Design concepts and support considerations for highly stressed tunnels in deep hard-rock mines

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Synopsis

The paper describes how stress analyses were used to show the effect of a tunnel's shape on the overstressing potential of its periphery. The best profile was found to be that of a horizontal ellipse, for which there was little significant scaling, permitting temporary support to be incorporated into a permanent tunnel-support system.

This investigation was followed by the modelling of several support systems for tunnels with a horizontal-ellipse profile, which showed that such a profile can reduce the installed support required for a tunnel of different cross-section by an average of 57 per cent.

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 SA ISSN 0038–223X/3.00 + 0.00. Received Sep. 1993; revised paper received Jan. 1994.

Introduction

Under highly stressed conditions, tunnelling in hard-rock mines becomes slow and difficult owing mainly to support considerations. Based on Spearing's preliminary investigations[1] into the need for high and sustained rates of tunnel advance, and on further analysis of the problems, design concepts and support systems have been devised for tunnels in high-stress conditions (or at depths in excess of 4000 m below surface). On any specific new or existing mine, the cost implications of slow tunnel development is highly significant because it has a direct effect on the establishment of new or replacement ore reserves.

Intrinsically Stable Tunnel Profiles

Under a high-stress regime in hard rock, the problem of support is exacerbated in tunnels of rectangular profile because the rock around the periphery of the tunnel has already been overstressed beyond its elastic limit (i.e. significant failure has occurred). This results in intense sloughing (scaling) of the sidewall and occasional collapses in the tunnel roof, making it virtually impossible to combine temporary support with the permanent support system. The main problem is that, when permanent support is to be installed, the temporary rockstuds usually protrude from the periphery of the excavation because of the significant scaling (caused by rock failure) mainly of the sidewalls. This is shown in Figures 1 and 2.

Method of Investigation

Stress analyses were carried out to show the effect of a tunnel's shape on the overstressing potential of its periphery. For domains of both elastic and inelastic rock, the following shapes of excavation were considered:

(1) circle with a diameter of 4,2 m
(2) square with sides of 3,7 m
(3) rectangle with sides 3,5 by 4,0 m wide
(4) vertical ellipse 5,1 m high and 3,5 m wide
(5) horizontal ellipse 3,5 m high and 5,1 m wide.

It should be noted that all these shapes give a cross-sectional area of 14 m^2. This is required on deep-level hard-rock gold mines in South Africa to ensure adequate access for men, materials, and rock, in addition to sufficient air volume.

The parameters that featured in the study are listed in Addendum 1. In this part of the study, the tunnel and surrounding rock were modelled without any installed support (giving a worst-case scenario).

Results of the Elastic Analysis

By use of the two-dimensional programmes BESOL P5002[2], and FLAC[3] with a linearly elastic constitutive model, the maximum value of vertical stress was found to occur (as expected) tangentially to the tunnel sidewall. The results shown in Table I should be regarded as qualitative, and not quantitative, mainly because of the significant rock failure around the periphery of the tunnel (i.e. inelastic effects).

Based on the elastic results, the variation of vertical stress along the springline (from the middle of the sidewall of each excavation along a horizontal axis) was plotted, as shown in Figure 3. From a consideration of the curve for each profile, the rating from best (most stable) to worst (least stable) in terms of vertical stress is as follows:

square (most stable)
rectangle
vertical ellipse
circle
horizontal ellipse (least stable).

Table I

Maximum vertical stresses (MPa)

Depth into sidewall at which stress is 125 MPa m	Shape	Springline stress MPa
2,9	Circle	225–250
3,5	Square	150
~5,0	Rectangle	150
2,8	Vertical ellipse	175
~5,0	Horizontal ellipse	200–250

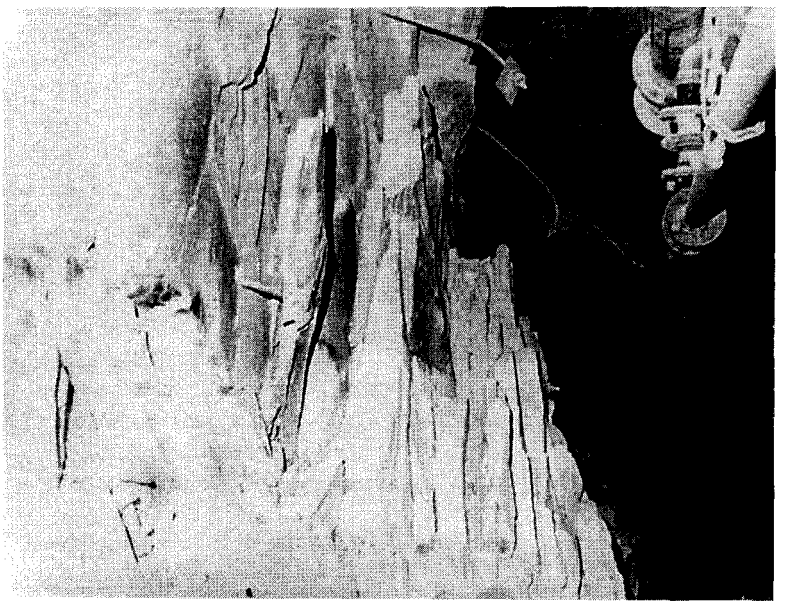

Figure 1—Relatively minor scaling of a sidewall in a tunnel, showing the protruding rockbolts

