Design and support of rockpasses at ultra-deep levels

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Synopsis

The DEEPMINE Research Programme sought to develop the technology and competence to mine gold safely and profitably at depths of 3 to 5 km. Among the many issues addressed was the stability and support of rockpasses at ultra-depth. It was found that high virgin and mining-induced stresses could cause the walls of rockpasses to spall, leading to the deterioration and failure. A comprehensive programme of numerical modelling was conducted to investigate alternative mining layouts and sequences to ameliorate the problem.

Introduction

The aim of the DEEPMINE Collaborative Research Programme (1998–2002) was to create the technology and competence needed to mine gold safely and profitably at depths of 3000 to 5000 m in the Witwatersrand basin of South Africa. The great technical and human challenges of mining at these depths led to unprecedented co-operation among mining companies, research institutions, universities, labour, and government. The research programme covered a wide range of disciplines, including industrial sociology, physiology, psychology, mining and mechanical engineering, and earth sciences. While DEEPMINE's focus was on mining at ultra-depth, many of the developments have direct application to current mining operations. Among the many critical issues addressed was the stability and support of rockpasses at ultra-depth. Over a dozen research tasks addressed aspects of this problem, and the principal findings are outlined in this paper.

Rock mass behaviour at ultra-depth

In situ *observations of stress and stress-induced fracture*

In order to predict the stability and define the support requirements of rockpasses at ultra-depth, it was first necessary to characterize the rockmass and the stress environment. Existing rock property and stress data were compiled and analysed (Güler *et al.*, 1999b and 2000b). It was concluded that the rock types and patterns of joints and faults that will be encountered at ultra-depth are similar to areas of current mining, although the thickness of the various strata will change and the rocks may become increasingly argillaceous. In the Witwatersrand Basin the vertical component of stress (σ_v) tends to increase according to the weight of the overburden (27 MPa/km). The horizontal stresses are subject to considerable variation, but tend to increase according to σ_h = 10 MPa +10 MPa/km (Jager and Ryder, 1999). The virgin stress k-ratio (average horizontal stress:vertical stress) was estimated from an analysis of the 23 best quality stress measurements that have been made close to the areas of potential ultra-deep mining. The depths of the measurements range from 1226 m to 2650 m below surface. The analysis indicates that the k-ratio at ultra-depth will be in the range 0.3 to 0.8. The intensity of fracturing is expected to increase significantly with depth, although the extent of the fracture envelope is expected to remain much the same as at present.

Prior to the DEEPMINE Programme, the greatest depth at which a stress measurement had been made in a South African gold mine was 2650 m. Stress was successfully measured at a depth of 3352 m at No. 4 Shaft, Kloof Gold Mine, despite complications due to thermal effects, borehole closure, and core discing (Coetzer *et al.*, 2000). The vertical stress was measured to be 91.0 MPa, in close agreement with the calculated overburden stress of 88.8 MPa. The north-south and east-west horizontal components were 75.5 MPa and 71.6 MPa, respectively, giving a k-ratio of 0.8.

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The major principal stress was calculated to be 142 MPa with a dip of 47°.

Observations were made of the scaling or 'dog earing' of boreholes drilled in highly stressed ground. For example, a vertical hole bored at a depth of 3500 m at Western Deep Levels as a precursor to shaft sinking was found to have dog eared from its original 2 m diameter to a span of 7 m. In this case, the ratio between the horizontal stresses was 1:0.7 and the k-ratio was 0.5 (Güler *et al.*, 1999a).

Laboratory, analytic, and numerical modelling studies

Slow- and rapid-loading triaxial tests were conducted on rock specimens containing circular openings (Güler *et al.*, 1999a). Based on the extrapolation of the laboratory data, it was predicted that failure around boreholes would initiate when the major *in situ* stress is equal to the uniaxial compressive strength (UCS), while failure around large circular openings such as shafts, tunnels and rockpasses would initiate at half the UCS. Underground data, however, indicated that failure may take place at even lower stress levels.

Theoretical studies of the stability of breakout zones were conducted using numerical modelling techniques calibrated by laboratory experiments (Güler *et al.*, 1999a). Quartzite discs were compressed between steel platens, inducing spalling on the edges of the specimens (Figure 1). The experimental results were used to evaluate various numerical modelling codes such as FLAC (Itasca, 1995) and DIGS (Napier, 1990). Typical results are shown in Figure 2. Constitutive models based on a shear failure criterion are often used in theoretical and numerical studies of the post-failure behaviour of brittle rock. It was found that they do not adequately replicate underground observations in highly stressed excavations, where fracturing typically gives rise to relatively slender slabs (spalling). The observed fracturing was replicated more successfully by plastic and brittle strain softening constitutive models.

Modelling based on strain softening behaviour and the Mohr-Coulomb failure criterion was used to investigate the effect of opening shape and size and rock mass properties on fracturing. It was assumed that the fracture zone is approximated by the zone in which failure is predicted. It was concluded that the depth of fracturing could conservatively be approximated as linearly proportional to the acting stress, for a given opening shape and rock properties (Güler *et al.*, 2000a).

Although the stress level at which fracturing is initiated is affected by the opening shape, rock properties and blasting, it was concluded that there is no practical way (either by modifying the opening shape, size, or excavation method) to prevent the occurrence of stress-induced

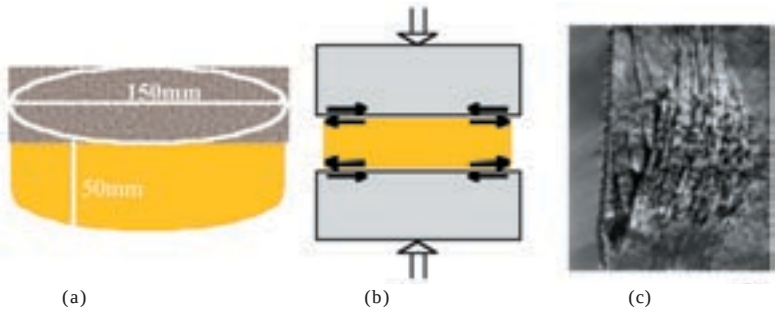


Figure 1—Laboratory tests conducted to evaluate numerical simulations of spalling (a) Specimen wrapped with tape to contain the fractured material (b) Dilating material confined between hard steel platens inducing horizontal compressive stresses (c) Observed fracture pattern in quartzite specimens. The fractures initiate near the edge of the discs and propagate towards the core with increasing vertical compaction

