FRACTURING AND DEFORMATION AT THE EDGES OF TABULAR GOLD
MINING EXCAVATIONS AND THE DEVELOPMENT OF A NUMERICAL MODEL
DESCRIBING SUCH PHENOMENA

RICHARD KENNETH BRUMMER

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Promotor: Professor C. G. Wilmot
Co-promotor: Professor N. C. Joughin

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ABSTRACT

This thesis describes an investigation into the nature of the fracture and deformation mechanisms which occur at the edges of tabular gold mining excavations. Published information on these phenomena is reviewed, and the necessary underground investigation required to consolidate the previous work is described. It is concluded that the rock near the reef plane at the edges of these mining excavations is subject to stresses sufficiently high to cause it to fracture through the formation of regular patterns of shear planes. These fractures can form in the solid rock some distance ahead of the mining excavation. Nearer the mining face, extension fractures form which result in slabbing or splitting of the exposed rock.

An idealization of the observed rock behaviour is proposed, which is then incorporated with conventional boundary element techniques into a numerical model (SEAMS) which is capable of analyzing two-dimensional tabular mining excavations where the rock near the reef plane at the edge of the mining excavation fractures, deforms and sheds load.

A sensitivity analysis of the numerical model is described which identifies those mining parameters capable of being used to advantage in controlling the size of the fracture zone. The model is then used to analyze two mining situations, namely a longwall face and a series of hypothetical pillars of various width to
height ratios. In both applications the model is shown to produce results consistent with behaviour observed in the field and laboratory.

It is concluded that the method of analysis described realistically and simply models the way a seam of rock fractures and deforms under high stress. This method can form the basis for the development of stress analysis packages which will permit rock mechanics engineers to design mine layouts taking into account the finite strength of most mined reefs. Although the numerical model described here is two-dimensional, the principle of the method can be extended to the analysis of three-dimensional tabular excavations.
1. INTRODUCTION

1.1 Rock Pressure in the South African Gold Mining Industry

For the past 100 years, almost since mining began on the Witwatersrand, rock pressure has posed hazards for the mining industry. The problem first manifested itself in the form of earth tremors, and it was concluded as early as 1908 that these were due to the "shattering of support pillars" (Ophirton Earth Tremors Committee, reported by Cook et al, 1966). At that stage, the mining activities on the central Witwatersrand were being carried out at depths shallower than 300 m.

The rock pressure problems which began at mining depths of 300 m, have become more severe as the depth of mining has increased; some mines are now operating at depths greater than 3600 m and even deeper mines are being planned. Experience since the early years of mining on the Witwatersrand has identified the rock pressure problem as the greatest hazard, threat and technological challenge to deep level mining. During 1985, rockbursts and rockfalls (hazards directly attributable to rock pressure) accounted for 59% of all fatal accidents and 26% of all injuries which occurred in South African gold mines (Legge, 1987). More significantly, rock pressure related fatal accidents have formed an increasing proportion of all South African gold mine fatalities since 1926 (Ibid.). Significant advances in the field of rock mechanics and its application in mines will have to be made before safe mining operations at depths greater than 4000 m can be planned.
with confidence.

It is assumed that the reader of this thesis is familiar with South African gold mining conditions, in particular the mining methods and geometries generally used. Terms such as stope, energy release rate (ERR), face etc. will not therefore be defined, as this would make the thesis laborious, and these have been defined elsewhere (see Budavari, 1983 for example).

1.2 The rock fracturing problem

The act of mining redistributes the virgin stress and induces increased stresses at the edges of tabular gold mining excavations. This results in extensive fracturing of the rock near the reef plane. The stope faces have to be advanced through this fractured region, and this causes several severe problems for the industry, as outlined below:

1. Support problems - The loose blocks of rock bounded by mining induced fractures and geological discontinuities need adequate support to prevent them falling in on mine workers.