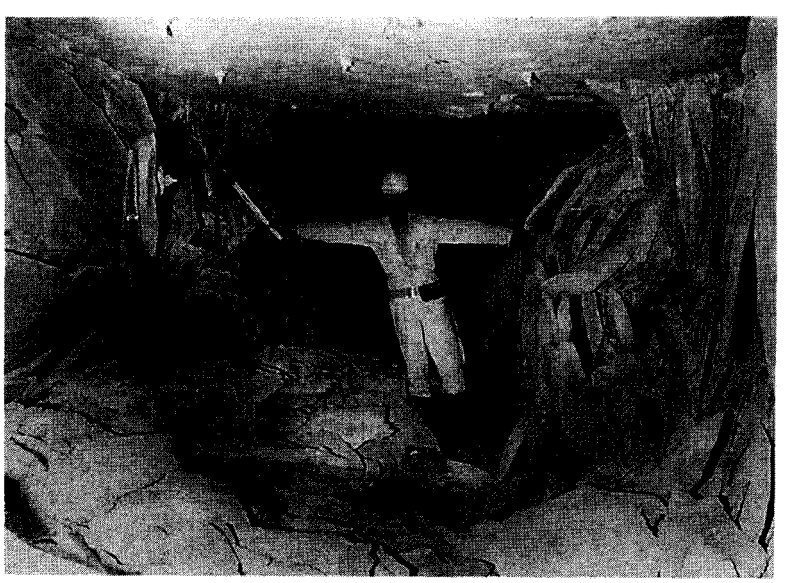

Figure 2—Severe sidewall scaling of a tunnel

Results of the Inelastic Analysis

It is obvious from the nature of the problem (i.e. where significant rock failure occurs) that the elastic solution is not a truly representative model of the conditions. This is mainly because the rock around the excavation has failed. Inelastic modelling was therefore undertaken by use of the two-dimensional non-linear stress analysis program FLAC Version 3,03/3,04[3].

The maximum values of plastic strain, volumetric strain increment, and sidewall displacement are given in Table II. It can be seen from these results that the rating (from most stable to least stable) is totally different from that found in the elastic analysis:

horizontal ellipse (most stable)
circle
square
rectangle
vertical ellipse.

The horizontal ellipse is the most stable, even though the horizontal displacement of the sidewall springline is the most, and the plastic strain and the volumetric strain increment are the least. This implies that only the sidewall is initially relatively mobile. However, owing to the squat nature of the profile, this can be controlled and stabilized. It is evident that, in the inelastic domain, profiles that have a relatively short, squat effective sidewall are more stable.

During the late 1970s at a controlled, instrumented tunnel site at East Rand Proprietary Mines (E.R.P.M.), Ortlepp[4] reported the following.

(i) In the highly stressed portion of the square tunnel, uncontrolled scaling of the sidewalls resulted in a horizontal-ellipse profile that was between one and two times the original cross-sectional area of the blasted square tunnel. This is clearly illustrated in Figure 4.

(ii) Once the profile became a horizontal ellipse, the scaling appeared to stop. This was because the micro-slabbing process was prevented from progressing deeper into the rockmass by the very steep stress gradient and the rapid increase in confinement at the sharp ends of the ellipse.

This practical experience illustrates the necessity for the horizontal-ellipse profile and the fact that, with such a profile, little significant scaling occurs. This lack of scaling would permit the incorporation of temporary support in a permanent tunnel-support system, thus saving considerable time and money.

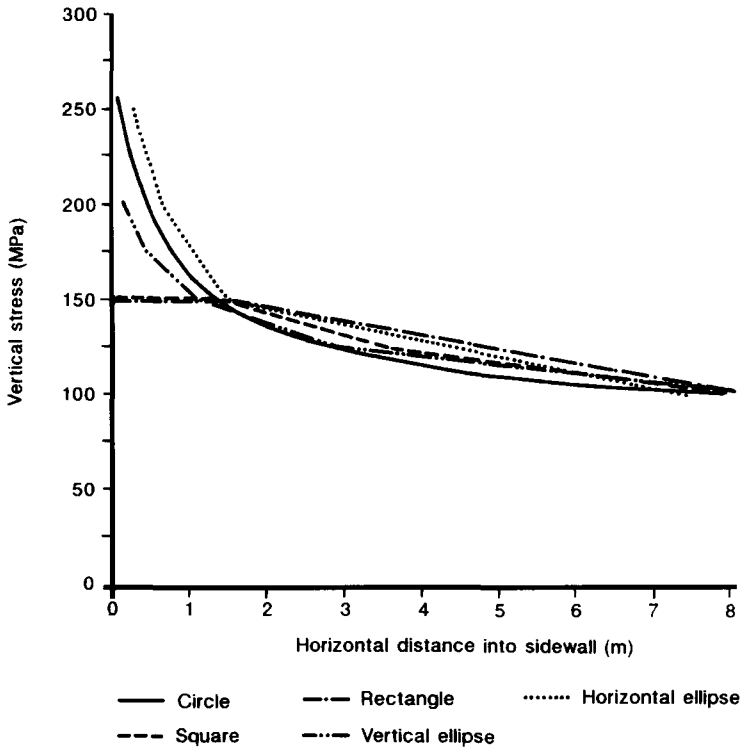

Figure 3—Variation of vertical stress along the springline under elastic conditions

