed design methodology described in this paper, rock mechanics practitioners and mine planners should be able to identify and quantify the critical factors influencing the stability of stope panels. The critical factors should then be used as input to the design of stable stope panels that will provide the necessary safe environment for underground personnel working in stopes.

The proposed design methodology is an iterative process, which includes aspects such as rock mass characterization, estimation of rock mass properties, identification of potential failure modes, consideration of appropriate stability analyses and other elements of the rock engineering design process. This process continues after implementation. During monitoring, new data should be included in the database and the design process repeated on a regular basis. Design approaches included in the proposed design methodology include empirical methods such as different rock mass classification methods, analytical methods such as elastic and Voussoir beam analyses, kinematic analyses, probabilistic analyses, and numerical analyses.

Keywords: stability, stope panels, design methodology, hard rock mines, shallow mining operations

Introduction

Instability in stope panels in near-surface and shallow mines manifests itself as rockfalls from the hangingwall. Rockfalls from unstable stope panels vary in size from rockfalls between support units, to rockfalls spanning between pillars or solid abutments, to rockfalls bridging several panels and pillars. The focus of this paper is major in-panel stability/instability, i.e. what spans between pillars will be stable given the geotechnical conditions?

Very few mines design stope panels according to a systematic design procedure or methodology. Rock mass characterization, estimation of rock mass properties, identification of potential failure modes, appropriate stability analyses and other elements of the rock engineering design process are often neglected. Instead, panel lengths are often dictated by the equipment in use and by previous experience under similar conditions. Consequently, stope panel collapses often occur on near-surface and shallow mines due to their excessive panel lengths. Although these incidents often occur during blasting, they pose a threat to the safety of underground workers and to the economic extraction of orebodies. Hence, a methodology for the rock engineering design of stable stope panels between pillars is of vital importance for optimum safety and production in shallow mining operations.

This paper describes the factors governing the stability/instability of stope panels in order to define a suitable design methodology for shallow mining operations. Using this methodology, rock mechanics practitioners and mine planners should be able to identify and quantify the critical factors influencing the stability of stope panels. The critical factors should then be used as input to the design of stable stope panels that will provide the necessary safe environment for underground personnel working in stopes.

Evaluation of rock engineering design methods

The design methods which are available for assessing the stability of stope panels can be broadly categorized as follows:

- empirical methods
- analytical methods
- observational methods.

Empirical design methods

Empirical design can be defined as experienced-based application of known performance levels. It is believed that empirical design of stope spans is the most predominant design approach. Rock mass classification methods form the formal part, and engineering judgement based on experience, the informal part of empirical design of stope panels.

Rock mass classifications relate practical experience gained on previous projects to the conditions anticipated at a proposed site. They are particularly useful in the planning and preliminary design stages of a rock engineering project but, in some cases, they also serve as the main practical basis for the design of complex underground structures.

Although rock mass classifications have provided a systematic design aid, modern rock mass classifications have never been intended as the ultimate solution to design problems, but only as a means towards this end. According to Bieniawski (1989), modern rock mass classifications were developed to create some order out of the chaos in site investigation procedures and to provide the desperately needed design aids. They were not intended to replace analytical studies, field observations and measurements, nor engineering judgement. Hence, rock mass classification should be used in conjunction with observational methods and analytical studies to formulate an overall design rationale compatible with the design objectives and site geology.

Of the rock mass classification systems reviewed by Swart *et al.* (2000), four systems could be considered for evaluating the stability of stope spans. These systems are:

- The geomechanics classification or rock mass rating (RMR) system developed by Bieniawski (1973)
- The Norwegian Geotechnical Institute (NGI), rock quality index or Q-system developed by Barton *et al.* (1974)
- The mining rock mass classification or modified rock mass rating (MRMR) system originally developed by Laubscher (1977). This system is a modification of the RMR system
- The modified stability graph method through the use of the modified stability number, *N*', originally developed by Mathews *et al.* (1981).

The geomechanics classification system

Bieniawski's (1989) rock mass rating classification (RMR_{89}) is the system that is most frequently used today. Over the years since the first publication, the system has benefited from extensions and applications by many authors throughout the world and has stood the test of time. The varied applications point to the acceptance of the system and its inherent ease of use and versatility.

The stability of non-stope excavations can be estimated in terms of stand-up time from the RMR value using the graph in Figure 1 (Bieniawski, 1993). Bieniawski (1973) developed this graph based on the original concept of stand-up time by Lauffer (1958). The accuracy of this stand-up time is doubtful since it is influenced by excavation technique, durability and *in situ* stress, effects which the classification system does not take into account. Therefore, this graph should be used for comparative purposes only.

The NGI or Q-system

On the basis of an evaluation of a large number of case histories of underground civil engineering excavations, most of which were supported, Barton *et al.* (1974) of the Norwegian Geotechnical Institute (NGI) proposed the Q System rock mass classification for the determination of rock mass characteristics and tunnel support requirements.

According to Barton *et al.* (1974), the maximum unsupported span can be estimated from the following equation:

$$Max\ span\ (\text{unsupported}) = 2\ ESR\ Q^{0.4}$$

where: *ESR* (*excavation support ratio*) is a value that is assigned to an excavation in terms of the degree of security that is demanded of the installed support system to maintain the stability of the excavation. Hutchinson and Diederichs (1996) recommend that an ESR not more than 3 be used for temporary mine openings.

