Discussion

Highly stressed tunnels in deep hard-rock mines

by A.J.S. Spearing, C.R. Speers, and M. Forsyth*

Contribution by W.D. Ortlepp†

The authors are to be commended for publishing their interesting view of an aspect of deep-level mining technology that is becoming increasingly topical and vitally important.

This is the second paper in the *Journal* within four months to deal with the crucial needs of highly stressed tunnel construction, the other being the paper on the design and testing of rockburst support in the February 1994 issue. Both papers re-examine important innovative ideas that are more than a quarter of a century old, but that have not yet been implemented by the industry. It is to be hoped that they will now, taken together, serve as a new starting point for a definitive examination of what constitutes proper tunnel-support design. There can be no question that the need to develop a design approach that will solve the problems of tunnelling at depths of 4000 m or more has now become crucial.

Support and overbreak

The two most important problems that seriously impede the rate of advance in highly stressed tunnels are difficulties in the installation of suitable support and in the containment of 'overbreak'.

The authors direct most of their efforts towards the theoretical aspects of determining what is suitable support. In discussing the scaling problem, they refer to an earlier study on ERPM, where the original cross-section of the square tunnel degenerated to a horizontal ellipse of 'between one and two times the original cross-sectional area of the blasted square tunnel'. In fact, the tunnel was not square but was carefully blasted to a D-shape, and the overbreak due to scaling varied between 1,6 and 2,2 times the original area. Unfortunately, logistical problems prevented the use of shotcrete at the time.

Clearly, the first need is to find some practical way of containing or controlling the micro-slabbing and extension fracture/buckling processes that caused these unacceptable amounts of overbreak. Without containment, it becomes impossible to install effective early support.

Intuitively it is felt that some kind of tenacious, deformable, sprayable 'caulking' material would perhaps arrest the process in its early stages.

Normal 'strong' shotcrete appears to be sufficiently stiff to attract enough stress to cause the shotcrete to flake off and fail in a way similar to that of the rock. So, while normal shotcrete may be very useful as an immediate temporary solution, there is a need to find ways of improving its ductility.

A formalized study of recent developments in fibre reinforcement of shotcrete was recently initiated by Steffen, Robertson & Kirsten Inc. to determine whether improved compliance, with reduced modulus and adequate flexural strength, can be obtained from sprayed cementitious materials. The authors' recommendation that the shotcrete should be 200 mm thick must be questioned, particularly if it is based on the assertion that the modulus should be de-rated to 12 GPa because of 'substantial creep during curing'. For the same reason, the strength would also be reduced!

Numerical modelling

The detailed mechanism of the fracture of brittle rock is a complicated process, and understanding is often totally absent or incomplete. This lack of understanding should not inhibit visionary thinking or the development of analytical approaches such as the authors have brought to bear, particularly when practical solutions are so urgently required. However, there is a tendency among some modern geotechnologists to accept the results of numerical modelling uncritically.

As Stacey warned in a contribution published in the August 1991 issue of this *Journal,* to a paper by Gürtunca and Adams (*J. S. Afr. Inst. Min. Metall.*, vol. 91, no. 3), 'the development of a new sophisticated computer model will not solve the industry's problems ... what is required is *greater engineering understanding and interpretation* ... and less blind acceptance of computer output, i.e. more engineering in rock engineering'.

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Moreover, it must be stressed that inelastic numerical modelling, in particular, will inevitably give invalid results unless it simulates the prevailing mechanism or physical process that actually takes place.

Therefore, the authors should present their results with greater caution and with more care in making reservations lest some of their numerical solutions are accorded greater credibility than is warranted, and some of their conclusions, stated or implied, are taken as realistic or correct in an absolute sense.

There are several instances in the paper where misunderstanding or misconceptions could arise for these reasons or possibly from the authors' lack of care in expanding the initial thought through a proper development of rationale into a clear statement of the design procedure.