Table II

Inelastic values

Depth of zero volumetric strain increment, m	Shape	Maximum plastic strain	Maximum volumetric strain increment	Horizontal movement of the sidewall springline, mm
4,7	Circle	0,09	0,20	573
6,2	Square	0,10	0,04	676
6,0	Rectangle	0,18	0,15	624
5,3	Vertical ellipse	0,15	0,23	1370
3,7	Horizontal ellipse	0,07	0,02	1886

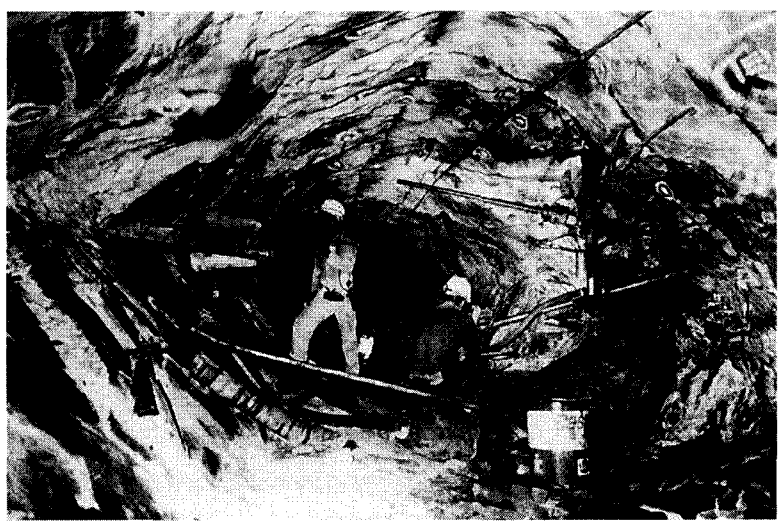

Figure 4—The test tunnel at E.R.P.M. under high vertical stress[4]. It should be noted that the profile was blasted square, but subsequent scaling produced the horizontal-ellipse profile

Support Requirements for a Horizontal Ellipse

Preliminary Investigation

The support requirements were determined by use of an inelastic computer code (FLAC). Two main support elements were modelled: grouted cable bolts and shotcrete.

Support cannot generally be installed on the face immediately after a blast because of the pile of broken rock (particularly significant in the relatively small cross-sectional area of a tunnel). This was simulated in the following sequence, as detailed in Addendum 2: running of the model without support, then installation of the cables and, finally, installation of the shotcrete. In practice, it would be possible to apply some shotcrete soon after the blast (particularly on the top half of the tunnel), and hence the results obtained from the modelling are probably conservative.

The properties of the support elements are also given in Addendum 2. The following series of conditions were simulated (on the assumption of a longitudinal spacing of the cable rings in the tunnel of 1,5 m as recommended by Spearing[1]):

(a) 5 m long locked-coil cable anchors with a shotcrete thickness of 100, 200, or 300 mm
(b) 8 m long locked-coil cable anchors with a shotcrete thickness of 100, 200, or 300 mm
(c) 5 m long strand cable anchors with a shotcrete thickness of 100, 200, or 300 mm.

All the cables were considered to be unbreakable. It was found that a system of cable anchors alone could not stabilize the excavation, and that shotcrete alone would be impracticable since, after the shotcrete had failed, the excavation would collapse. Seismic considerations had also to be considered in the practical support design, and hence yielding of the support units was considered to be essential.

The results are illustrated in Figures 5 to 7. The following conclusions can be drawn.

(1) For any given cable and length, the maximum average stress in the shotcrete decreases with increasing shotcrete thickness. This is in spite of the shotcrete taking additional load with increasing thickness.
(2) The shotcrete stresses are basically unaffected by increased cable length or a change in the type of cable.
(3) Except for the horizontally orientated sidewall cable, the cables are not heavily stressed. The sidewall cable is very heavily stressed, but only over a length of 3 to 4 m. This is because the shotcrete annulus is effectively stiffer and therefore attracts load more quickly than the cable (influenced by the squat profile of the ellipse).