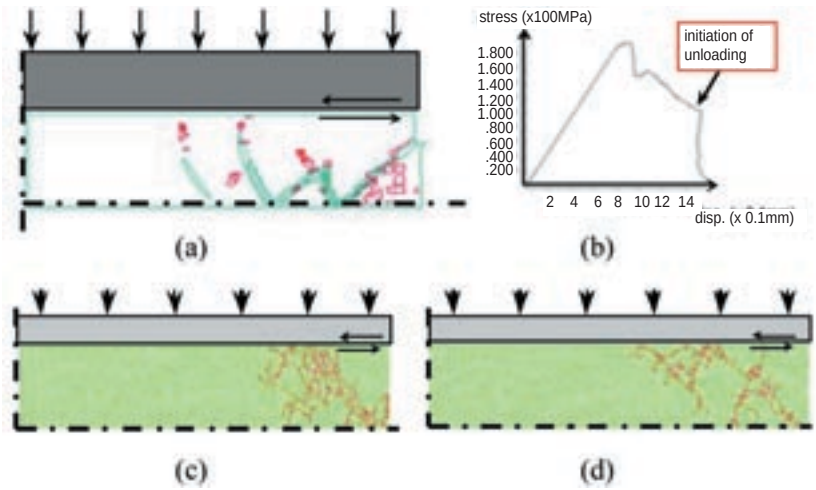


Figure 2—Numerical modelling studies (a) FLAC strain softening model. Tensile stresses (red) are induced by plastic shear deformations (green). Failure is localized in shear bands, unlike the observed behaviour where failure is localized in closely spaced extension fractures. The modeled load-deformation relationship (b) also appears to be unrealistic. (c) DIGS perfectly plastic model (cohesion = 25 MPa, friction angle = 30°, tensile strength = 2 MPa and (d) DIGS brittle strain softening model (cohesion = 20 to 0 MPa, friction angle = 0° to 30°, tensile strength = 2 MPa) better replicate the observations

fracturing at the virgin rock stresses anticipated at ultra-depth. Shotcrete (or similar coatings) can be used to stabilize overstressed rock under static stress conditions, while additional measures are required for dynamic loading.

Support components and systems

Tendons

Sheared rock tendons have been observed in rockburst investigations, and it was deemed important to quantify this aspect of their performance (Güler *et al.*, 1999a). The specifications, test results, and costs of tunnel support components and systems were exhaustively reviewed, and laboratory tests conducted to fill some important knowledge gaps (Figure 3). It was found that grout properties and borehole dimensions play an important role. The shear strength is generally greater than the maximum tensile strength, the shear deformation capacity being optimum where the grouted borehole is twice the diameter of the tendon.

Fabric

The fabric of a support system refers to the elements that contain fractured rock between tendons or props. In the case of a tunnel or rockpass, the fabric is typically comprised of mesh and lacing, and/or shotcrete. A programme of dynamic tests was conducted to provide empirical criteria for the design of support systems (Figure 4). The tests showed that the ability of a system to contain damage depends on the spacing between rockbolts, the thickness of the shotcrete, and the presence of components such as lacing that contribute tensile strength (Güler *et al.*, 1999a).

Design methodology

Investigations, such as those described above, indicated that systems comprised of currently available components provide viable solutions to the support of tunnels at ultra-depth. A methodology to design tunnel support systems was formulated (Güler *et al.*, 2000a, see Figure 5.)

> Tendon spacing is determined by an energy criterion. It is assumed that the entire fractured region is subjected to rockburst loading with a peak velocity of 3 m/s and must be contained by the tendon support. The depth of damage is the only input parameter. The support spacing is determined by the energy-absorbing capacity of each type of support unit, e.g. cone bolt, grouted rebar, hoist rope, cable bolt. It is assumed that the tendon is anchored in undamaged rock.
> Shotcrete thickness is determined by a punch strength criterion. For a given shotcrete panel span (determined by the tendon spacing) and thickness, the punch strength of the panel must be greater than the yield strength of the associated tendons.
> Different combinations of tendons and shotcrete that meet the energy and punch strength criteria should be considered to derive the most cost-effective design.

Lining of rockpasses

Hagan and Acheampong (1999a) noted that while the raiseboring of rockpasses offers the advantages of speed and reduced blast damage to the wall, the technology is unable to prevent the rapid development of 'dog earing' and subsequent unraveling of the rock. They recommended the development of a method to shotcrete concurrently with raiseboring. An engineering feasibility study was undertaken (O'Brien, 2002), which included the design of the collapsing reamer, a shotcrete applicator, the powering of this equipment, and methods to remotely control the lining operation (Figure 6).

Shaft rockpasses

Best practice

A comprehensive survey of rockpasses in deep gold mines was conducted by Hagan and Acheampong (1999a, 1999b). It was concluded that rockpasses could be used at ultra-depth, provided current knowledge and experience is prudently applied. Best practices with respect to the design, support and maintenance of rockpasses were identified. The ideal ultra-deep rockpass (Figure 7) should be:

> in competent rock, or lined if in a very weak and jointed rock mass and in potential impact zones to prevent damage to support units and to redistribute lateral stresses