For example, questions and misgivings might arise from a consideration of the analytical results in Table II. It is not clear which set of parameters determines the rating order given in the text. The rank order does not correlate with the numerical-value order of any the four parameters listed in Table II. Whilst it is not difficult to accept from N.G.W. Cook's original treatment in the late 1960's that the horizontal ellipse *is* the most desirable shape, it is not easy to understand from the authors' explanation why this should be so. Also, it is difficult to comprehend how the very large sidewall displacement of nearly 2 m can occur when the value of volume strain is the lowest of all the shapes. Is this merely an anomalous artifact of numerical instability, or does it purport to be an expression of 'real' behaviour? If the latter is the case, it raises doubts as to the validity of the whole analysis.

Despite this uncertainty, the authors go on to examine, in considerable detail, variations in cable type, length, and stiffness and variations in shotcrete thickness. Further analyses explore the effects of replacing the cables with very stiff rebar, with yielding bolts, and with Swellex dowels, and of reinforcing the shotcrete. Interestingly, the rather sparse circumferential spacing between tendons is not varied.

The results of only the first set of analyses are presented, and Figures 5, 6, and 7 show no significant difference between any of the eight cases other than that an increase in the thickness of the shotcrete lowers its stress.

The aspects of yieldability is also given insufficient attention in the overall rationale underlying the authors' treatment of the whole problem of support design. For example, what would happen to the sidewall stability if the stiff support elements (e.g. the cable bolts or 25 mm Y-bars) were really subjected to the unrealistically large unit forces and Table IV, when they would, inevitably and abruptly, break?

Questions like this cannot be examined by numerical analyses, even when the rock behavioural parameters are correctly estimated, if the actual mechanism of failure is not modelled. This is unfortunate because it is the effect of *yielding* (as the authors confirm in some parts of the text) that is vitally important in the consideration of how highly stressed tunnels will behave in reality.

Stability

It is felt that the main omission in the authors' overall treatment lies in their failure to define *stability* and to develop the rationale that relates the load resistance and the bolt spacing to the development of that stability. There is also the strong probability that, because the actual mechanisms of the fracture/failure/dilatation processes are not modelled, the numerical values yielded by the inelastic modelling procedures are not valid or realistic.

Then, the use of unrealistic values (the horizontal displacement at the mid-point of the sidewall) as the criterion for relative stability has to be questioned. It is no better than the criterion based on the stress in the shotcrete, which the authors rejected because of its 'different failure mechanisms'. There was no indication anywhere else in the paper that failure mechanisms had been modelled or even considered.

For the same reason, the central recommendation of a particular pattern of nine anchors with specific load capability should not be advocated so insistently. Why should the anchors in the roof, for example, have a capacity of 80 t (Conclusion 5) when the loads induced in them vary between 0,5 t and 3,0 t (Figures 5 and 6)?

Purpose of this contribution

It has to be emphasized that this contribution is not directed against the concept of the horizontal ellipse or of numerical modelling as such. On the contrary, it is intuitively felt that this shape may well be the basis of the eventual optimum solution. It is felt, however, that more careful thought must be given to the formulation of the physics of the problem, the development of the design rationale, and the selection and validation of the numerical-modelling procedure.

While it may appear contentious, this contribution is intended to be constructive and is offered in the hope that it will stimulate ongoing debate that will eventually lead to greater understanding. At the same time, it completely supports the thrust to get practical solutions tested and implemented as soon as possible.

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Contribution by A.M.J. Vervoort*

Although the overall paper is very interesting and the future research (comparison between numerical simulations and underground measurements) will be very challenging, I would like to make some remarks regarding the set-up of the numerical model and the interpretation of the elastic results.

Set-up of Numerical Model (Elastic Analysis)

From the paper, it is not obvious whether the elastic results for a square and a rectangular opening are based on BESOL or FLAC models. But, independently of this information, the results for these openings are relatively inaccurate, which has, as will be shown, an effect on the comparison between the various shapes (e.g. Table I).