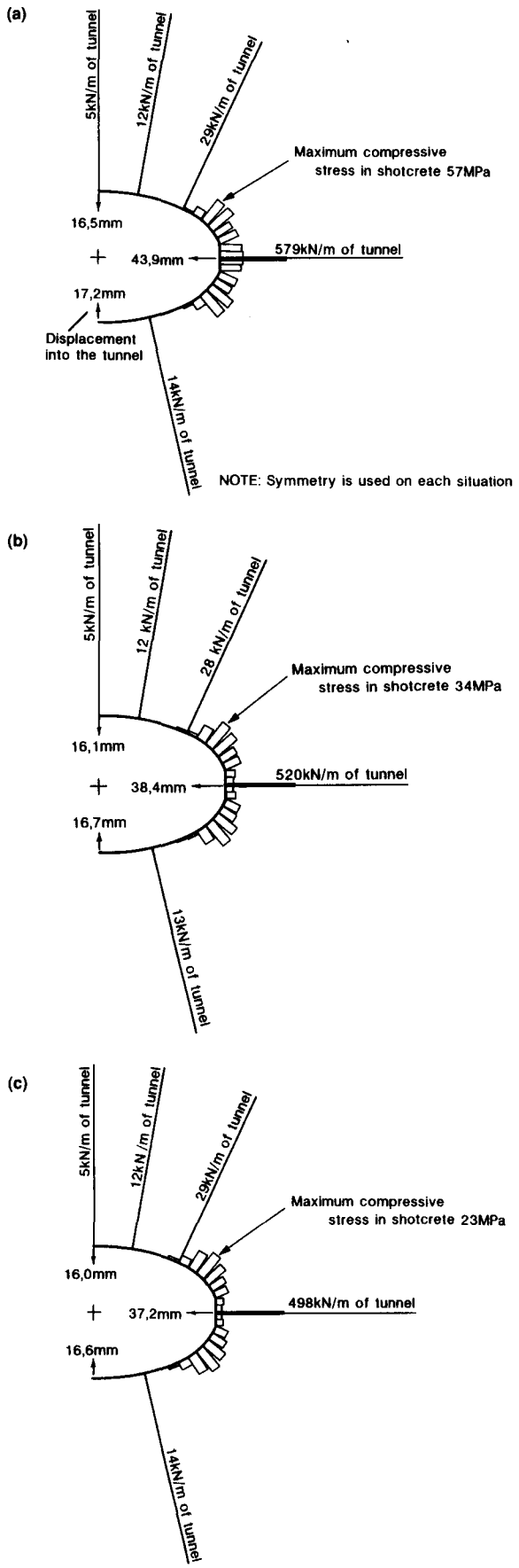

Figure 5—5 m long locked-coil cables with the following thicknesses of shotcrete: (a) 100 mm , (b) 200 mm, (c) 300 mm

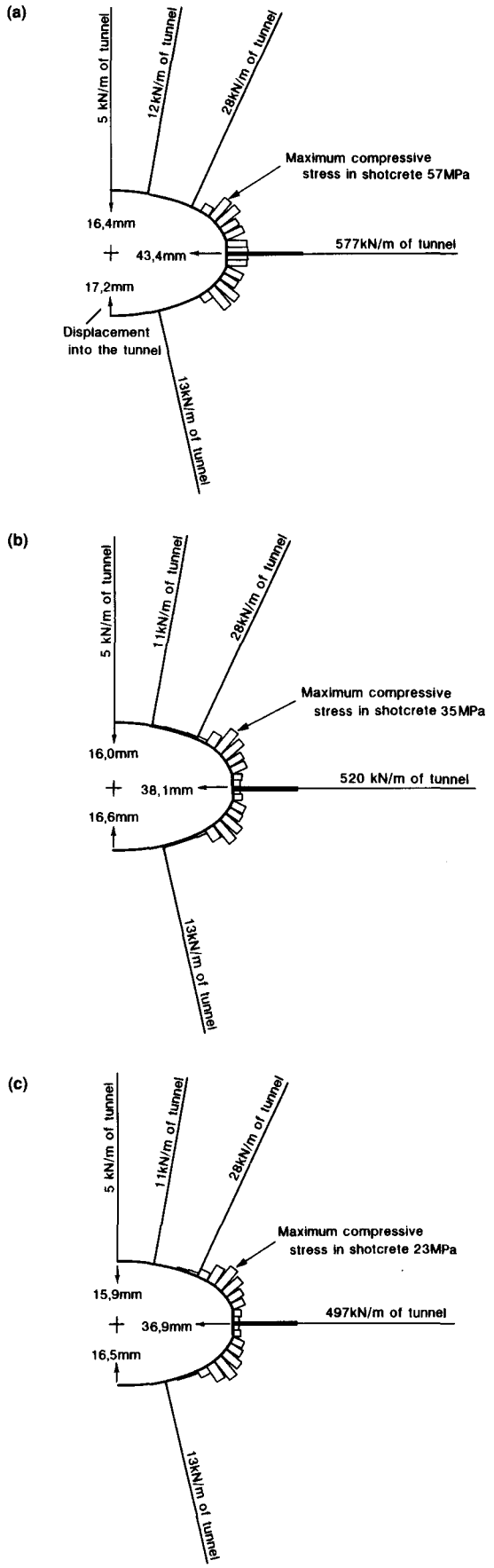

Figure 6—8 m long coiled cables with the following thicknesses of shotcrete: (a) 100 mm, (b) 200 mm, (c) 300 mm

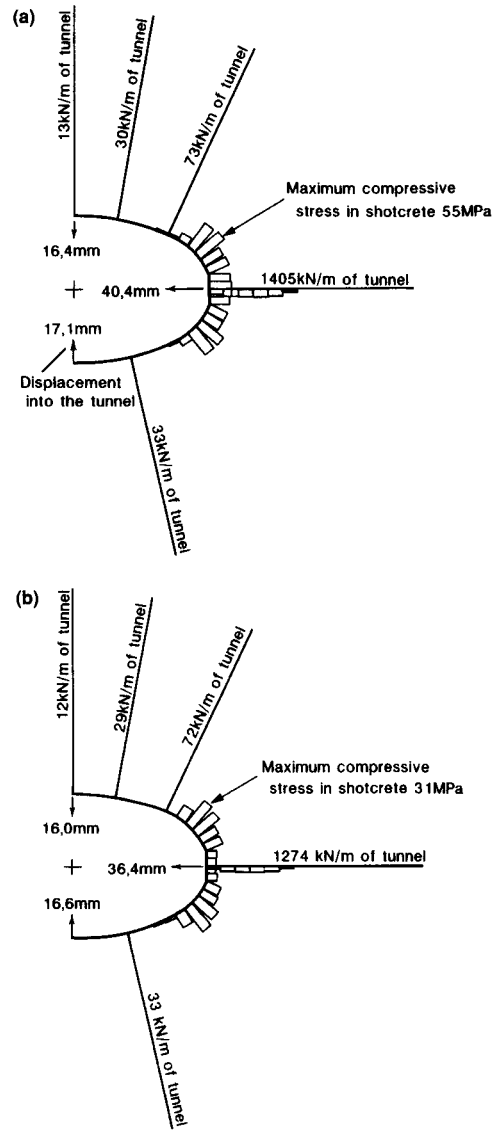

Figure 7—5 m long strand cables with the following thicknesses of shotcrete: (a) 100 mm, (b) 200 mm

(4) The stress in the horizontal sidewall cable is dependent only on the sidewall displacement (which decreases with increasing thickness of shotcrete).