2. Control of stoping width - This problem is closely related to the support problem, and occurs where extensive falls of ground result in the unavoidable removal of large quantities of barren rock along with the reef. This reduces the grade of the reef which has to be treated and can make the mining operations uneconomic.
3. Drilling difficulties - Conventional mining is carried out by drill-and-blast methods. Experience has shown that it is more difficult to drill into heavily fractured rock than into intact rock.

4. Problems regarding mechanization - Research is currently being carried out into the feasibility of large scale mechanization of gold mining. One of the techniques being examined is non-explosive mining of the reef by means of various mechanical rock breaking systems. It has been observed in an experimental mechanically mined stope that occasionally the regular pattern of fractures on the face does not form, and this makes it very difficult to mine mechanically. Current mechanical engineering technology is such that an unfractured face cannot be broken economically without explosives.

5. Transport of the broken rock - Rock handling difficulties are caused by the occasional production of large blocks of unfractured rock, even by conventional blasting. The rough footwall produced by the heavily fractured rock also makes it difficult to move the broken rock out of the stope.

1.3 Energy considerations

In South Africa two major advances in the field of rock mechanics have occurred in the last two decades. The first is the
recognition that outside the fracture zone surrounding mine excavations, the rockmass behaves elastically. The second is the development of the Energy Release Rate (ERR) concept (Cook, 1967 or summarised in detail by Salamon, 1984) and its use to determine how seismically hazardous a mine layout is likely to be (Hodgson and Joughin, 1967). These two advances have provided rock mechanics engineers with a rational method for designing the layout of a deep mine.

If the rockmass behaved entirely elastically, the ERR would correspond to the amount of strain energy actually stored in the rock removed from the face during mining (Salamon, 1984). In practice, the removed rock does not "contain" this amount of strain energy; the energy is released or dissipated in fracturing and deforming the rock near the stope face. Since stored strain energy can potentially be released catastrophically in the form of seismic events, a better understanding of the energy changes which occur in the fractured rock could thus hold the key to designing safer mining layouts.

1.4 Scope of work

The research described in this thesis was undertaken in order to improve the understanding of the behaviour of the fracturing rock at the edges of tabular gold mining excavations, and to develop a numerical model capable of describing the essential aspects of the observed behaviour. In order to achieve these objectives, the following investigations were necessary:
1. Firstly it was necessary to determine the nature and pattern of the fractures which form ahead of stope faces and at the edges of tabular mining excavations. This was done by reviewing the extensive published literature on the subject, and by carrying out underground fracture mapping in greater detail and over larger areas than had been done before. In this way, it was possible to combine the previously published work with the detailed fracture mapping and thereby to generate a representative fracture pattern.

2. In parallel with the investigation of the fracture pattern, it was necessary to determine the deformation mechanisms which occur in association with the formation of fractures. This was done by reviewing the published literature on the subject, and by making additional supplementary measurements and observations of the nature and amount of deformations which occur in the fractured rock near the reef plane.

3. After the fracture pattern and mode of failure of the reef rock had been identified, an idealized mechanistic model for the behaviour of the fracturing reef rock was proposed, and the behaviour of this failed rock model was fully investigated. It was demonstrated that the model can be viewed as defining the constitutive law for the fracturing rock and that this constitutive law is qualitatively similar to the behaviour of triaxial rock specimens tested in the
laboratory. With appropriate model parameters, the model also produces quantitatively realistic behaviour. This model for the falling rock was then incorporated into a boundary element scheme which enables one to model a two dimensional stope, and allow the rock near the reef plane to deform inelastically, while the rock remote from the mining excavation behaves according to the laws of elasticity.

4. Parameter studies were then performed, where the sensitivity of the model to the various model parameters was investigated. This was done partly to verify that the model was capable of modelling a relatively wide range of situations, and partly to investigate whether any of the parameters could be used in a real mining situation to control the behaviour of the fracture zone.

5. The results produced by the model were then shown to be consistent with both the observed depth of fracturing and amount of dilation measured in a well documented investigation. A further study was then carried out to demonstrate that the behaviour of brittle rock pillars can be realistically modelled by means of this technique.