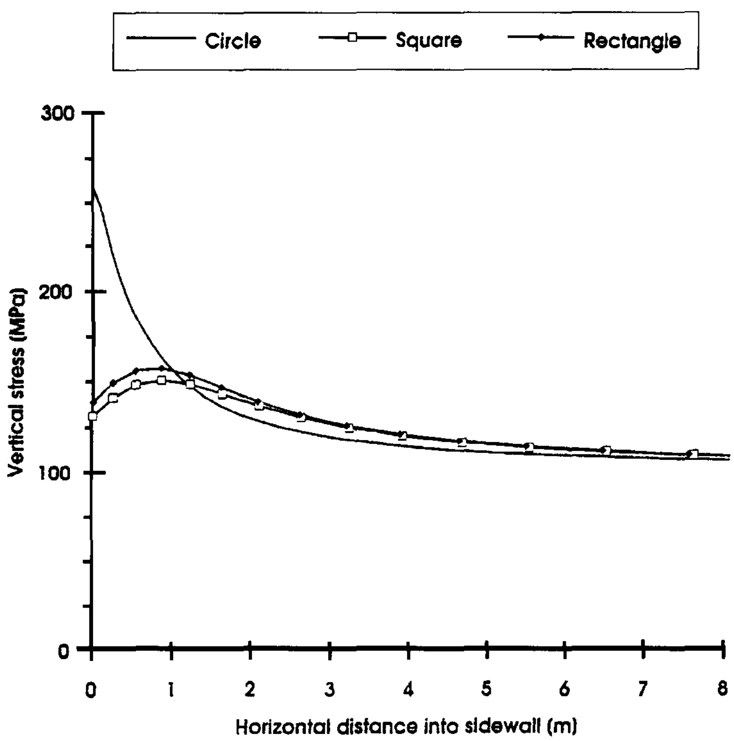

Figure 1—(replacing Figure 3 of the original paper): Variation of vertical stress along the springline under elastic conditions

Table I

Maximum vertical stresses (MPa)

Depth into sidewall at which stress is 125 MPa		Springline stress, MPa	
m	Shape	Circumference	Maximum
2,37	Circle	260	260
3,19	Square	131	151
3,33	Rectangle	139	158

My Figure 1 presents the correct or detailed vertical stress profiles (for a circular, rectangular, and square opening. For the circular opening, as for the elliptical openings, an exact or theoretical elastic solution exists, which one finds in every basic book on the theory of elasticity (as for example, that by R.T. Fenner[1]). The formula for the tangential stress along the horizontal 'axis' for a circular opening is as follows:

$$\sigma_\theta = \frac{\sigma_v}{2}\left[2 + \frac{a^2}{r^2} + 3\frac{a^4}{r^4}\right] + \frac{a_H}{2}\left[\frac{a^2}{r^2} - 3\frac{a^4}{r^4}\right],$$

where: σ_θ = tangential stress
σ_V = vertical stress applied
σ_H = horizontal stress applied
a = radius of circular opening
r = radial distance into sidewall (r = 0: centre of opening).

This results in a maximum tangential stress of 3 * $\sigma_V - \sigma_H$ in the middle of the sidewall at the circumference (3 * 104 – 52 = 260 MPa). The results for the circular opening presented by the authors conform to this formula. However, for the square and rectangular openings, a too coarse mesh was probably used. The profile presented by the authors is characterized by a constant vertical stress close to the opening (over nearly 2 m), while a detailed stress profile shows a maximum of nearly 1 m into the sidewall (see my Figure 1). These results are based on a finite-element model (ANSYS code) with elements 0,25 m wide. That, for rectangular and square openings, the maximum stress is not at the circumference is entirely logical and is due to a type of arch formed by the maximum principal stress. The shape of this arch is a circle or an ellipse, a function of the vertical to horizontal stress ratio. The 'arch' must be entirely situated in the rockmass. So, a circle or ellipse is described round the square or rectangular opening, and the maximum stress is apart from the areas and round the corners further into the rockmass.

For the profiles presented here, I have compiled a new Table I. Comment will be given in the next paragraph on the criterion of the depth into the sidewall at which the vertical stress has dropped to 125 MPa.

Reference

1. FENNER, R.T. *Engineering elasticity.* Ellis Horwood Ltd, 1986.

Before discussing the interpretation of the elastic results, I would like to point out that all the calculated stresses in an elastic model are independent of the elastic constants (shear and bulk modulus), given in Addendum 1. Also, if one considers the stresses relative to the applied stresses and the distances relative to the size of the opening (e.g. diameter of the circle), the calculated stress profiles are dependent only on the shape and the ratio between the vertical and the horizontal stresses applied. These relative stresses and distances are independent of the size of the opening or of the absolute stress level.