(5) Larger forces are induced in cables of higher modulus (and larger cross-sectional area).

Furthermore, a shotcrete thickness of 100 mm is too thin for practical application in the long term. A thickness of more than 300 mm is probably too large since it would require a larger tunnel profile (to still give the required final area) and more shotcrete material, creating serious logistical problems.

More-detailed Investigation

The preliminary investigation, which considered only shotcrete and cables (unbreakable), showed that the following warranted more analysis and investigation.

(i) In addition to the high compressive stresses generated in the shotcrete, there were unacceptably high shear stresses.

(ii) The large forces generated in the sidewall cables (in excess of 1000 kN per metre of tunnel longitudinally) cannot reasonably be provided and are probably not required if yielding anchors are utilized.

The support systems modelled in this, more-detailed investigation were as follows:

(a) mesh-reinforced 200 mm thick shotcrete with 4 m long untensioned strand cables with a 40 t useful load at a spacing of 1,5 m with face plates

(b) as in (a) but with the cables pre-tensioned to 10 t

(c) as in (a) but using Y25 rebar instead of cables

(d) as in (c) but with the rebars pre-tensioned to 10 t

(e) as in (a) but with 22 mm diameter (19 t yield) cone bolts

(f) as in (a) but using Swellex dowels

(g) as in (a) but using 6 m cables pre-tensioned to 10 t

(h) as in (b) but using fibre-reinforced shotcrete.

The displacements along the horizontal axis of the ellipse (middle of the sidewall) on the perimeter of the tunnel showed the following.

(1) Pre-tensioning to 10 t had no significant effect on the stability of the tunnel.

(2) Stability (of the sidewall) could not be achieved unless support loads in excess of 300 kN per metre of tunnel were provided.

(3) Even mesh- or fibre-reinforced shotcrete is damaged owing to the high dilation, but it is necessary as a support element and the damage will not be catastrophic.

(4) To limit displacement of the tunnel sidewall into the excavation (still on the assumption of 9 cables per ring), the support forces shown in Table III are required.

Table III

Support force requirements

Tunnel displacement mm	Support force (kN/m of tunnel)
400	1000
500	800
600	500

(5) A yield capability of the cable bolt is required, not only in the quasi-static case modelled, but also under the dynamic conditions (seismicity) that exist in a deep hard-rock mine.

Measurements taken by Wojno and Jager[5] indicate that dilation of 250 mm is not uncommon in existing deep tunnels. This tends to reinforce the findings of the present study since larger dilation (displacements) can be expected under the higher stress conditions.

It should be possible to further optimize the layout of cables per ring to reduce the very highly stressed unit in the horizontal sidewall axis. However, caution should be taken not to increase the number of cable bolts per ring since this would severely hamper the production cycle, as indicated by Spearing[1].

Comparison between Horizontal-ellipse and Square Profiles

A change in tunnel profile from the traditional square or rectangular shape to a horizontal ellipse is not always acceptable to production personnel. Therefore, to 'sell' the need for an elliptical profile, a comparison was made between the equivalent support conditions.

Initially, the stress generated in the shotcrete was used in the comparison of equivalent stability, but this was found to be unsatisfactory owing to the different failure mechanisms in the shotcrete around the perimeter of each profile.

After further simulations, it was decided that the equivalent stability criterion should be associated with the horizontal displacement (dilation) of the mid-point of the sidewall of each profile. This was considered acceptable because, as shown by experience, sidewall stability is the largest strata-control problem under high vertical stresses. In addition, access is curtailed with severe dilation. The results of the comparative simulations are given in Table IV.

Table IV

Required sidewall support pressure and forces

Sidewall displacement mm	Square		Ellipse	
	Stabilizing pressure kPa	Total horizontal force kN/m of tunnel	Stabilizing pressure kPa	Total horizontal force kN/m of tunnel
300	2050	7671	1800	4399
400	1600	5967	1000	2444
500	1150	4303	700	1711
600	870	3256	460	1124

The results show that (on average) at least 57 per cent more stabilizing pressure (or horizontal force) is required to limit the dilation to the same magnitude. However, in the comparison of the average horizontal force per metre of longitudinal tunnel, the horizontal ellipse requires 140 per cent less force for equivalent stability. Therefore, by the development of a tunnel with a horizontal-ellipse profile, in place of a traditionally shaped square tunnel, the savings in support cost should be substantial.

The Effect of Overstoping

During the effective life of most long-term tunnels, overstoping will occur. This has the effect initially of increasing the maximum stress acting on a tunnel and also changing its orientation. As overstoping proceeds, the vertical stress on the tunnel decreases to a minimum as de-stressing occurs. The final stress state could approach the virgin stress once total closure in the stope above had occurred (i.e. the mining span becomes large).