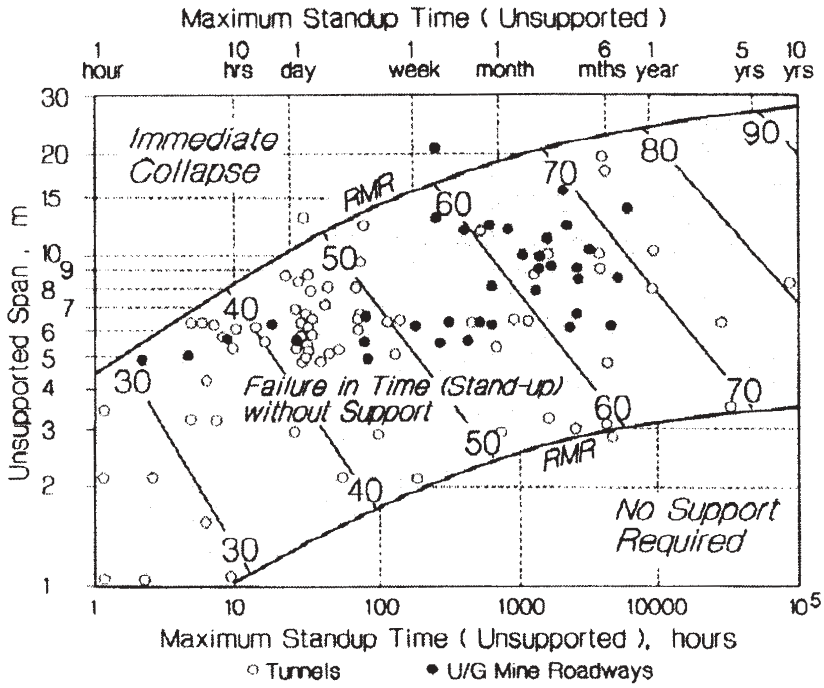

Figure 1. Relationship between unsupported span, stand-up time and RMR (after Bieniawski, 1989 and 1993)

Houghton and Stacey (1980) produced a graph (Figure 2) showing the relationship between unsupported span and *Q* value for different factors of safety. Owing to the different methods of excavation and in rock mass parameters, stability cannot be precisely determined. Stacey and Page (1986) suggest the use of a factor of safety of 1,0 or less, depending on exposure of personnel.

Hutchinson and Diederichs (1996) produced a similar graph (Figure 3) illustrating the relationship between Q value and maximum unsupported span for different ESR.

The mining rock mass rating system

Laubscher has developed a stability/instability diagram (Figure 4), which is based on case studies mainly from Zimbabwe, Chile, Canada, USA and South Africa. It is used to estimate the stability of a given excavation in terms of mining rock mass rating (MRMR) and hydraulic radius (HR).

The modified stability graph method

The factor *Q*' is used along with factors *A* (rock stress factor), *B* (joint orientation adjustment factor) and *C* (gravity adjustment factor) to determine the modified stability number, *N*', which is used in the modified stability graph method (Mathews *et al*., 1981; Potvin, 1988; Bawden, 1993 and Hoek *et al*., 1995) for dimensioning of open stopes in mining.

176 case histories by Potvin (1988) and 13 by Nickson (1992) of unsupported open stopes are plotted on the stability graph shown in Figure 5. This graph can be used to evaluate the stability of stope panels.

Estimation of stable stope panels using different rock mass classification systems

Different rock mass ratings for borehole core drilled into the anorthosite hangingwall above an MG2 chromitite seam are compared in Table I.

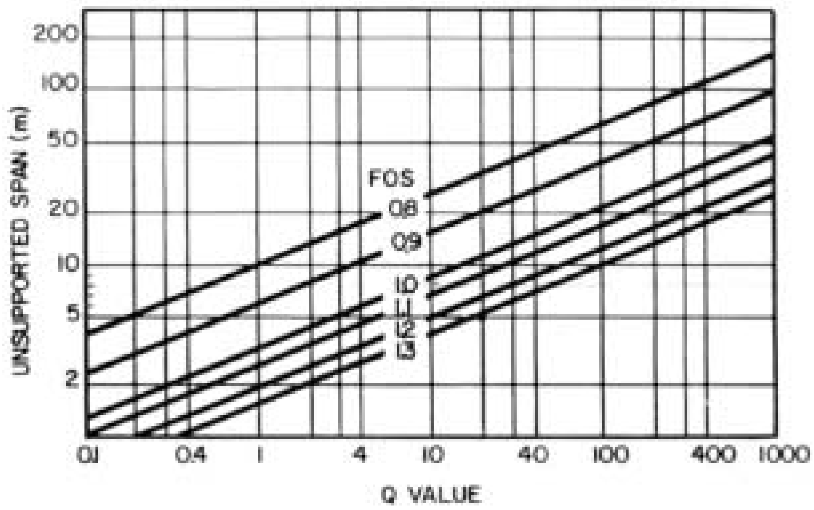

Figure 2. Relationship between unsupported span and Q value (redrawn after Houghton and Stacey, 1980)

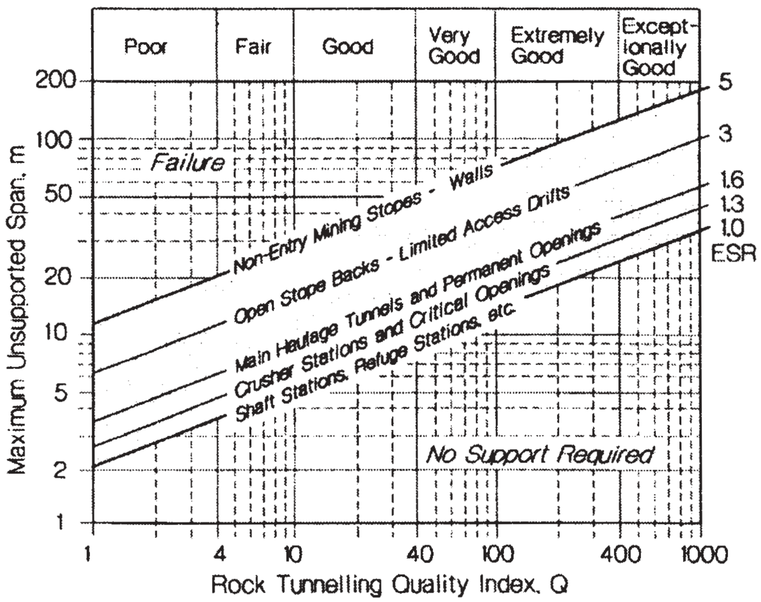

Figure 3. Relationship between maximum unsupported span and Q value (after Hutchinson and Diederichs, 1996)