The numerical model developed and described here requires several input parameters, some of which are very difficult or impossible to measure in the field. However, it is usually possible to back-calculate the required parameters from known situations
underground, or to make reasonable assumptions. In this way, the numerical model can be tuned to match a particular set of circumstances, and then used to extrapolate beyond known conditions. Aspects of the model which are not well established, or where information is unavailable due to a lack of experimental data (stresses, for example are notoriously difficult to measure in highly stressed fractured rock) are critically reviewed near the end of the thesis.

1.5 Structure of thesis

This thesis is arranged broadly around the investigations described above, and consists of the following sections:

Section 2 contains a review of the relevant literature on mining-induced fracturing, deformation and numerical modelling of mining excavations.

Section 3 describes work done to investigate and explain the pattern of fracturing commonly observed to exist at the edges of tabular mining excavations at depth and concludes with a proposed typical fracture pattern.

Section 4 identifies and describes deformation mechanisms which are observed to occur and shows how these mechanisms operate together to permit the large scale overall deformations of the rock mass. A mechanism for the gross inelastic deformations observed is proposed.
In Section 5 the details of a numerical model (called SEAMS) based on the proposed mechanism and standard boundary element theory are then developed. The section concludes with various examples demonstrating the results produced by the numerical model and comparing these results with analytical solutions and other published data.

Section 6 contains a brief discussion of the parameters required by the model, suggesting and justifying suitable values for these. A sensitivity analysis is also presented which investigates the effect which each parameter has on the results produced for a typical case, i.e. the fracturing at a stope face.

Section 7 comprises a validation exercise, and demonstrates applications of the model for the analysis of the fracture zone which forms at the stope face and the behaviour of pillars of various width to height ratios.

Section 8 contains a critical appraisal of the method developed and discusses the major uncertainties and assumptions which have had to be made in developing the model. These uncertainties indicate areas where further research is necessary. The conclusions of the thesis are presented in Section 9.

A listing of the computer code developed in the course of the research is contained in the Appendix.
rock are shear fractures; the extension fractures are only observed within a few metres of the 'stope face'. This is very strong evidence which shows that the shear fractures form first, i.e. furthest ahead of the face, and the extension fractures form at a later stage, due to the proximity of the actual stope face (See also Cook, 1984 and Section 7.2). The orientation of the shear fractures is such that they form conjugate pairs while the orientations of the extension fractures are such as to coincide with the expected direction of maximum principal stress associated with the stope, as indicated in Figure 3.6.

3.4 Summary of fracture characteristics

The fracture mapping carried out revealed that although there were variations in the observed fracture patterns between sites, certain common features existed. These characteristics of the fracture patterns are summarised below:

1. Shear fractures appear to form first, i.e. furthest ahead of an advancing face.

2. The shear fractures were observed to be spaced at fairly regular intervals. They were spaced at intervals of 1,0 m to 1,5 m at most of the sites mapped, which had ERR values of between 10 MJ/m$^2$ and 40 MJ/m$^2$. No consistent difference in spacing due to ERR was found.

3. In the footwall, a set of shear fractures forms which dips away from the mined area. The fractures curve slightly, such that they become vertical about five metres below the
reef plane, as can best be seen in Figure 3.5.

4. For reasons of symmetry, it is speculated that a similar but opposite pattern of shear fractures must exist in the hangingwall. For safety reasons it was not possible to investigate fracturing further than 2 m into the hangingwall, and this was only done at one site.

5. Near the reef plane, the shear fractures were observed to form conjugate pairs and it is suggested that this is due to an 'overlap' of the hangingwall and footwall fracture patterns.

6. Because the extension fractures were found to closely follow the shape of the stope face both on plan and in section, and because they were found to truncate on the shear fractures, it must be inferred that they form nearer to the advancing stope face, and after the formation of the shear fractures. This is clearly shown in Figure 3.6. It is thus probable that they form due to a lowering of the minor principal stress to the extent that the rock fractures in an almost uniaxial (plane strain) stress field, and are therefore phenomenologically identical to the 'slabbing' observed on tunnel sidewalls and the vertical splitting observed during uniaxial testing of rock samples.