On the other hand, the parameters that are important in the evaluation of the results are, for example, the type of solution algorithm applied (e.g. elastic theory, finite elements, finite differences, or boundary elements), the program used, and the mesh generated (e.g. type and size of elements). The importance of these parameters and of a proper model set-up is even greater for non-elastic calculations. With such calculations, the rock properties also play a role. In the future (as the authors already do), more and more non-elastic calculations will be conducted, but some first elastic calculations will remain advisable—not necessarily to derive practical conclusions, but mainly to verify the model set-up (e.g. size of elements, size of model, and boundary conditions) and detect basic errors in the model. With more complex constitutive laws, the effect of these errors will become less visible, but their effect on the results and on the practical conclusions can become much larger. With non-elastic calculations, it will become more difficult to forecast the results or to judge whether the calculated results are correct.

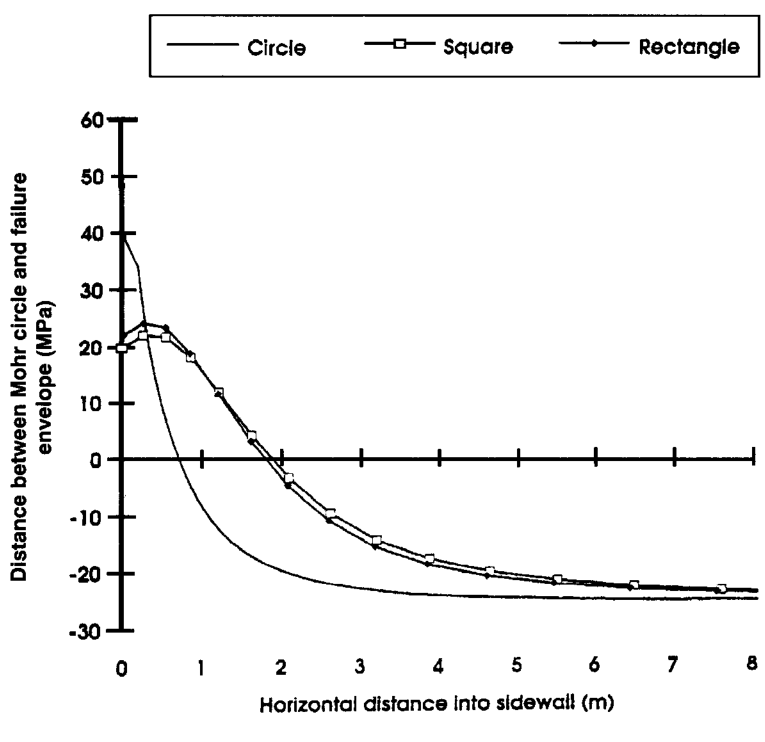

Figure 2—Minimum distance between the Mohr circle and failure envelope along the springline under elastic conditions

Interpretation of elastic results

By the application of a more complicated constitutive law, the calculated results should become more realistic—at least, if sufficient data are available for the input parameters of the constitutive law. But, even then, one has to interpret the results correctly. Even with the most complex constitutive law applied, the numerical model will NEVER entirely represent the reality. This is also not the aim of conducting simulations. Unfortunately, the interpretation of the results becomes more hazardous and difficult with a more complex constitutive law.