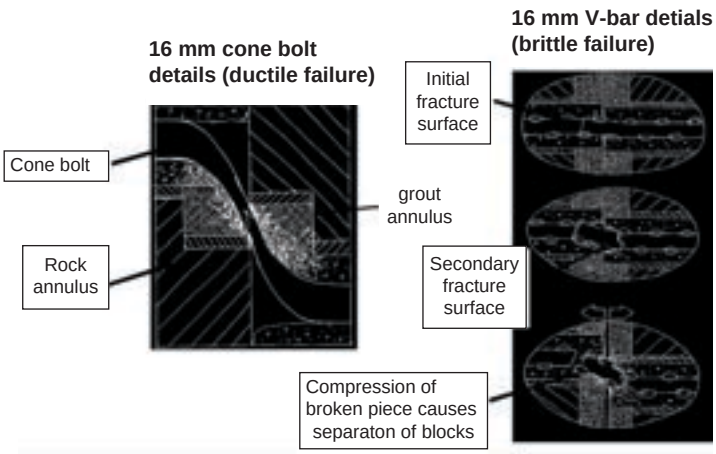


Figure 3—Laboratory studies of the shearing of tendons

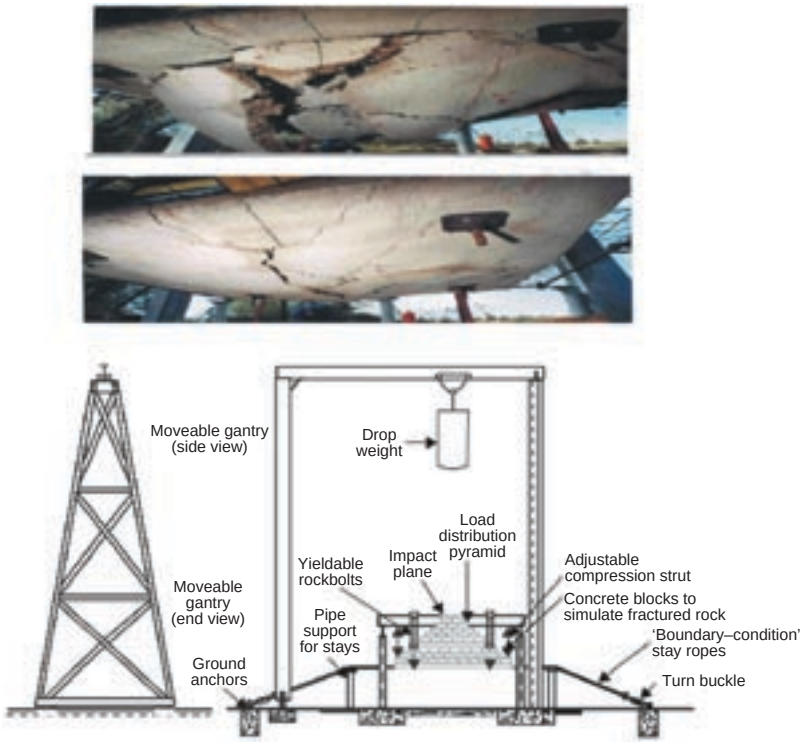


Figure 4—Dynamic weight-drop testing of support systems

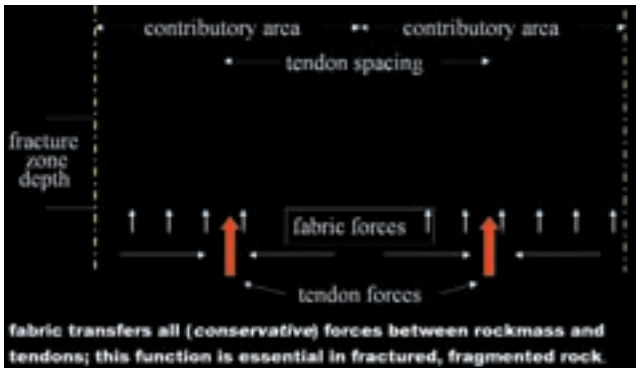


Figure 5—Support design model

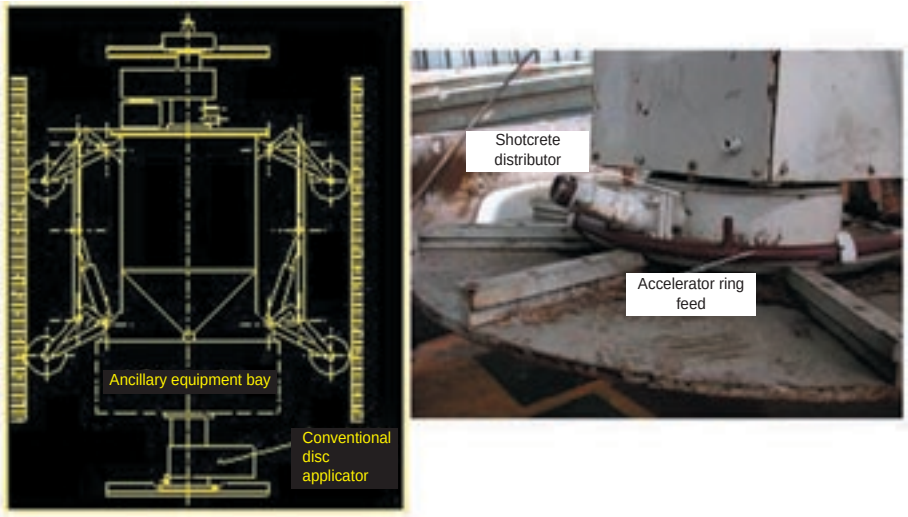


Figure 6—System to line rockpasses concurrent with reaming

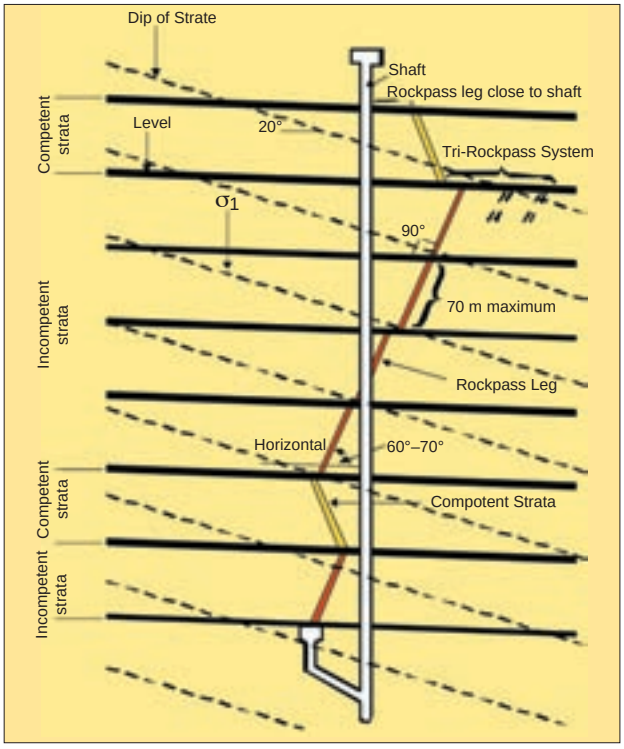


Figure 7—Best practice for ultra-deep rockpasses

as small as possible (while maintaining a suitable rockpass: rock size ratio to prevent blockages) to reduce the area of rock exposed to damage and to reduce support requirements and cost,
supported during development to prevent falls of loose rock, control spalling, and to prevent erosion of loose and fractured sidewall
inclined between 60° and 70°, and orientated as close as possible to the direction of maximum field stress
kept full to inhibit deterioration by providing confinement and reducing impact shock and
regularly monitored and maintained, a tri-rockpass system being recommended to enable regular monitoring and rehabilitation.

Support

Slabbing and spalling failures are typical in highly competent rock under high stress, but can be resisted by a dense pattern of rockbolt or rope anchor support. In moderately competent rock, block and wedge failures are of greater concern, while generalized failures or unraveling may occur in incompetent rock. Tendons should be supplemented by wire mesh or lacing and a strong lining, otherwise wear will cause the rock between the bolts to fall out, and the bolts will protrude and become ineffective (Hagan and Acheampong, 1999a and 1999b).

Rockpasses have been lined with precast reinforced concrete (e.g. Kloof Gold Mine, No. 4 Shaft) and steel tubes (e.g. East Driefontein Gold Mine, No. 1 Sub-shaft). However, shotcrete is a particularly attractive material as it also has the ability to confine the rock surface, enabling the rock to 'support itself'. As ore and waste in South African gold mines are often highly abrasive, special types of concrete/shotcrete (e.g. corundum or andesite lava constituents) are often required.