The conditions modelled (Figure 8) were chosen because the 80 m wide pillar above the tunnel gave the highest stress field in the tunnel. A smaller pillar tended (with the rockmass parameters modelled) to exhibit significant degrees of pillar failure and, hence, associated load shedding. The totally overstoped scenario was modelled to show the change in orientation of the maximum stress.

The results are given in Table V. The secondary increase in the stress field acting on the tunnel as a result of mining causes further failure around the excavation. The magnitude and extent of the increased failure are proportional to the increase in the field stress.

The results of the simulations indicate that maximum secondary failure occurs at a depth of about 100 m below the stoping. This result coincides with a high vertical stress and a relatively low k ratio (i.e. the horizontal stress is relatively low) as indicated in Table V.

It is thought that this reduced horizontal confining stress results in a lower sidewall confining stress and, hence, an increase in sidewall failure. Once the tunnel has been totally overstoped and, hence, largely destressed, additional failure occurs as a result of the change in orientation of the maximum stress (principal stress), which becomes sub-horizontal.

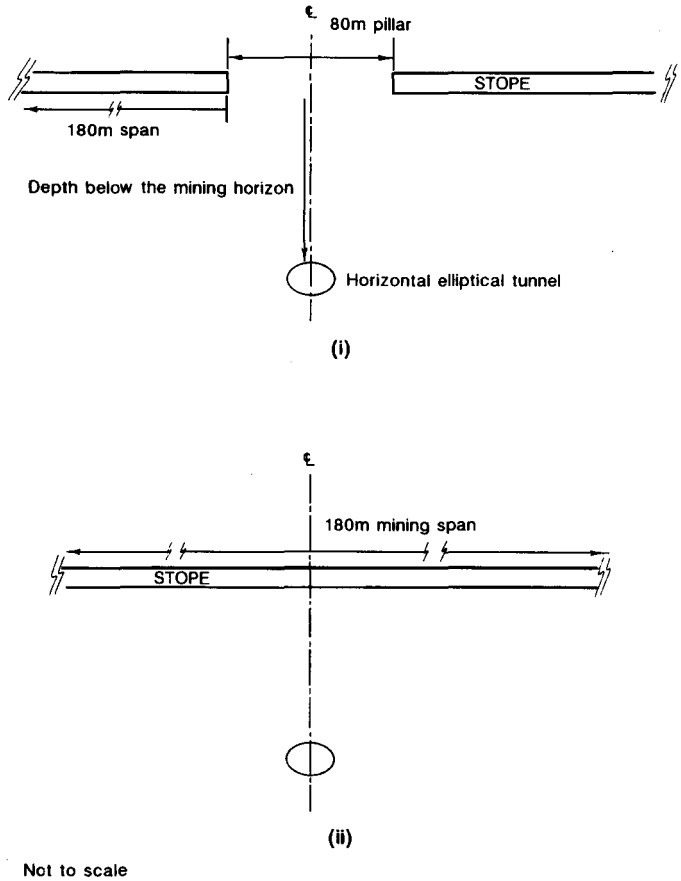

Figure 8—The overstoping configurations modelled

TableV

Results of overstoping on the stress regime of the tunnel

Modelling condition	Depth below stoping horizon m	Vertical stress* MPa	Horizontal stress† MPa
80 m pillar	60	145	50
	80	130	40
	100	120	30
	120	105	35
Totally overstoped	60	0	30
	80	10	40
	100	20	50
	120	25	50

* Virgin stress 104 MPa
† Virgin stress 52 MPa

An examination of the cumulative dilation (displacement) caused by the stress increase and subsequent decrease (after overstoping) showed that the most extensive damage occurs 100 m below the reef elevation. However, the total dilation decreases substantially if the tunnel is located 120 m (or more) below the reef plane.

Conclusions and Recommendations

Although all the analyses were limited to a specific homogeneous strain-softening quartzite[6], it is believed that valid conclusions can still be drawn. These conclusions obviously require to be fully validated by underground trials and further calibration (from underground measurements), but still indicate important results in maximum field stresses of up to 150 MPa.

(1) A horizontally elliptical profile should be adopted since this significantly reduces (by an average of 57 per cent) the installed support required to stabilize a tunnel.

(2) Any blast damage to the periphery of a tunnel will obviously increase dilation and, hence, some form of smooth-wall blasting (pre- or post-splitting) would be advantageous.

(3) A sidewall dilation of 300 to 500 mm should be expected at great depth in tunnels, and allowance should be made for this when the final dimensions are selected for a tunnel. In addition, allowance should be made for at least 200 mm of shotcrete.

(4) Primary shotcrete (50 to 100 mm thick) should be applied as soon after the development blast as possible to prevent initial deterioration and also weathering of the rock (if applicable).

(5) Nine cables (in the pattern indicated by Spearing[1]) 4,0 m long should be installed in each support ring. The capacity of each cable should be 80 t if the rings are 1,0 m apart, and 120 t if the ring spacing is 1,5 m. Adequately robust, large face plates are needed. The cable bolts should be cone cables as designed by Wojno[7], and should have a minimum yield in excess of 600 mm. This is because the expected dilation is about 500 mm in the sidewalls, and any additional dilation as a result of seismicity must also be accommodated.