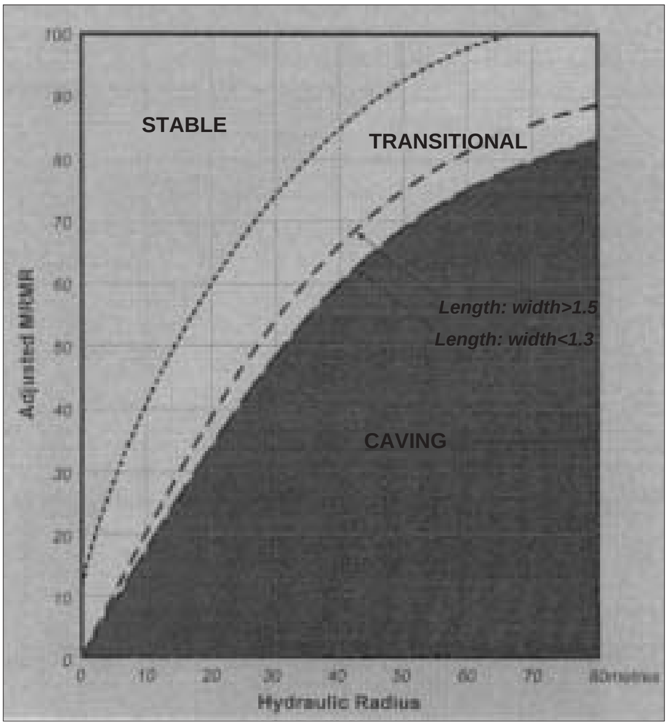

Figure 4. Stability diagram illustrating the relationship between MRMR and HR (after Laubscher, 1993)

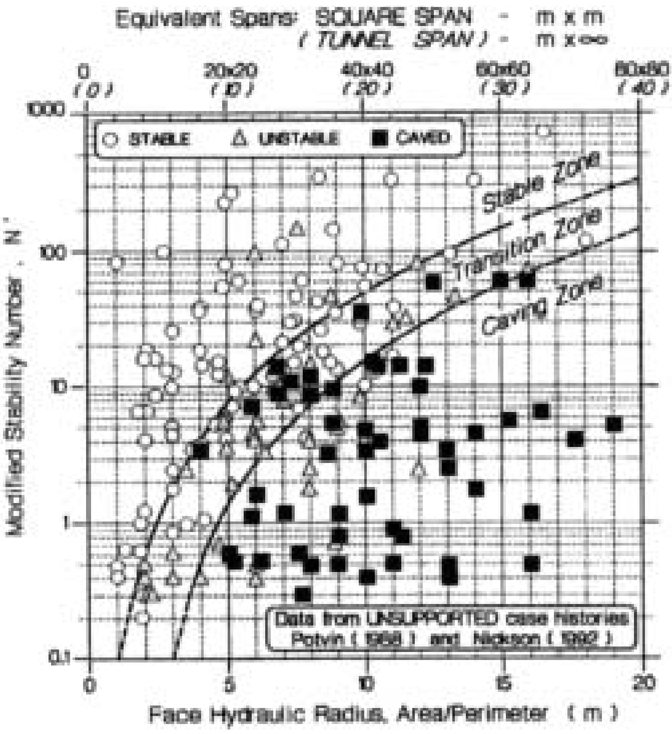

Figure 5. Stability graph (after Potvin, 1988 and Nickson, 1992)

The corresponding stable panel spans determined from these rock mass ratings are summarized in Table II. The results show a reasonable correlation, which is acceptable for prefeasibility studies. It also shows the importance of using more than one rock mass classification system in order to make decisions.

Other rock mass classification systems

Existing rock mass classification systems are sometimes modified or new systems are developed to suit local conditions. Two examples are:

- The modified NGI system or Impala system (Watson and Noble, 1997)
- The rating system developed by the Australian Geomechanics Society (Pells, 2000).

More recently, Watson (2004) concluded that none of the current rock mass classification systems adequately describe the relevant geotechnical conditions of the

Table I
Comparison between different rock mass ratings

Lithology	Borehole No.	RMR_B	RMR_L	Q	Q'	MRMR	N'
Anorthosite	1	82	68	13	36	55	197
	2	82	75	62	74	61	411
	3	77	70	59	71	57	391
	4	60	53	8	10	43	56
	5	53	47	5	7	38	36
	6	77	65	22	27	53	149
	7	82	74	42	50	60	277
	8	74	64	8	10	52	55
	9	82	82	119	143	67	789
	10	77	74	34	41	60	229
	11	75	71	63	75	58	416
	12	82	75	63	75	61	416

Table II
Stable panel spans using different rock mass ratings

Borehole	Stable panel spans (m)			
no.	RMR	Q	MRMR	N'
1	27	17	32	34
2	27	31	42	>40
3	25	31	36	>40
4	16	14	24	22
5	13	12	16	18
6	25	21	30	30
7	27	27	40	36
8	24	14	30	22
9	27	41	50	>40
10	25	25	40	38
11	24	31	36	>40
12	27	31	40	>40

Bushveld Platinum Mines. A hybrid of several current systems, the 'New Modified Stability Graph' was therefore developed.