Multi-level in-shaft loading: an alternative to shaft rockpasses

Alternatives to shaft rockpasses were investigated by Adendorff *et al.* (2001). A literature review was conducted to identify shafts utilising multi-level in-shaft loading. A visit was made to a mine performing this method of rock handling, various hoisting systems were evaluated, a simulation model was developed to determine the feasibility of applying the method to an ultra-deep gold mine, and a financial analysis was conducted.

It was found that a multi-level in-shaft loading system would present less risk to production than a shaft rockpass system due to the independent operation of each loading facility. It is also possible that a multi-level in-shaft loading configuration would facilitate early development and exploitation of reef during sinking operations. Simulation studies show that the hoisting rate will be detrimentally affected if the capacities of ore silos or bins on each of the levels are too small. On the other hand, the creation of large storage capacities would be costly and require rock engineering design. Simulation studies showed that a 1000 ton facility (e.g. a silo 29 m high and 8 m in diameter) on each of twelve tipping levels would provide sufficient storage capacity to support the hoisting of 300 000 tons per month. It was found that friction and single drum winders cannot hoist the required production capacity for ultra-deep mines. Electrically coupled, double drum winders would be suitable for a multi-level in-shaft loading installations. It was concluded that multi-level in-shaft loading installations are a technically feasible and economically attractive alternative to internal shaft orepasses for an ultra-deep mine.

Stope rockpasses

Stope rockpasses are sub-vertical excavations made below stopes to allow for the transfer of rock from stoped faces by means of gravity. Stope rockpasses are normally developed from cross-cuts, and hole into the centre raise the intersection between the strike gullies and centre raises. Functionally, they allow rock to be tipped, stored and transferred, thus forming an integral part of the rock-handling system. The factors that influence the design of shaft rockpasses includes rock mass discontinuities (geology), orientation, size, shape, excavation method, inclination, and proximity to other excavations (Hagan and Acheampong, 1999a). The extraction sequence is an additional parameter that must be considered in the design of stope rockpasses.

Ultra-deep level stope layouts

The stope layouts most likely to be used at ultra-depth were documented and analysed by Vieira *et al.* (2001). In this section the layouts are briefly described, with special reference to stope rockpasses. It must be noted that the stope rockpasses are often also important components of the ventilation and cooling system. Detailed rock engineering evaluations are described below.

Longwall with strike stabilizing pillars (LSP)

Stope rockpasses are developed from upper and lower follow-behind haulages, (Figure 8a). The stress has generally been relieved, except for the rockpasses close to stabilizing pillars or abutments. Rockpass lengths are relatively short, generally less than 50 m.

Sequential grid method (SGM)

One of the most demanding aspects of sequential grid mining is the development of the rockpasses servicing the uppermost part of the stope, as the vertical distance from the cross-cut to reef may be as great as 100 m, depending on the dip of the reef and the back length (Figure 8b). Another major disadvantage of the layout is that the rockpasses are subject to large stress changes during stoping, which may compromise their stability. At a depth of 4000 m, the changes may be as great as from 190 MPa to 230 MPa.

Sequential down dip method (SDD)

This layout employs very short stope rockpasses (Figure 8c). All broken rock is moved from the face area to the wide raise either by scraping or water jetting. It is then scraped to the boxhole at the foot of the raise. A ventilation connection is made from the top of the worked-out section to the upper

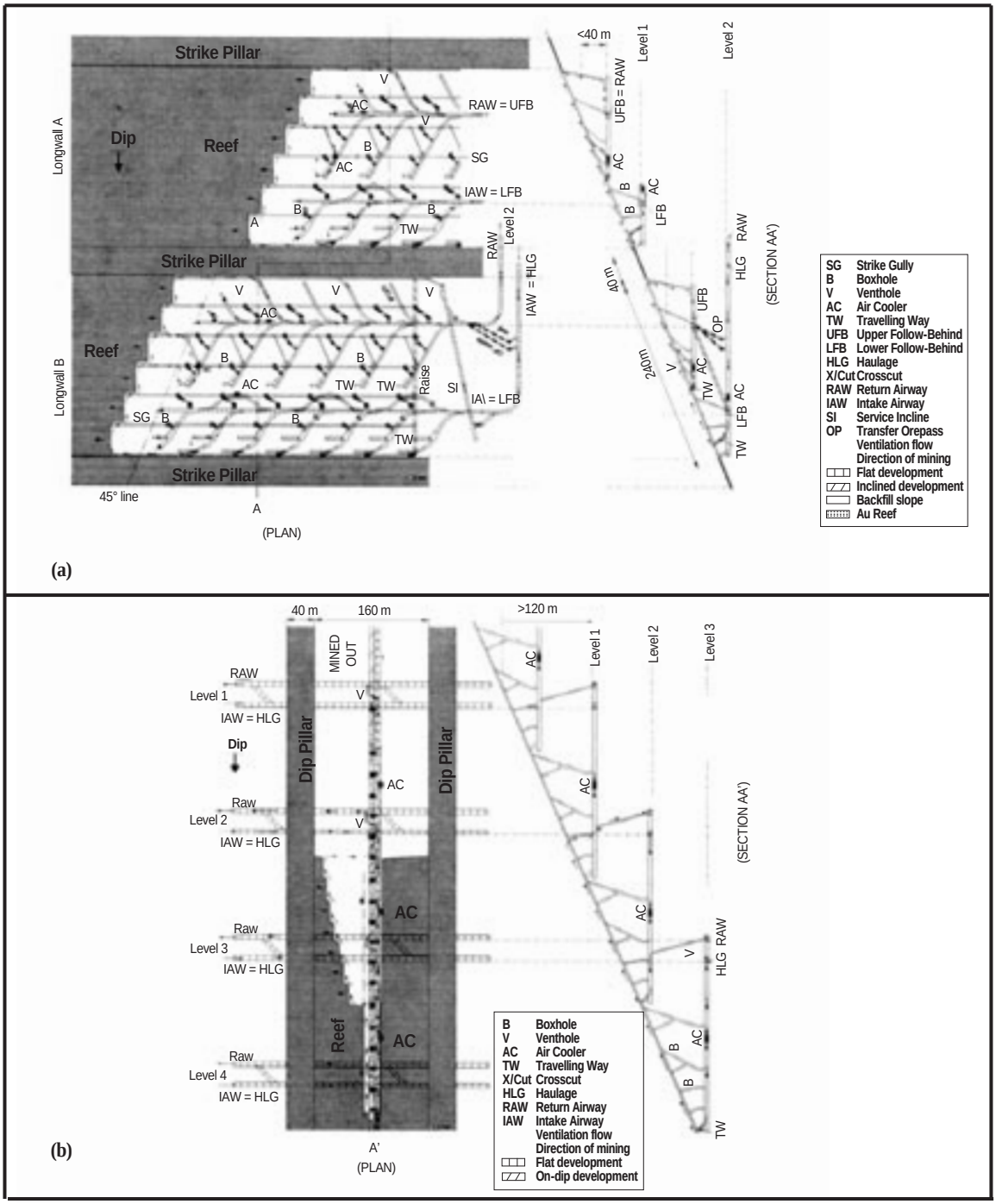


Figure 8—Ultra-deep stope layouts (a) Longwall with strike stability pillars, (b) Sequential grid mining with dip pillars