(6) Secondary shotcrete should be applied at some convenient distance from the advancing tunnel face. The reasons for its installation distance from the face are as follows.

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- It will not unduly hinder the production cycle.
- Much of the tunnel dilation will have already occurred, and hence the extent of the expected failure of the secondary shotcrete will be reduced.
- It can be used to repair the primary shotcrete, which can be expected to be severely damaged.

The secondary shotcrete should be reinforced with steel fibre to improve the shear and flexural strength of the material. The use of mesh would also be acceptable, but it may prove too labour-intensive. The final thickness of the shotcrete should be in excess of 200 mm.

(7) During the useful life of the tunnel, remedial measures may still be necessary to repair the permanent support. This will mainly include the re-application of portions of the shotcrete (particularly after overstoping or major seismicity in the vicinity).

Future Research and Investigations

Based on the research and analyses conducted to date, the following should still be undertaken.

- Further computer analysis is required so that the orientation of the sidewall cables and the number of cables per metre of tunnel advance can be optimized. This will necessitate an investigation of the cycle times for the various options (which are critical to minimize interference with the production cycle), together with the support cost of each option.
- The effect of the rockmass not being homogeneous (i.e. without geological influences) needs to be investigated, especially the effect of tunnels developed perpendicular or parallel to the strike of the orebody.
- Suitably designed cone-cable anchors should be tested in the laboratory to ensure that the tonnage and yield, as specified in this paper, can be achieved, and to establish the unit cost of such anchors and ancillaries.
- An underground highly stressed tunnel site should be established for investigations of the stability of a horizontal ellipse and the suitability of the support system recommended, and for back-analyses against the computer simulations so that a more accurate model can be established.
- Site-specific modelling should be undertaken to calibrate the model for the particular conditions.

Addendum 1: Parameters Used in the Computer Analyses

Vertical stress = 104 MPa
Horizontal stress = 52 MPa
Rock density = 2 700 kg/m^3
Bulk modulus = 33 GPa
Shear modulus = 29 GPa

The strain-softening model listed in Table AI was used (for a typical pebble quarzite as determined by Kersten[6]) for the inelastic modelling.

Addendum 2: Properties of the Support Elements Modelled

Support Installation

The following sequence was used in the modelling of the support installation in the tunnel by the use of FLAC. This method was found to be reliable following back-analysis by Speers.

- Run the model for 400 time steps, keeping the rock elastic (to allow some initial re-distribution of stress without allowing failure, particularly tensile failure).
- Lower the friction angle, cohesion, and strength to the initial strain-softening parameters for the rockmass.
- Run the model plastically for a further 200 time steps to simulate the initial inelastic deformations that take place before support can be installed.
- Install the cable bolts in the model and run for a further 200 time steps.
- Install the shotcrete in the model and run to stability (about 3000 time steps).

This modelling sequence tends to overcome some of the problems that arise in the representation of a three-dimensional problem in two dimensions.

Cable Bolts

Untensioned cement-grouted cables:

Shear bond stiffness 10 GPa

Two different types of cables were modelled:

- Locked-coil cable (as in the FLAC manual)
  Young's modulus 98 GPa
  Cross-sectional area 500 mm^2
- Strand cable
  Young's modulus 180 GPa
  Cross-sectional area 694 mm^2.
  In all the computer-modelling runs, nine cables per ring (as recommended by Spearing[1]) were installed at a longitudinal spacing of 1,5 m.

Table A1

Rockmass properties

Strain ε	Cohesion (Co) MPa	Friction degree	Dilation degree	UCS mass* (σ) MPa
0	17,5	27,5	0	57,7
0,001	16,0	32,5	20,0	58,3
0,002	13,5	32,5	20,0	49,2
0,004	11,0	32,5	20,0	40,1
0,006	7,5	32,5	20,0	27,3
0,008	4,0	32,5	6,5	14,6
0,010	0,5	32,5	3,5	1,8
0,015	0,5	32,5	5,5	1,8
0,100	0,5	32,5	5,5	1,8

* The unconfined compressive stress (UCS) of the overall rockmass is calculated as follows:

$$\text{UCS mass} = \frac{2\,Co \times \cos\phi}{(1-\sin\phi)}.$$

Shotcrete

Typically, shotcrete with an unconfined compressive stress of 30 MPa was used. Generally, such shotcrete should have a Young's modulus of about 28 GPa but, because (in this application) the shotcrete is loaded prematurely and substantial creep would occur during curing, a de-rated modulus of only 12 GPa was used in the analysis. ◆

First full-time director of Miningtek

Dr Johann Fritz has been appointed by the CSIR, in conjunction with the Research Liaison Committee of the Chamber of Mines, as the first full-time Director of the CSIR's Division of Mining Technology (Miningtek), formerly COMRO, the Chamber of Mines Research Organization. This appointment took effect from 1st April, 1994.

Dr Fritz (46) has had a long association with the mining industry, including, in the early 1980s, running the CSIR's Research Unit for Mining Equipment, which grew substantially under his leadership. He has been involved in a wide range of mining-equipment projects covering all aspects of mine ropes such as design, manufacturing techniques, and in-service behaviour. He was also actively involved in an extensive enquiry into the validity of the present regulations concerning safety factors of mine hoisting cables, on which he is an acknowledged authority.

Starting off his career with the CSIR in 1967, Dr Fritz was appointed Director of the Division of Aeronautical Systems Technology in 1987 and, after the amalgamation of that Division with the Division of Production Technology in 1992, he was appointed to the position of Director of the Division of Manufacturing and Aeronautical Systems Technology (Aerotek) of the CSIR.