3.5 Typical pattern of fracturing

Since the object of mapping the mining-induced fractures was to explain and understand the failure and deformation mechanisms producing these fractures, the common characteristics of the
fracture patterns observed have been combined to produce a representative picture of the fracture pattern which forms ahead of a typical stope face in a deep gold mine under similar geological and stress conditions to those mapped during this study. This pattern is shown in idealized form in Figure 3.7. A number of zones are indicated in the figure, each of which will be described in turn.

Zone 1
This represents the boundary between fractured rock and intact rock and is thus the position where the shear fractures first form. These fractures appear to separate the rock into distinct blocks of relatively intact material since the microfractures localize in a narrow plane of weakness, leaving the rest of the rock unfractured, as has been described in laboratory specimens by Hallbauer et al, 1973. From arguments presented earlier, it appears that in a borehole, the initial formation of these shear fractures appears as a zone of intense vertical fracturing. (See Adams, et al, 1981).

These shear fractures are parallel to the overall longwall direction and are spaced fairly regularly (see Figures 3.3 to 3.6). They curve slightly as can best be seen in Figure 3.5.

The formation of these shear fractures results in an initial dilation of the rock toward the stope, as was measured by Legge (1984).
Zone 2

The stresses in this zone can be described by points on a residual Mohr envelope, that is for rock in a state of limiting frictional equilibrium. Since the shear fractures have formed, the rock will have negligible cohesion along any fracture.

Two processes appear to occur in Zone 2:

1. On a macroscopic scale, movement is observed on the shear fractures formed in Zone 1. (This movement will be described in Section 4.)

2. On an intrablock scale, local high stresses at asperities and point contacts (Chappell, 1975) can cause individual blocks to split in a direction more or less parallel to the maximum principal stress direction, thus generating a number of extension or tensile fractures, with little or no relative displacement. (Where two shear fractures intersect, the "wedge tips" thus formed result in such a local high stress area, where severe crushing is commonly observed.)

Zone 3

This is a zone of local longitudinal splitting or extension fracturing right on the face, parallel to the direction of maximum principal stress, and either brought about or promoted by the presence of the free surface of the face. This process produces the slabbing on the immediate face which is usually observed in
stopes, and on the sidewalls of most tunnels at depth.

It is probable that in shallow or small-span stopes (having ERR values less than about 2 MJ/m²), this type of fracturing will be the only type of fracturing observed, as the stresses ahead of these faces will not be high enough to cause shear failure of the rock, as described in Zone 1. Hence the depth of fracturing ahead of these stopes will be only 0,5 m to 1,0 m of vertically split rock on the face. The sides of tunnels are probably the most common manifestation of this phenomenon.

Zone 4

This zone consists of the distressed strata in the hangingwall of the stope, where stresses are too low to result in the formation of new fractures.

Zone 5

A similar layer of fractured rock must exist in the footwall of the stope. This layer is often relieved of horizontal stress because of the development of dip gullies as shown in Figure 3.7.

This fracture pattern forms the basis for the numerical model developed in Section 5.
closure, and therefore possibly also on the stope conditions, and the fracturing and deformation which occurs ahead of the stope face.

4.5 Deformation mechanism for gross inelastic movements

Based on the fracture patterns as described in Section 3 and the deformations described here, the following mechanism for the gross inelastic deformation is proposed:

High vertical stresses, induced in the solid rock ahead of the face by the mining operations, cause the rockmass to fall in shear, forming macroscopic inclined shear fractures, on which normal fault type displacements occur. This permits the rock to yield vertically, and shed vertical load, but results in a horizontal dilation, accompanied by an overall volumetric increase. The horizontal dilation is facilitated by the presence of the parting planes which exist in the host rock, since slip movements occur on these planes. These deformation processes occur over a height greater than the actual mined stope width, a height referred to here as the "effective stope width". The dilation of the fracturing rock ahead of the face appears to force the layers of rock in the hangingwall and footwall horizontally into the mined area. If insufficient support is installed in the stope, these layers can buckle into the stope. The layers of rock in the hangingwall and footwall must therefore provide confining stresses (the magnitudes of
which are not known at this stage) to the deforming rock ahead of the stope.