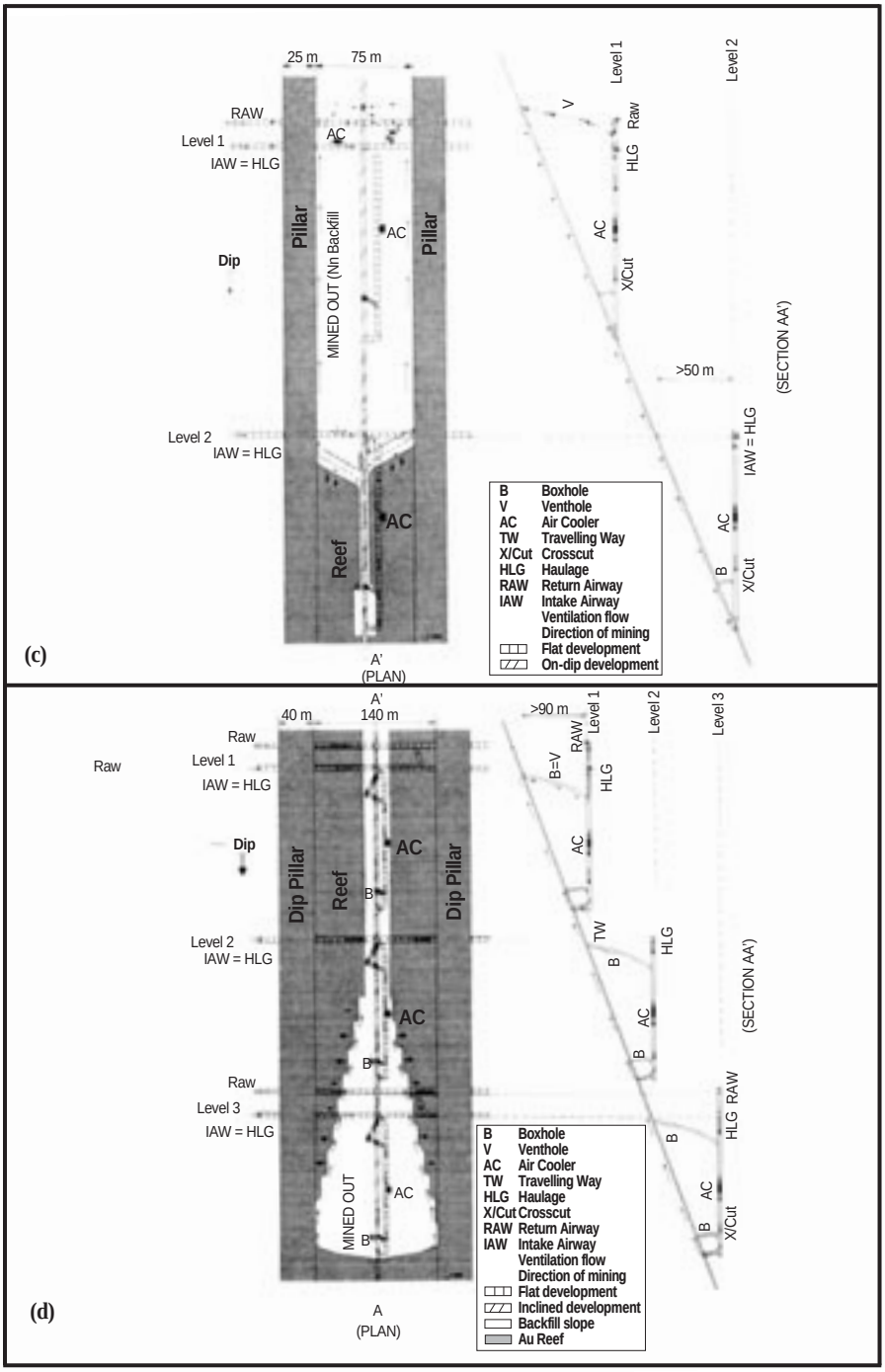


Figure 8—(c) Sequential down-dip with dip pillars, (d) Closely spaced dip pillar layout

level return airway. The dependency on a single short rockpass with limited storage may constrain production output.

Closely spaced dip pillar method (CSDP)

Only two rockpasses service the entire back: the boxhole near the foot of the raise is used only during the excavation of the raise, while the long rockpass at the top of the raise serves as the passage for all the ore once production begins (Figure 8d). The broken rock is moved from the face area to the raise either by scraping or water-jetting, and then moved to the boxhole at the top of the raise by means of a continuous scraper pulling updip. The rockpass at the top of the raise may be as long as 100 m.

Alternatives to stope rockpasses

As stope rockpasses are likely to fail at ultra-depth unless they are well supported, the possibility of dispensing entirely with stope rockpasses was investigated. (MacNulty *et al.*, 2000; Rupprecht *et al.*, 2001). It was concluded that there would be high capital and operational costs and technical risk in dispensing entirely with stope rockpasses. However, the number of rockpasses may be reduced significantly while still

delivering the required output by implementing continuous up-dip scraping techniques. Up-dip scraping also allows water to drain, reducing the amount of water in the rockpass and the risk of mudrushes A reduction in the number of rockpasses places a greater premium on stability, as failure would have an even greater impact on production.

Mechanized methods of developing rockpasses are preferred as they allow practical, safe and timely support. Rupprecht *et al.* (2001) emphasize that the support requirements of stope rockpasses should be evaluated individually, and that the quality of rock will usually be the critical factor in designing a system. Due to the short life span of stope rockpasses, shotcrete combined with rope anchors should provide adequate support in most cases. In some cases, only the sections close to the stope and footwall excavation will require support.

Assessment of tunnel and rockpass stability

The stability haulages, cross-cuts, and rockpasses was assessed for the various mining layouts. The stability of off-reef excavations is inextricably linked: the smaller the middling between the reef and haulage, the less development is required, saving time and money. However, greater mining-induced stresses and stress changes are experienced by the excavation, leading to instability, risk of failure, and greater primary support and rehabilitation costs. An optimum trade-off must be found.