Dr Fritz received both his honours degree in mechanical engineering and his doctorate in engineering from the University of Pretoria, and he is a registered professional engineer. He is married, with three children. ◆

Report on MINEFILL 93

by J.M. Stewart*

The Fifth International Symposium on Mining with Backfill, held in Johannesburg in October 1993, brought together, in one forum, expertise in backfilling from across the world, with 222 delegates from 17 different countries attending.

The main theme of the Symposium, the systems nature of backfilling operations, provided a holistic view of backfilling: how it satisfies a wide variety of mining requirements, as well as the installation and operation of a backfilling system, and how this affects personnel from the majority of disciplines on a mine. The proceedings were arranged in three major themes, taking place on each of the three days of the Symposium.

The themes covered backfill–rock mass interaction, backfill systems engineering, and backfilling operations. These themes reflected, not only the systems nature of backfilling, but also the balance of research, innovation, and practical application that is currently occurring in the field of backfilling. The 53 papers presented at the Symposium made a variety of information available, from fundamental knowledge generated by laboratory experimentation, and mechanistic models for use as design tools, to guidelines based on empirical investigations and practical experience of backfill systems.

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Backfill–Rockmass Interaction

In the area of backfill–rockmass interaction, useful guidelines were presented for the selection of backfill materials with appropriate properties to satisfy the rock-engineering requirements. By consideration of the variables to describe the mining situation, the guidelines would be used to specify appropriate materials and placement strategies, and would contribute significantly to overcoming the problem of implementing complicated technologies.

The numerical modelling of backfill and rockmass behaviour ranged from the evaluation of fundamental mathematical or constitutive relationships to several case studies on the utilization of backfill as a support medium. The complex, specialized nature of modelling means that there is scope for informal international co-operation.

A key feature in this area was the assessment of the economic benefits to be achieved through the improved support characteristics of backfill. Although the benefits are strongly case-specific, these include enhanced exploitation of the orebody through pillar extraction, and the use of backfill as a prerequisite to the establishment of mining operations.

Backfill Systems Engineering

In terms of the engineering problems associated with backfilling systems, a number of technologies, although not all yet evaluated, were proposed to overcome some of the major outstanding technological challenges. The removal of ultra-fine material and the control of backfill preparation through accurate particle sizing were key topics. Although in terms of the benefits of improved support through backfill, their importance should not be under-estimated, neither should the potential problems of handling fines on surface. In backfill placement, the selection of geofabrics to achieve satisfactory performance at reduced cost was also noteworthy.

Another two fundamental issues addressed were those associated with the transportation of backfill from the preparation plant to the paddock.

Firstly, new and improved models for predicting pressure losses in backfill pipelines for full-flow regimes have the potential to facilitate the design of efficient and cost-effective backfill-distribution systems. These studies range from empirical laboratory investigations of flow behaviour to mechanistic modelling approaches, and computer-aided design tools for specifying the configurations of backfill piping systems.

Secondly, pipe wear in the free-fall zone is a major single contributor to maintenance costs, with the time wastage in the replacement of pipe sections representing a major indirect cost in terms of shaft downtime. Potential solutions include alternative piping materials and wear-resistant linings, which appear to offer considerable scope in reducing costs; system designs to avoid the free-fall zone; and an- emphasis on correct installation procedures.

Backfilling Operations

The opportunity for backfill practitioners from around the world, including Canada, the USA, Germany, Japan, Slovenia, and Australia, as well as a large number of case studies from South Africa, to share their practical experience provided exposure to a wide range of backfilling systems and applications, and yielded a number of important issues, one being the wide variety of materials being used in backfilling operations. Depending on the application and the availability of material, the various mines are engineering suitable materials to minimize costs while achieving the necessary benefits. A complete range from waste rock to classified tailings or paste is being used, with or without the addition of a wide variety of cementitious binders, and is indicative of the considerable understanding of backfill materials and backfill–rockmass interactions that is now available.

A notable aspect of this discussion, however, was the concentration on the positive aspects of backfill implementation with, possibly, disproportionately few references being made to practical difficulties or mistakes. In this respect, data presented at the Symposium showed that South African mines have an installed backfill capacity of 8 Mt per annum, but that they are placing only about 4 Mt. Also, in 1989 placement rates were predicted well above those which have been achieved. Reasons for the current shortfall include the tight economic climate, with mines making every effort to contain costs. This emphasizes the need to convincingly expose the full range of backfill benefits, while another important factor is the difficulty of arranging effective multi-disciplinary involvement in, and commitment to, the operation and maintenance of backfill systems. To overcome these difficulties, there is a major role for appropriate training and awareness programmes and effective communications systems.

Concluding Remarks

In conclusion, it was agreed that the challenge of those who attend the Symposium was to identify how best to implement the new knowledge that the Symposium had contributed to the field of backfill technology. The progress from the Fourth Symposium showed that several topics, addressed by papers describing theoretical or laboratory investigations in 1989, were addressed through papers describing practical applications in 1993, providing evidence that backfill practitioners have been active in applying the information presented in the previous symposium. ◆

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The volume of the proceedings of the Fifth International Symposium on Mining with Backfill (MINEFILL 93) is available as Symposium Series 13 from The South African Institute of Mining and Metallurgy.