This proposed mechanism, illustrated in Figure 4.18, forms the basis for the numerical model developed in Section 5.
Figure 4.18  Mechanism for inelastic deformation near typical stope face

(Extension fractures omitted for clarity)
5. DEVELOPMENT OF THE FRACTURED ROCK MODEL

The investigation of the fracture and deformation processes resulted in the identification of the deformation mechanism described at the end of Section 4. This mechanism is qualitative, since it was idealized from many underground observations. In order for such a conceptual model to be useful, it is necessary that the model be shown to be quantitatively realistic. It was thus necessary to develop and expand the conceptual model within a numerical framework.

Since the primary objective of this research was to improve the understanding of the deformation phenomena, it was not necessary to develop a numerical model capable of being used as a mine design tool. Rather it was desired to have a model which could firstly be demonstrated to be a realistic model for the true processes which occur underground, and secondly which could be used to explore the sensitivity of the deformation mechanisms to various parameters which can possibly be used to influence the fracture zone.

5.1 Criteria for a suitable model

From the arguments presented above, it was possible to decide on the characteristics required of the proposed model. These requirements are outlined below:

1. The model should capture the essential elements of observed
rock behaviour at the edges of tabular mining excavations.

2. The model should be able to predict the horizontal extent (ahead of the face) of the fracture zone.

3. The model should be able to predict the amount by which the rock ahead of the face is crushed (vertically).

4. The model should be able to predict the amount by which the rock ahead of the face dilates.

5. The model should be able to account for the energy changes which occur in the fractured rock.

6. The model should explain or predict the stress distribution within the fracture zone ahead of the face.

In order to meet these criteria, the observed fracture pattern (Figure 5.1(a)) and inelastic deformation mechanism (Figure 5.1(b)) were idealized for the purpose of numerical modelling as shown in Figure 5.2.

The illustrated model embodies several simplifications to the observed rock behaviour. The most important of these is that the elastic rockmass is separated from the fracturing rock by distinct parting planes in the hangingwall and footwall. In practice, horizontal slip is usually observed on several parting planes both above and below the stope. However, as a first approximation, the movement is regarded as occurring on two planes only, in the interests of simplicity. This is felt to be reasonable, and refinements to this approximation can be implemented later should they be found to be necessary. In addition, since the model is
Figure 5.1  Fracture pattern (a) and inelastic deformations (b) near typical stope face
Figure 5.2  Idealization of fractured rock near stope face
elements are in equilibrium with the surrounding rockmass and satisfy their constitutive laws. Figures 5.14 to 5.16 illustrate the operation of the mining steps. Figure 5.14 represents a 240 m span stope, the "face" of which was mined in two steps, to produce Figures 5.15 and 5.16, which are essentially identical to the initial state, with stress profiles advanced by the size of the mining step.

5.12 Work done on the fracture zone
When the mining face is advanced, or an element is "mined", the equilibrium between the remaining seam elements and the surrounding elastic rockmass is disturbed. In order for equilibrium to be re-established, the remaining seam elements deform non-linearly and the fracture zone advances ahead of the face by approximately the same distance that the face was advanced.

If it is assumed that the readjustment process occurs quasistatically, then it is possible to calculate the amount of work which the elastic rockmass does on the seam elements. Previous workers have merely stated that part of the "released" energy is expended in doing work on the fracture zone. The numerical technique described in this thesis makes it possible to quantify the amount of energy expended in this way.

Since the elastic boundary (neglecting second order effects) only crushes the rock in the vertical direction, with no net ride
Figure 6.7 Results of base run of sensitivity analysis
numerical model developed here does not account for this mode of fracturing, being based entirely on shear deformation mechanisms. Within the limitations imposed by the assumptions made in developing the model, agreement between observed and predicted dilation profiles is thus good.