Stability criteria

A number of factors affect the stability of tunnels: the virgin stress, rock type and strength, shape and size of the excavation, the nature and design of support, the excavation method, and stress changes induced by nearby stoping. The extent of damage caused by seismically induced shaking is related to the condition of the tunnel walls. In this study, the Rockwall Condition Factor (RCF) was used to estimate the effect of stoping on the stability of the main service tunnels for various mining layouts and sequences at ultra-depth (Vieira, 2000 and 2004). The MAP3D code (Wiles, 2000) was used for the evaluation.

$$RCF = \frac{3\sigma_1 - \sigma_3}{F\sigma_c} \qquad [1]$$

σ_1 major three-dimensional field stress components (major subsidiary principal stress) acting normal to the long axis of a tunnel
σ_3 minor three-dimensional field stress components (minor subsidiary principal stress)
σ_c uniaxial compressive strength of the rock, and
F empirical rock mass condition factor (for competent rock $F = 1$)

If $RCF<0.7$, tunnel conditions are said to be excellent; if $RCF>1$, conditions rapidly deteriorate and increased levels of support resistance and areal coverage are required; while if $RCF>1.4$, severe deterioration of sidewall conditions is anticipated and a high degree of support is required (Jager and Ryder, 1999). RCF values that indicate poor rock conditions and a need for high quality support were obtained for sections of tunnels positioned below dip-pillars (Figure 9). While an increase in the middling between the reef and the tunnels would improve rock conditions, development costs would also increase (Vieira *et al.*, 2001; Vieira and Durrheim, 2001). Alternatively, the stoping sequence could be modified. An iterative design process is often required, therefore, to arrive at an optimal layout design and scheduling that offers the lowest risk possible of tunnel instability, over its operational life.

The strength factor (*SF*) was used to assess rockpass stability (Wiles 2000). It assumes that a section of rockpass wall will fail when the induced field stresses exceed the stipulated strength of the rock.

$$SF = \sigma_1 / \left[\sigma_c + q\sigma_3\right]$$
$$q = \tan^2\left(45 + \varphi/2\right) \qquad [2]$$

σ_1 major three-dimensional field stress components (major subsidiary principal stress) acting normal to the long axis of a tunnel
σ_3 minor three-dimensional field stress component (minor subsidiary principal stress)
σ_c unconfined compressive strength, and
φ friction angle

Since the exact stress path to failure in rockmasses is unknown, the calculation of strength factor is non-unique. The following properties were used to make strength calculations: UCS of 180 MPa, friction angle of $\phi = 30°$. It was assumed that significant failure conditions along rockpass

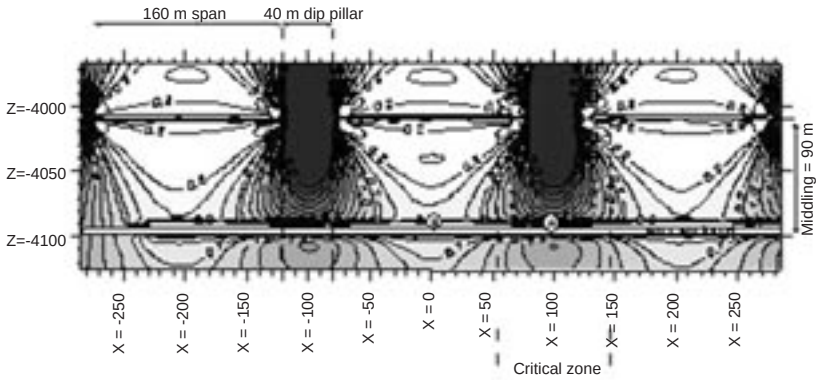


Figure 9—Rockwall Condition Factor for a strike haulage 90 m below reef at a depth of 4000 m. Note that *RCF* >1.4 beneath the dip pillars

walls begin when *SF* > 1, and become critical when induced field stress levels are 2.5 times greater than the stipulated rock strength, i.e. when *SF* > 2.5. Using these criteria, critical failure conditions were identified for all layouts.

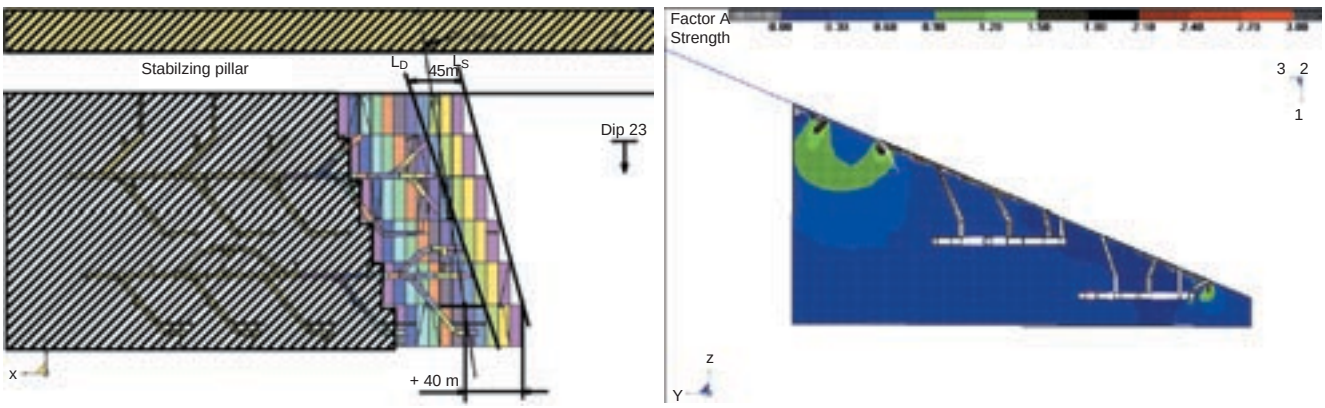
Influence of stoping layout and sequence

Stope orepasses and cross-cuts in LSP layouts are developed in overstoped, stress-relieved rock, avoiding high stress changes caused by stoping. Strength factor analysis indicates that rockwall failure may still occur in the areas adjacent to the strike-stabilizing pillars (Figure 10). Strength factors as high as 8 were calculated in these zones, indicating that both the bottom orepasses and the upper ventilation-holes would be at risk of failure at ultra-depth, the risk becoming more severe as the area mined increases.

High abutment stresses are induced in the footwall along the centre line of SGM raises, due to an unbalanced distribution of regional loading. This is brought about by the asymmetrical two-stage extraction sequence applied in SGM stopes. These conditions prevail until stoping begins on the other side of the raise. Strength factor analysis indicated that SGM orepasses could be severely affected by this asymmetrical loading condition (Figure 11). *SF* values as high as 10 were estimated around SGM rockpasses and travelling ways, with the longer excavations being affected the most. When stoping advances in the opposite side of an SGM raise-line, the high stress field moves away from the line of footwall development, thus stress-relieving the footwall. When this occurs, the prevailing stress field changes from compressive to tensile, causing fractured walls to dilate and induce further instability. These results indicate, therefore, that single-sided SGM sequences should be avoided at ultra-depth, and that double-sided sequences could improve conditions along the line of footwall development. Because the rate of extraction would be potentially higher in stopes subjected to double-sided extractions, care should be exercised when planning such operations, however, as sudden loading of regional pillars and an increase in the rate of face seismicity could occur. These aspects must be investigated prior to the implementation of a double-sided extraction sequence at ultra-depth.