Analytical design methods

Rock mass classification systems have several limitations and cannot be used in isolation. It is therefore necessary to extend the design process to include analytical design methods. Analytical methods involve the formulation and application of certain conceptual models for design purposes. The aim is to reproduce the behaviour and response of the stope panel.

Analytical methods include such techniques as closed form solutions, numerical methods and structural analysis. They are effective in designing stope panels because they enable comparative assessments of the sensitivity of stope panel stability for varying input parameters. It is important that analytical methods and failure criteria be selected that can model the anticipated or identified failure mechanism and mode of failure most appropriately. As for the empirical design methods, it is important that more than one analytical design approach be used so that an understanding can be gained of the likely failure zones and extent of failure.

The following are examples of analytical design approaches for different geotechnical conditions:

- Design of stable stope panels in stratified or bedded rock using beam-type analyses
- Design of stable stope panels in blocky ground using keyblock or wedge-type analyses
- Design of stable stope panels in massive rock using numerical stress analyses and appropriate failure criteria.

Beam analyses

Mining in stratified rock masses or rock masses with pseudo-stratification is common. Such 'stratification' is not only the result of sedimentary layering but can form through excavation-parallel stress fracturing of massive ground or can be the result of fabric created through igneous intrusion/extrusion or metamorphic flow processes.

This structure is an important factor in the consideration of stability of the roof of excavations in such rocks. Two important factors influence the behaviour of a laminated roof: firstly, that the tensile strength perpendicular to the laminations is very low or zero and, secondly, that the shear strength on these laminations is very low compared to the shear strength of the intact rock. It is, however, possible for the rock in the roof to span the excavation by forming a rock beam. Beam analyses include elastic and Voussoir beam analyses.

The following conclusion can be drawn from elastic beam analysis theory:

- The maximum beam deflection is at the centre
- The maximum shear forces are at the abutments of a beam
- The maximum bending moment is at the abutments of a beam
- Axial stresses reach a maximum at the end of a beam
- Shear stresses reach a maximum at the end of a beam. Therefore, relative movement along the interfaces of different roof strata is likely to occur close to the ends of a beam. Because the shear stresses in the centre of a beam is zero, roofbolts installed at the centre of a panel contribute little to the formation of a composite beam
- The thickness and span of a beam have a significant influence on the maximum deflection of a beam and the horizontal stresses at the end of a beam. For example, if the effective thickness of a roof beam comprising four individual layers of thickness *T*/4 can be increased to a total thickness *T* by roofbolting, then the maximum deflection of the bolted beam is 1/16th and the maximum horizontal stress is $^1/_4$ of the four-member beam. Also, by reducing the panel span from *L* to *L*/4, the maximum deflection of the beam is 1/256th and the

maximum horizontal stress is 1/16th of a full-length beam. Therefore, controlling the panel span is the most effective means of controlling the roof without artificial means.

Elastic beam theory is useful in explaining the deformation and failure of the mine roof in bedded deposits and can be used to design safe stope panels if the limitations of the theory are appreciated. If, however, sub-vertical joints are present in the roof, the tensile strength of the rock beam will be zero. A stable roof beam will only form if a stable compression arch can develop.

Evans (1941) introduced the Voussoir arch concept into rock engineering to explain the stability of a jointed rock beam. The most significant difference between the Voussoir beam theory and the elastic beam theory is the fact that the Voussoir beam material has no tensile strength in the horizontal direction. The essential features of a roof supported by Voussoir action are illustrated in Figure 6 for a horizontal beam.

Diederichs and Kaiser (1999) proposed an iterative solution scheme for the determination of stability and deflection of a Voussoir beam by calculating the buckling limit, the factor of safety for crushing at midspan and the abutments, the factor of safety for sliding at the abutments, and the midspan deflection.

A buckling parameter of 35% has been determined by Hutchinson and Diederichs (1996) as a limit above which a roof should be considered unstable. This design limit of 35% happens to correspond to a midspan deflection of 10% of the beam thickness. Therefore, arch stability can also be assessed by monitoring the displacement at midspan, relative to the undeflected state.

In the Voussoir analysis, if the plate thickness is decreased, then the snap-through and crushing FOS decrease and the shear FOS increases. The arch midspan displacements also increase. When the midspan displacement reaches about 10% of the plate or beam thickness, arch collapse is imminent.

The input parameters and results from a Voussoir beam analysis carried out for a pyroxenite hangingwall beam above an LG6 chromitite seam are summarized in Table III.

According to the analysis, snap-through or buckling failure is the dominant failure mode with very high probabilities of failure, even for 'good' ground conditions. In this case, the beam thickness is small in relation to the beam length. Therefore, the probability of shear failure is low.

Keyblock analysis

In shallow underground excavations in hard rock, failure is frequently controlled by the presence of discontinuities such as faults, shear zones, bedding planes and joints. The intersection of these structural features can release blocks or wedges, which can fall or slide from the surface of the excavation. Failure of the intact rock is seldom a problem in these cases where deformation and failure are caused by sliding along individual discontinuity surfaces or along lines of intersection of surfaces. Separation of planes and rotation of blocks and wedges can also play a role in the deformation and failure process.

Analysis of the stability of these excavations depends on:

- Correct interpretation of the structural geology
- Identification of potential unstable blocks and wedges
- Analysis of the stability of the blocks and wedges, which can be released by the creation of the excavation
- Analysis of the reinforcing forces required to stabilize these blocks and wedges.