On the other hand, elastic calculations can, with a correct interpretation procedure, result in interesting and logical conclusions. The authors base their interpretation of the elastic results on only the vertical stress at one point (at the circumference in the middle of the sidewall). They also use an 'arbitrary(?)' chosen value of 125 MPa. This is a relatively limited criterion, resulting in an incorrect practical conclusion (that square and rectangular openings are the most stable). A good criterion should at least refer to the stress components in all directions (Mohr circle), and rather to an area than to a point. At the same time, one should compare the calculated stresses with the strength of the material, e.g. expressed by the linear Mohr–Coulomb failure criterion. In my Figure 2, the minimum distance (expressed in MPa) between the Mohr–Coulomb failure criterion and the Mohr circle of the calculated stress is presented for each point along the horizontal 'axis'. As parameters for the Mohr–Coulomb failure criterion, the cohesion and the friction angle were selected from Table A1 for a zero strain (cohesion = 17,5 MPa and friction angle = 27,5 degrees). A positive value on my Figure 2 means that a portion of the Mohr circle is above the failure criterion, and a negative value means that the Mohr circle does not intersect the failure criterion. Although the inequilibrium between the calculated stresses and the failure criterion is at the circumference itself larger for a circular opening than for a square or a rectangular opening, the 'unstable' area (positive values) is much larger for the square and the rectangle than for the circle. It is for this entire unstable area that the stresses have to be redistributed during failure of the rock; or, in other words, the elastic calculations show clearly that the square and rectangular opening are the most unstable.

Conclusions

I feel that the published paper offered me a good opportunity to make some critical points and general remarks regarding numerical simulations in the field of rock mechanics. These remarks must be seen independently of the value that the paper has and of the importance of the research that the authors are currently conducting. I certainly did not want to criticize or question the knowledge and experience that all three authors have in the field of rock mechanics and numerical simulation.

My main remarks can be summarized as follows:

- The solution algorithms used and the model set-up (e.g. the mesh generated and the boundary conditions) are the most important parameters in numerical simulations. In a comparative study, the same algorithm has to be used and the model parameters should be as similar as possible. The model set-up often requires a great deal of time.
- Even though a proper model set-up is very important, the interpretation of the final results is as important and as crucial. The interpretation becomes more important and more difficult with the complexity of the constitutive law applied. The time necessary to interpret the results should not be under-estimated. During the interpretation, one has often to refer to simplified models.
- Even by using a simple constitutive law, one can derive valuable results as long as a correct interpretation procedure is followed. Such a procedure should look at all the stress components, i.e. at an area rather than at a point, and at a strength criterion.
- Taking all these points into consideration, we should all aim to advance our knowledge in the area of numerical simulation. More complex constitutive laws should be part of our aim, but good and detailed underground measurements should also form a part. They are a necessary input for all numerical simulations.

Authors' reply

The contributions by Dr A. Vervoort and Mr W. Ortlepp are very welcome since both provide complementary information on the subject, the former on the theoretical background and numerical modelling, and the latter on valuable practical observations.

Vervoort's contribution

Vervoort queries whether the elastic stress results for the square and rectangular tunnels were obtained using BESOL or FLAC. They were from a FLAC simulation on a quarter-symmetry model using 2500 zones, with stresses applied to the outer boundaries and with a gravity gradient of zero. The formula quoted by him was derived by Kirsch[1]. It can be shown that the springline stress at the excavation boundary is $(3-k)p$, where k is the ratio of horizontal to vertical stresses and p is the vertical stress. Similarly, the crown stress is $(3k-1)p$. In the problem in the paper, this predicts a sidewall stress of 260 MPa and a crown stress of 52 MPa; hence, the failure in the sidewall but not in the hangingwall (since the UCS is 57,7 MPa). Speers has always advocated the use of closed-form solutions such as that of Kirsch for an elastic circle, and Ladanyi[2] for a plastic circle under a k ratio of 1 and, where the failed annulus has lost all cohesion and is purely frictional (i.e. maximum strain softening has occurred). This is because such solutions give a greater understanding of the problem and also allow the numerical modeller to check and compare his results.

The use of the finite-element code by Vervoort instead of the FLAC code (finite difference) is actually preferable in elastic simulations since the solution is faster and more accurate. The choice by the authors to use the criterion of distance into the sidewall to where the tangential stress drops to 125 MPa was arbitrary. As pointed out by Vervoort, a number of other criteria could have been used. What is interesting to note from Vervoort's Figure 1 (similar, but more accurate than Figure 3 in the paper) is that, if the UCS of the rockmass is above about 140 MPa, no sidewall overstressing is predicted at the excavation boundary of both the square and the rectangular tunnel, whereas overstressing will still occur on the circular boundary (since the stress induced is 260 MPa). In other words, the circular shapes (circle and ellipse) become more applicable when the stresses increase (with greater depth or overstoping, etc.), or if the rock strength decreases. This may explain that the square shapes have persisted so long because they are theoretically better at shallower depths or under lower stresses.