7.3 Energy consumption in the fracture zone

As outlined in Section 5, SEAMS may be used to determine the energy absorbed in the fracture zone. The mining step analyses of Figures 5.14 to 5.16 were therefore used to calculate the work done by the elastic rock on the seam for typical conditions at the Doornfontein site. This calculation is shown in Table 7.1, and the results are presented graphically in Figure 7.3, which shows that the work done on the rock is a maximum some seven metres ahead of the face, i.e. at the boundary between the fractured and the unfractured rock. The total work done by the elastic rock on the seam in this example may be found by summing the work done on each element. In this case a total of 13,0 MJ of work (due to a face advance of 0,5 m) was done on the seam, part of which was stored elastically in the seam elements, but most of which was expended in doing work on the first 7 m of rock ahead of the face which is deforming inelastically. In this particular example, only a very small amount of stored strain energy is removed in the element which is mined from the face. The nominal Energy Release Rate for a slit of comparable span to the example presented is 25,7 MJ/m. For an advance of 0,5 m this corresponds to 12,85 MJ,
### Table 7.1: Work Done on Seam by Elastic Rockmass

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</tbody>
</table>

Total work done on seam during 0.5m mining step 13.006 MJ

This corresponds to 26.0 MJ/m of face advance.
Figure 7.3  Work done on seam ahead of the face by the elastic rockmass

Area = 13 MJ
in close agreement with the value calculated from the model. Thus ERR in a realistic scenario of rock fracture is seen to be accounted for in terms of energy absorption taking place stably in the fracture zone.

Patrick (1983) showed that microseismic events were concentrated ahead of the face as shown in Figure 7.4. The location accuracy of the events was between 2 m and 5 m but a clear trend is evident, with most events being concentrated up to 7 m ahead of the face, and within the vertical extent of the Effective Stope Width (7 m in this case). Assuming that these microseismic events are caused by slip movements on the many slip planes which make up the fracture zone, it is evident that there is good agreement between the locations of these energy-consuming processes in the model and ahead of a real mining face.

7.4 Load-deformation behaviour of model pillars

The load-deformation behaviour of rock pillars is not well understood, mainly because of the extreme difficulty involved in measuring both the load carried by a real pillar underground and the actual deformation of such pillars. Much of what is currently known about the strength and deformation of pillars has therefore been inferred from back analyses of well documented case histories and laboratory tests of model pillars, a notable exception being the insitu coal pillar tests described by Wagner (1974). If, by fine-tuning of the parameters used in SEAMS, the results can be made to compare closely to the results of a series of laboratory
Figure 7.4 Microseismic event distributions near the stope face at the Doornfontein Experimental Site (after Pat trick 1983)
tests, then it should be possible to carry out realistic analyses of real pillars underground, by making appropriate adjustments to the parameters used.

Tests on model pillars are often criticized because the dissimilarity between the elastic properties of the pillar material and the steel of the loading platens can result in unrealistic stress distributions within the pillar. (A possibly more serious shortcoming of such tests is that foundation failure of the rock below and above a pillar is not possible in a model pillar test where the rock is confined between steel platens.) It is possible, using SEAMS, to evaluate the significance of differences in the elastic properties between the "pillar" (or model) and the "host rock" (or loading platens), since these properties are specified independently.

It is known from underground observations and laboratory studies that wide pillars do not shed load after failure, but that narrow pillars can shed load and even become unstable when the post peak negative slope of the pillars' load/deformation curve exceeds the "system stiffness" of the surrounding rock.

In order to investigate these aspects of pillar behaviour, a series SEAMS analyses is compared here in a qualitative way to a series of laboratory tests on model pillars carried out and described by Wagner and Madden (1984). This comparison is not ideal, since SEAMS considers only two dimensional plain strain
situations, and the model pillar tests were axisymmetric. However, no comparable truly plain strain results were available at the time of writing.

The numerical results were produced by analysing two metre high pillars (ESW = 2.0 m) with widths of 2, 4, 8 and 12 m which were "loaded" in a mining situation with 20 m mined "bords" on either side.

The geometry chosen was as shown in Figure 7.5, and the pillars were loaded by increasing the virgin (or field) stress in the host rock.