Exposed stope hangingwalls, which are intersected by numerous joints or fractures and which contribute to the formation of unstable keyblocks, should be stabilized by supporting as many keyblocks as possible. It is impractical to attempt to map each joint or fracture and carry out a stability analysis to identify potential keyblocks. A design tool is therefore required which will allow the evaluation of the type and frequency of keyblocks that may be formed and of the effect of different support systems on the probability of failure of the keyblocks.

Several different types of keyblock models have been developed. Daehnke *et al.* (1998) evaluated keyblock models and concluded that keyblock stability can be evaluated best by using a probabilistic design approach. Such models should be used to complement beam stability analysis to determine stable spans for jointed beams.

According to Daehnke *et al.* (1998), probabilistic keyblock analysis shows that, as the size of keyblocks increases, the probability of falling out between supports decrease but the probability of the block failing the supports increase. Larger blocks, however, might become self-supporting through other mechanisms such as the Voussoir beam concept.

Observational design methods

Observational design methods rely on the monitoring of ground movement during mining to detect measurable instability. If necessary, the original design is then adjusted to optimize panel stability. The observational approach would require a large database and would have to be implemented in the initial stages of stoping in order to achieve some reliable measurement of stability.

Peck (1969) formalized the observational design method. In this approach, further data is collected during excavation, and the performance of the excavation is monitored. The new data is then fed back into the original characterization results and the models, designs or conclusions revised as appropriate. Therefore, adding further data during the monitoring of stope performance is an essential component of the ongoing rock mass characterization process.

Data collection from selected mines

Swart *et al.* (2000) visited several shallow base metal and diamond mines in South Africa in order to investigate stable and unstable stopes under different geotechnical conditions and to assess the influence of factors governing the stability of stope panels. The following conclusions were drawn from this survey:

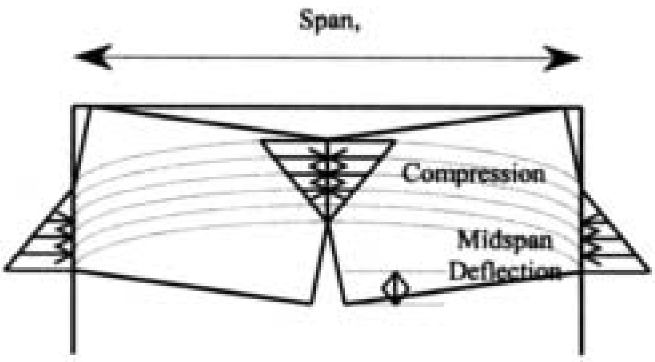

Figure 6. A horizontal beam supported by Voussoir action

Table III
Example Voussoir beam analysis

Input Parameters:										
Ground Conditions		Bad ground			Good ground			Very good ground		
Panel Spans (m)		28 x 15 m	28 x 20 m	28 x 28 m	28 x 15 m	28 x 20 m	28 x 28 m	28 x 15 m	28 x 20 m	28 x 28 m
Beam thickness (m)		0.8	0.8	0.8	0.8	0.8	0.8	0.8	0.8	0.8
Rock unit weight (MN/m^3)		0.034	0.034	0.034	0.034	0.034	0.034	0.034	0.034	0.034
Overburden load (MN/m^3)		0.043	0.043	0.043	0.043	0.043	0.043	0.043	0.043	0.043
Face dip (degrees)		10	10	10	10	10	10	10	10	10
Overburden thickness (m)	mean	0.4	0.4	0.4	0.4	0.4	0.4	0.4	0.4	0.4
	std. dev.	0.1	0.1	0.1	0.1	0.1	0.1	0.1	0.1	0.1
Support pressure (MPa)	mean	0	0	0	0	0	0	0	0	0
	std. dev.	0	0	0	0	0	0	0	0	0
RMR	mean	40	40	40	60	60	60	70	70	70
	std. dev.	20	20	20	20	20	20	20	20	20
Intact (m)	mean	10	10	10	13	13	13	17	17	17
	std. dev.	4	4	4	4	4	4	4	4	4
Intact strength (MPa)	mean	70	70	70	100	100	100	130	130	130
	std. dev.	40	40	40	40	40	40	40	40	40
Rock mass modulus (MPa)	mean	10000	10000	10000	30000	30000	30000	40000	40000	40000
	std. dev.	5000	5000	5000	20000	20000	20000	20000	20000	20000
Poisson's ratio	mean	0.2	0.2	0.2	0.2	0.2	0.2	0.2	0.2	0.2
	std. dev.	0	0	0	0	0	0	0	0	0
Results:										
Shear	mean SF	5.09	5.04	46.31	12.25	12.24	50.21	22.07	21.49	20.58
	std. dev.	2.96	3.05	47.42	6.91	6.61	44.28	13.93	12.51	11.13
	fail. prob. (%)	8.36	9.27	16.96	5.18	4.46	13.32	6.52	5.08	3.93
Arch snap-through	mean buckling param.	10	41	78	2	14	54	0	4	22
	std. dev.	10	28	22	2	14	46	0	4	18
	fail. prob. (%)	medium	high	v. high	low	medium	v. high	low	low	medium
	mean midspan displ. (mm)	37.2	108.3	140.3	16.5	41.9	36.1	9.1	22.8	63.9
Compression	mean SF	11.5	6.4	1.9	17.4	11.1	9.3	23	15.2	3.9
	std. dev.	6.8	4.1	2.6	7.2	4.7	3.2	7.5	5	4.5
	fail. prob. (%)	6.01	9.37	35.59	1.17	1.63	0.51	0.16	0.22	26.17

- Not one mine is using a proper engineering approach to stope panel and support design
- Although most FOG incidents/accidents are associated by failure along geological structures, most mines do not use any design methodology based on structural analysis. In a few cases, complicated numerical analysis programs are used on an ad hoc basis to assess structurally controlled panel stability
- In most cases, panel lengths are based on local experience and equipment requirements
- Stope pillars are normally designed conservatively and the probability of regional instability, involving several stope panels, is unlikely
- Valuable information regarding panel stability are often lost because FOG incidents are not investigated, recorded or back-analysed
- Beam stability analyses are applicable to several mines and should be considered as one of the potential failure modes
- There is a need for a systematic engineering approach to the design of stable stope panels.