Down-dip mining sequences in SDD layouts can also cause unstable rockwall conditions at stope-entrances and bottom rockpasses. Instability is particularly evident around tip areas, when down-dip panels advance pass these respective rockpass positions (Figure 12). Wide-ledging can stress-relieve the rockpass rockwalls to some extent, but does not completely prevent loss of confinement, and consequent failure, at the tip area. Changes in stress from compressive to tensile, in both rockpasses and stope entrances, are responsible for rockwall conditions worsening when descending stope faces approach such positions. A siding to the stope entrance could be winzed during ledging, pushing the zone of highly fractured rock beyond the reef intersection position.

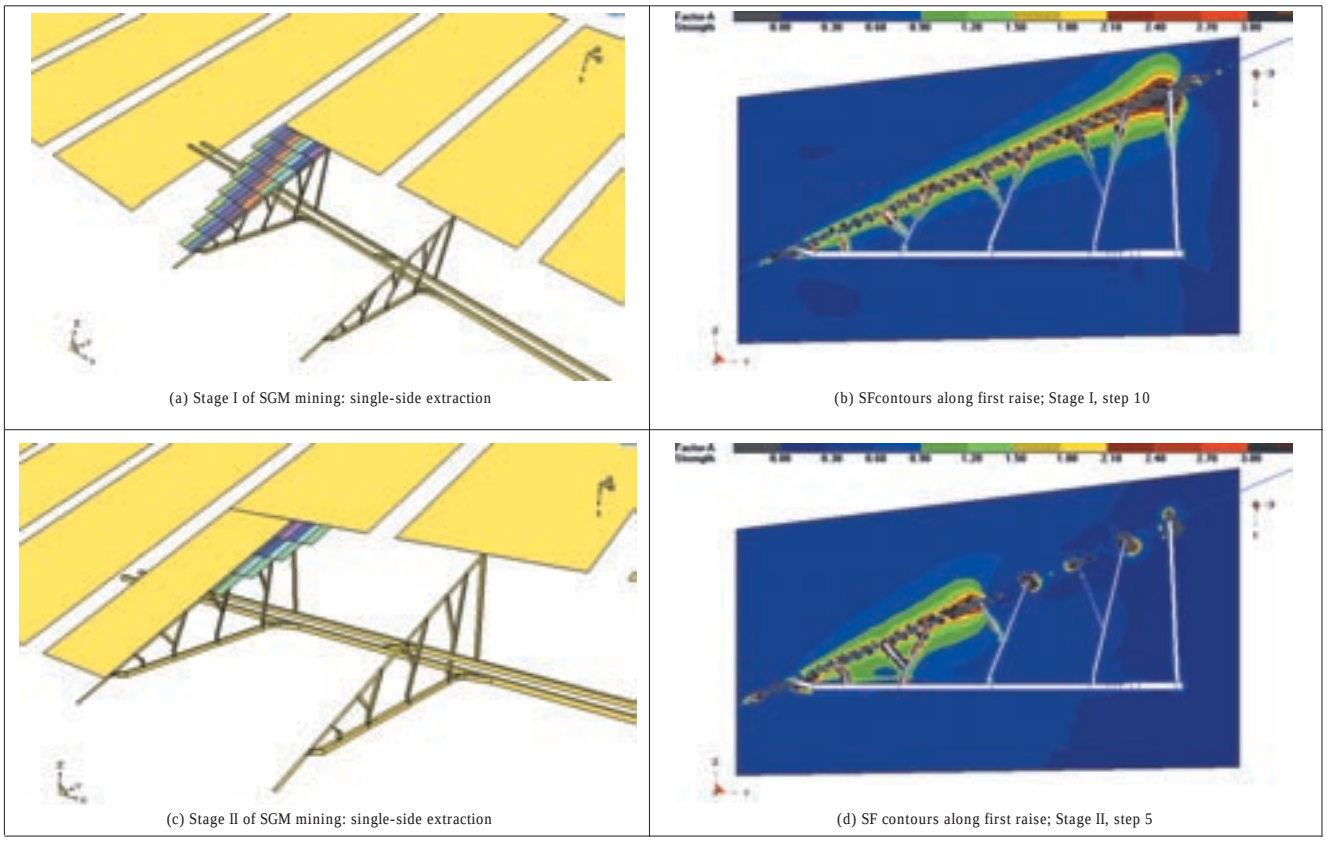
CSDP breast mining sequences, with overall up-dip advances, also cause unstable conditions in the footwall infrastructure. Stoping commences at a limit depth below surface and progresses up-dip with overall 'inverted v' geometry. The mined-out areas above are distant and have negligible influence on the workings below. Strength factor analysis indicates poor rock mass conditions near reef intersections during early stages of mining, as abutment-like loading conditions prevail in these areas at this time (Figure 13a 13b). The 40 m wide dip-stabilizing pillars, spaced 140 m apart, only slightly reduce abnormal stress concentrations along raises. Bottom orepasses, in particular, are subjected to a cycle of high stress concentration followed by stress relaxation and stress reversals. As in the other grid layouts, induced field stresses change from compressive to tensile once panels pass over the tip area, causing unconfined rockwalls to dilate further. Failure of bottom orepasses ceases to be a problem when the top orepasses become available to accommodate the production output. Early in the mining of CSDP stopes, the top orepasses appear relatively stable.



(a) Step 12

(b) *SF* contours along first raise; Step 12

Figure 10—Growth of the failure zone beneath LSP stopes, reported on a vertical plane orientated oblique to dip in order to pass through as many orepasses as possible. Stoping at average depth of 4000 m