Elastic solutions, no matter what criterion is chosen for the rating of tunnel shapes, should always be a prerequisite to non-elastic modelling; however, not too much debate is necessary on the interpretation if it is shown by the results that the induced elastic stress exceeds the strength of the rockmass. We certainly encourage practical modellers to read the papers on elastic solutions (e.g. in the textbook by Brady and Brown: *Rock mechanics for underground mining*).

Ortlepp's contribution

The contribution made by Ortlepp provides useful input on the observational approach to the solution of rock-engineering problems in excavations. He should be encouraged to publish more on his experiences.

He misses a problem that seriously impedes high rates of advance in a high-stress regime, and that is the time taken to install support within a mining cycle. Overbreak can also largely be contained by the adoption of a more favourable tunnel shape, and this has already proven successful in numerous practical underground applications. The tunnel shape is therefore the first basic consideration and should take account of local geology (e.g. bedding planes), the stress state, and the rockmass properties.

The matter of shotcreting at the development face is a new approach in deep mines, and still requires a great deal of investigation into the mechanics of the problem. The de-rating of the modulus was not arbitrary and was based on calculations using the code guidelines that are available for concrete, taking into account the early loading. The comment about having a greater understanding is welcome, and this can come about by the application of all the available techniques (observational method, and *in situ* and laboratory tests, theoretical background, and numerical modelling). The comments raised on the apparent anomalies and omissions will be addressed in future investigations; the paper was not meant to be the final word on the subject. It is proposed that on-going work be carried out on the development of a definitive guide for support design.

In addition, if use is made of the very applicable and comprehensive shotcrete research and studies overseas (particularly in Europe), effects such as 'creep during curing' can be largely overcome without the need for a long-drawn-out local study.

It is generally agreed that models (especially computer models) should be handled and interpreted very carefully; however, they are very useful for qualitative analyses in general.

Part of the problem is that 'ancient geotechnologists' have not experimented adequately with available computer codes and hence tend to ignore the valuable contribution that they can make if correctly used. Total understanding of the physical laws is not always a prerequisite for use. An example of this is the design of modern aircraft based on laws (and computer models) that preclude the bumble bee from flying. No one, however, told the bumble bee this, and yet our aircraft designs are very successful and frequently used, even by 'ancient geotechnologists'.

The results in Table II were for simulation without support, and represent the response of the shapes to large strain softening. There could be anomalies in the detailed results because of this. In practice, support is installed at the face, which would inhibit such large strains. The significance of the results in Figures 5 to 7 is that a ring of shotcrete in a circular or elliptical excavation is relatively much stiffer than other support and therefore attracts load. The real question that should be asked is: Can the shotcrete take the load and how? As regards the comment about Y25 bars taking the forces in Table IV, it should be realized that these forces are the total required capacity per metre longitudinally, and not that required by individual elements. Also, the values quoted represent an upper bound since full strain softening was allowed in the simulations. Another paper by the first two authors is in draft form and will describe the modelling. Mr Ortlepp mentions that the actual mechanism was not modelled, but fails to actually say what, in his opinion, the mechanism is! As regards the comments on bolt spacing and length, the paper did not address these issues since the object was to compare the total support requirements for the sidewalls of the square and the ellipse. Once these have been determined, bolt spacing can be chosen depending on the capacity of individual elements, which should also have a yielding anchor so that it yields under seismic accelerations but still maintains its load-carrying capacity. It is recommended in the paper that optimization of bolt length, spacing, and orientation should be undertaken.

Conclusion

In conclusion, therefore, the authors reiterate the following points made in the paper.

- The conclusions need to be validated by underground trials and measurements (and this is to be carried out).
- Future research and investigations are being undertaken in line with the work described in the paper.

It must be remembered that all forms of models have limitations that must be considered by the modeller, whether they relate to computer modelling, scaled-down models, or models using blasting to simulate underground rockbursts. The key is whether the model helps to produce an understanding and/or a result that in hindsight is proved to be correct. ◆

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