Risk assessment

Joughin *et al*. (1998) analysed 328 FOG accident records for the period 1988 to 1997, including 55 fatal accident records available from the South African Mines Reportable Accident Statistics System (SAMRASS) and back-analysed 42 fatal accidents using empirical methods as part of the SIMRAC research project OTH 411. Some of the conclusions drawn from this research are:

- The average injury and fatality rates on some smaller mines were significantly higher than on the larger mines
- Although more injuries and fatalities occurred on the larger mines, some smaller mines had almost as many, and sometimes even more injuries and fatalities than the larger mines
- The injury and fatality rate for mines with tabular orebodies were significantly higher than for mines with other orebodies
- The most common forms of failure were wedges, blocky hangingwalls and weak hangingwalls
- Only 10 per cent of rocks had dimensions greater than 5.9 m x 2.8 m x 1.2 m
- In most of the fatal FOG accidents, support standards were inadequate or no support standards existed. This could be attributed to the lack of rock engineering involvement and the fact that support standards are often compiled by mine management with very little understanding of the geotechnical conditions of the rock mass.

Joughin *et al*. (1998) found that, even after analysing the accident information stored in SAMRASS, and all the available fatal accident reports for falls of ground, root causes were still difficult to identify and suggested that the fault-event tree methodology for risk assessment be used to identify significant hazards and quantify the significant risks.

Swart *et al*. (2000) assessed the significant risks associated with stope panel stability during two one-day workshops. The risk assessment was based on the fault-event tree analysis technique. First, the significant hazards associated with stope panel collapses were identified using the information obtained from a literature survey, site visits and personal experience. The significant hazards were then analysed systematically to form a cause tree. Probabilities of occurrences were then allocated based on a judgemental basis to form a fault tree.

The significant hazards identified can be grouped into the following main categories:

- inadequate rock wall strength due to *in situ* rock mass conditions
- inadequate rock wall strength due to adverse geometry (size, shape, orientation, location)
- inadequate support strength
- adverse loading conditions.

Considering the significant hazards identified in the risk assessment, it is important that an engineering approach to stope panel design be followed. Such an approach should include aspects such as:

- rock mass characterization
- estimation of rock mass properties
- identification of potential failure modes
- appropriate stability analyses.

Rock engineering design methodology

Bieniawski (1984) distinguishes between the following ten stages in the design process:

- Recognition of a need or a problem.
- Statement of the problem
- Collection of information
- Concept formulation in accordance with design criteria: search for a method, theory, model, or hypothesis
- Analysis of solution components
- Synthesis to create a detailed solution
- Evaluation and testing of the solution
- Optimization
- Recommendation
- Communication
- Implementation

Stacey and Page (1986) suggest that the following approach to underground excavation design be followed:

- Determine shape and size of excavation based on the purpose of the excavation. In the case of stoping, the shape and size will be determined by the orebody geometry and the chosen mining method
- Consider an 'ideal' excavation which best satisfies the purpose
- Consider the practicality of this 'ideal' opening in relation to the properties of the rock mass in which it will be located
- Ascertain the stability of the 'ideal' excavation by:
  -determining the mode of any identified instability
  -testing for stress induced failure around opening
  -testing for instability of large blocks
  -classifying the rock mass and test for rock mass instability
  -if necessary, optimizing 'ideal' excavation in terms of size, shape, orientation or location in order to overcome instability. (Loop back to (3) if any geometrical changes are made)
  -if modified excavation is still unstable, determining support required to overcome instability. (Appropriate support will depend on the risk associated with an excavation.)

Brown (1985) listed the following components of a generalized programme for underground excavation design:

- Site characterization
- Geotechnical model formulation
- Design analysis
- Rock mass performance monitoring
- Retrospective analysis.

Proposed design methodology for stable stope panels

From the above, it is concluded that the design of stable stope panels should be a process of defining the means of creating stable stope panels for the safety of underground workers and optimum extraction of the orebody. Therefore, a method is required whereby all rock properties, their variability, and an understanding of all rock mechanisms affecting the stability of stope spans are used as a fundamental base. A procedure for identifying the mechanisms and rock properties relevant to the specific problem is then required. In this way, existing knowledge should be used in an optimal way to design site-specific stable stope spans.

Hence, it is proposed that the design methodology illustrated in Figure 7 be used for the design of stable stope panels. It is important to note that it is an iterative process with feedback loops that test/evaluate the design aspects such as:

- design assumptions/premise
- design objectives/desired outcomes
- the acceptability of the risk (i.e. are the significant risks tolerable?)

Rock mass characterization

The roof or hangingwall of stope panels must be characterized in order to:

- define the rock mass condition
- evaluate the rock mass strength and deformation behaviour
- identify the most likely modes of potential rock mass failure
- determine the most appropriate method of stability analysis or design
- define geotechnical areas
- define rock mass properties
- evaluate the stability of the rock mass
- evaluate the support requirements of the excavation.