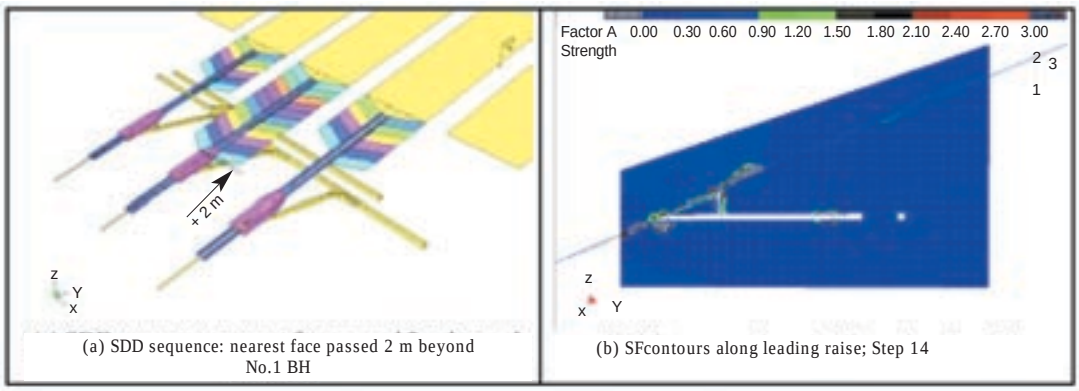

(a) Stage I of SGM mining: single-side extraction

(b) SFcontours along first raise; Stage I, step 10

(c) Stage II of SGM mining: single-side extraction

(d) SF contours along first raise; Stage II, step 5

Figure 11—(a, b) Failure zone beneath SGM single-side stopes at average depth of 4000 m, reported on a vertical plane that passes through all footwall excavations beneath the raise (c, d) Elimination of the failure zone after Stage II mining



(a) SDD sequence: nearest face passed 2 m beyond No.1 BH

(b) SFcontours along leading raise; Step 14

Figure 12—Growth of the failure zone beneath SDD down-dip mined stopes, reported on a vertical plane that passes through the footwall excavations

As the advancing stopes approach the old upper level workings, the top rockpasses are increasingly affected by high stress concentrations (compare Figure 13 c, d e, f and g, h). The top rockpass is the most vital infrastructure in a CSDP raise as it serves the continuous ore-handling scraper system. Stresses in this rockpass become highly compressive as the stope faces approach, and reach a maximum value when the face position coincides with the rockpass perimeter. The stresses relax once the face passes the rockpass. Tensile failure zones may develop around walls of the second rockpasses, to a depth of about 4 to 6 m from reef, when 55 per cent of raise-line extraction is achieved. Under these conditions, the rockpass is expected to hang up at some stage of its life owing to sidewall spalling. In this event, the entire CSDP stope line, comprising 16 panels, eight on either side of the raise, could become inoperative.

The failure of the upper-level travelling way also presents a serious problem for the egress of men and materials, and the control of ventilation and cooling. With only one main orepass to operate, the CSDP ultra-deep stope will be more vulnerable than other grid layout stopes to the impact of the mining sequence.

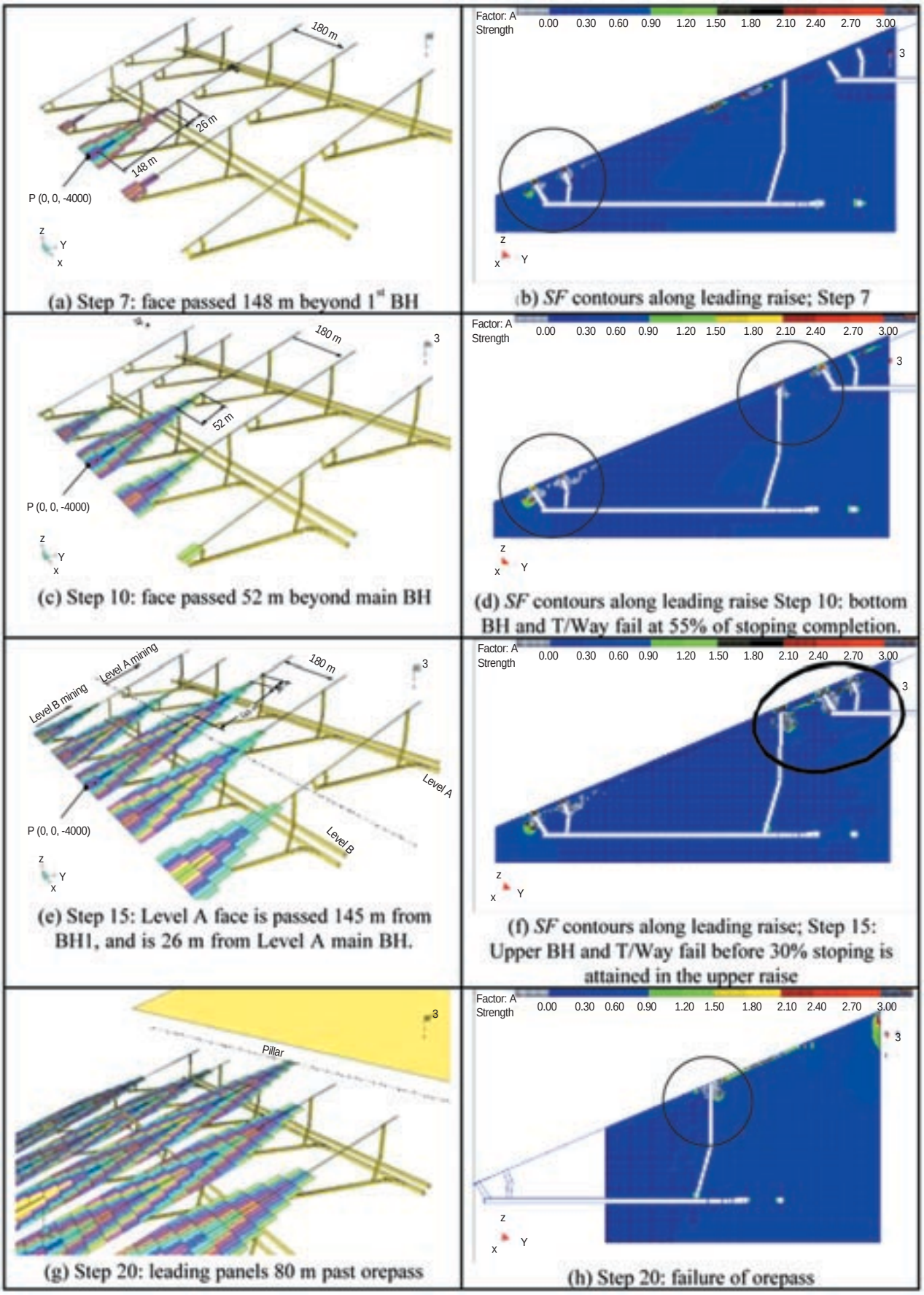


Figure 13—CSDP from a limit depth of 4000 m (a, b) early stoping, (c,d) faces approach main rockpass, (e,f) completion of lower-level stoping, upper-level infrastructure fails in tension (g, h) stoping approaches a strike-boundary pillar that separates current mining from old mined-out areas

Conclusions

The studies conducted under the auspices of the DEEPMINE Programme concluded that it is feasible to excavate and support shaft and stope rockpasses at depths as great as 5 km, unless particularly adverse rock conditions are encountered. Numerical modelling has been used to devise practical approaches to reduce the risk of rockpass failure, by adapting mining layouts and sequences. The stability of rockpasses is dependent on the overall layout design. Currently available support units and support are capable of supporting rockpasses at ultra-depth.

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