The following steps are required to characterize the rock mass in the roof of stope panels:

- Collection of geotechnical data
- Evaluation or estimation of the boundary conditions
- Rock mass classification
- Recording and presentation of geotechnical data.

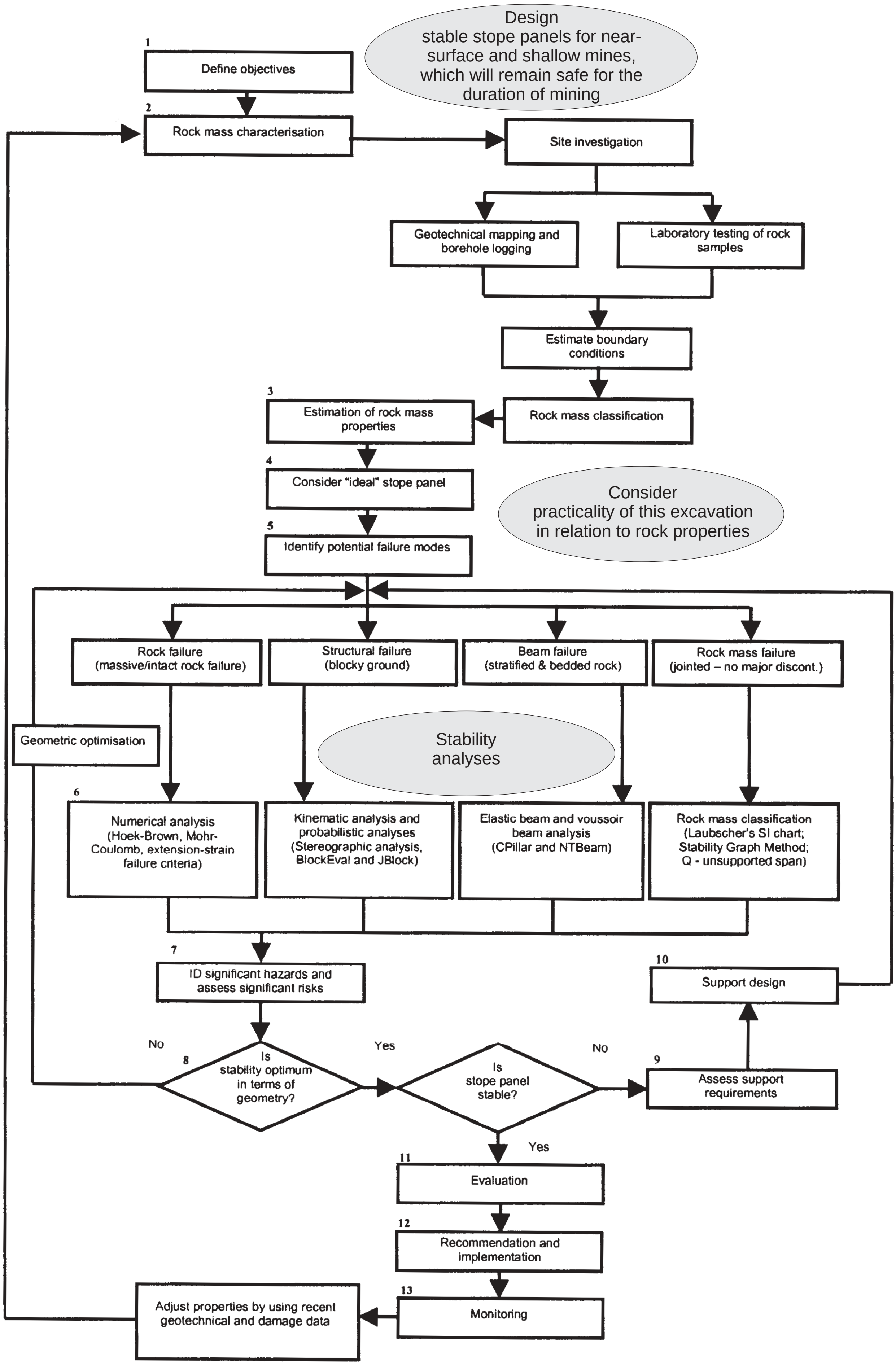

Figure 7. Proposed design methodology for stable stope panels

Collection of geotechnical data

The collection of sufficient and reliable geotechnical data forms the basis on which the rock mass will be characterized and is essential for the design of stable stope panels. This information represents the independent variables which cannot be controlled by the design engineer. The source of potential geotechnical data and the extent of the existing geotechnical database depend on the stage of the investigation (prefeasibility, feasibility preliminary design or final design stage).

Geotechnical data should be obtained through:

- site investigations
- geotechnical drilling and logging of borehole core
- mapping of exposed rock surfaces
- laboratory testing of rock samples.

Evaluation or estimation of boundary conditions

In situ stresses determine the confinement imposed on the rock mass and are an essential boundary condition for the evaluation of stability. They can have the following effects on stability:

- instability may occur if the stress is low since rock blocks may have the freedom to fall out
- the rock mass will be well confined and stable if the stress is higher
- instability may occur due to rock fracturing if the stress level is sufficiently high.

Stress measurements using an overcoring technique, small flat jacks or hydrofracturing are expensive and time consuming. It is therefore suggested that *in situ* stress conditions be estimated based on the work carried out by Stacey *et al*. (1998).

Rock mass classification

Rock mass classification can be used for the design of stable stope panels. In essence, rock mass classification relates practical experience gained on previous projects to the conditions anticipated at a proposed site. It is particularly useful in the planning and preliminary design stages of a rock engineering project but, in some cases, it also serves as the main practical basis for the design of complex underground structures.

To derive input properties for further analyses, data obtained from underground mapping, logging of borehole core and laboratory tests should be rated according to an appropriate rock mass classification system. It is suggested that at least two rock mass classification systems be used to obtain a picture of the rock mass fracturing, the characteristics of the fractures (planarity, roughness, infilling and continuity) and the location of major, continuous fault structures. The average and range of the rock mass classification values (e.g. Q and RMR) should be determined as a means of estimating the variability of the quality of each rock type.

Recording and presentation of geotechnical data

Geotechnical data collected must be recorded and presented such that it will be readily available and easily understood. This could include presentation of:

- borehole data in well-executed geotechnical logs
- mapping data as spherical projections
- relevant geological and geotechnical data for rock mass classification purposes
- longitudinal sections and cross-sections of structural geology
- construction of a geotechnical domain model.

Estimation of rock mass properties

It is extremely important that the quality of input data matches the sophistication of the design methods. It is, however, almost impossible to perform controlled laboratory tests on large, jointed rock samples. Therefore, estimates of the strength and stiffness properties are typically made by using rock mass classification in combination with laboratory determination of intact rock strength.

The following rock mass properties can be estimated from the rock mass classification data:

- Hoek and Brown m and s parameters
- E.

Consider 'ideal' excavation

The geometry of an 'ideal' excavation is typically controlled by factors such as:

- drilling equipment
- cleaning equipment
- scheduling
- full constraints
- labour and equipment efficiencies
- orebody dimensions.

Identification of potential failure modes

The following failure modes should be considered:

- structurally controlled, gravity-driven failures
- stress-induced, gravity-assisted failures.

Stability analyses

Appropriate analyses should be carried out to assess the stability of the 'ideal' excavation. Wedge and block failure types could be analysed using analysis programs such as JBlock (Esterhuizen, 1994). Beam failure types should be analysed using the Voussoir beam analysis procedure proposed by Hutchinson *et al.* (1996) or programs such as CPillar or NTBeam.

Identification of significant hazards and assessment of significant risks

All significant hazards should be identified using an appropriate risk assessment methodology. Care must be taken to ensure that the hazards are identified systematically. Significant hazards/risks should then be eliminated or reduced to acceptable limits by optimizing the geometry, installing support, etc.

Geometrical optimization

If stability analyses show that the assumed 'ideal' stope panel is unstable, or the support required to ensure stability is either unpractical or uneconomical, the 'ideal' stope panel must be optimized by considering one or more of the following geometrical changes:

- location
- orientation
- shape
- size.

Evaluation of support requirements

Most 'ideal' stope panels, even when optimized in terms of geometry, are inherently unstable and require support to improve the strength or capacity of the rock mass such that it will remain stable and safe, at least during extraction of the panel, and as long as access is required through the panel.

Appropriate support will depend on the risk associated with the excavation. Therefore, although the probability of rockfalls occurring could be high, the risk of rockfall accidents could be minimized by minimizing entry to the stope panel.

At the same time, a database and damage maps should be developed identifying the location and mechanisms of failure in the mine. These failures should be back-analysed using appropriate failure models and criteria in order to prevent a reoccurrence.

Roof supports are used to help stabilize underground openings. Their performance characteristics must be properly matched to the loading environment and ground behaviour if they are to succeed.

The key characteristics of any support includes its maximum load carrying capacity, stiffness, and residual strength. Other important factors are timing of installation, the stability of the support as it is loaded, and the capability of the support system to provide skin control.

Evaluation of ideas and solutions

The solution proposed must now be interpreted and compared with the original objectives. This calls for a clear understanding of all pertinent interacting factors; that is, for the exercise of engineering judgement. If the evaluation shows up deficiencies or suggests more promising alternatives, loop back to the stability analysis stage.

Optimization

Optimize the 'ideal' opening with respect to location, orientation, shape and size. If any geometrical changes are made, loop back to Stage 4.

Conclusions and recommendations

Provide a concise statement of the answer to the problem, point out the limitations or restrictions and indicate the direction to be followed in implementing the solution.

Monitoring

Monitor performance and take remedial measures in case of instability.

Conclusions

The following general conclusions could be drawn:

- Most mines do not use proper engineering approaches to stope panel and support design
- Although most FOG incidents/accidents are associated with failure along geological structures, most mines do not use any design methodology based on structural analysis. In a few cases, complicated numerical analysis programs are used on an ad hoc basis to assess structurally controlled panel stability
- In most cases, panel lengths are based on local experience and equipment requirements
- Valuable information regarding panel stability is often lost because FOG incidents are not investigated, recorded or back-analysed
- Beam stability analyses are applicable to several mines and should be considered as one of the potential failure modes
- There is a need for a systematic engineering approach to the design of stable stope panels
- Rock mass classification forms an integral part of stope panel design. Although it cannot be used directly for stability analysis purposes, it should be used to estimate rock mass properties required for analytical designs.

Recommendations

- More than one rock mass classification method and analytical design approach should be used to assess ground conditions and to carry out stability analyses
- A systematic design approach should be followed to design stable stope panels
- Analytical methods and failure criteria should be used that can model the anticipated or identified failure mechanism and mode of failure most accurately
- All failure modes should be considered during stope panel design. This also applies to FOG investigations where hangingwall failures are often described in terms of 'blocky' and 'weak' only. Beam failure modes (buckling, shear and crushing failure) should be included
- The Voussoir beam analysis technique has been used successfully in other countries and should be considered for stope panel designs in stratified rock
- The proposed design methodology should be used during all stages of the mining process, from pre-feasibility to final design and implementation, and when compiling codes of practice to combat rockfall accidents.

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