The average stress carried by each pillar was plotted against the average pillar deformation at each stage of loading. In this way, an averaged stress/deformation curve was obtained for each pillar. The results for each pillar are discussed below:

2 m x 2 m Pillar - Figure 7.6
Because of its relatively narrow width, this pillar is roughly equivalent to a uniaxial test specimen, and one would thus expect its average stress/deformation behaviour to be similar to that of such a lab test specimen.

In the analyses, it was found that on increasing the virgin stress from 40 MPa to 41 MPa (corresponding to a pillar stress equivalent to the uniaxial strength), the pillar dramatically failed,
Figure 7.5  Geometry of test pillars

Pillar Width

20 m "bord"

20 m "bord"

2 m
Figure 7.6  Average stress/strain behaviour for 2m x 2m pillar
shedding virtually all load, as would a real pillar in a similar situation (e.g. Salamon, 1983, Brady, 1979). The reason for this behaviour is that the post peak stiffness of the pillar was less than that of the "system" of surrounding rock. The numerical model behaved entirely stably in this case - the Instability apparent in the stress drop from A to B is a real, physical one, the start and end points of which the model is perfectly capable of following, as demonstrated by Figure 7.6.

4 m x 2 m Pillar - Figure 7.7
The behaviour of the 4 m x 2 m pillar was qualitatively very similar to that of the 2 m x 2 m pillar.

Note that the slope of the line from C to D which represents the "local mine stiffness" (See Hoek & Brown, 1981) appears to have been reduced, but this is as a result of the fact that average stress and not load has been plotted. The local mine stiffness in terms of load is relatively constant throughout the analyses presented in this section.

8 m x 2 m Pillar - Figure 7.8
The 8 m x 2 m pillar was sufficiently wide that significant confinement was generated by the fractured edges. This resulted in the pillar being capable of supporting a substantial load after the peak strength had been reached. In order to generate this confinement however, significant crushing of the pillar was necessary, as can be seen in Figure 7.8.
Figure 7.7 Average stress/strain behaviour for 4m x 2m pillar
Figure 7.8  Average stress/strain behaviour for 8m x 2m pillar
The behaviour of this pillar was significantly different from the two shown in Figures 7.6 and 7.7. The stress (load) drop after peak strength was far less severe. In practical terms, this suggests that the 1:1 and 2:1 pillars failed violently, but the 4:1 pillar yielded in a more controlled and stable manner.

12 m x 2 m Pillar - Figure 7.9
This pillar, with a 6:1 width to height ratio, did not fail violently. In the analysis using 250 MPa virgin stress the central core of the pillar was still elastic. At higher loads, the pillar can be expected eventually to yield throughout, but it is so wide that large confinement stresses are generated. This means that the "residual strength" of the pillar is large compared to the uniaxial strength of the rock and although the material itself is strongly strain softening, the overall behaviour of the pillar is stable because of the loading geometry and confinement forces generated.

Discussion
The pillar analyses are in qualitative agreement with observed pillar behaviour in the laboratory and insitu. A point which arose from the analyses carried out and which is worthy of discussion is the fact that as the pillars were made wider, the stress at which yield first occurred apparently was lower. At first glance this behaviour appears to be contrary to expectation.
Figure 7.9  Average stress/strain behaviour for 12m x 2m pillar
The explanation for this phenomenon is that the stress distribution within the pillars depends on the interaction between the elastic pillar and the surrounding elastic rockmass which has elastic modulus similar to the pillar. This interaction results in a highly non-uniform stress distribution within the pillar, with the edges carrying substantially more stress than the core. This effect worsens with increase in width to height ratio. The hangingwall rock tends to "sag" over the pillar, which promotes early failure of the pillar edges.

This situation is very different to the laboratory test, where the steel of the loading platen is usually much more rigid (higher elastic modulus) than the rock specimen, and the stress distribution is therefore much more uniform.

In order to confirm this explanation, the series of model pillar simulations was redone using a host rock modulus considerably higher than that of the pillar. The analyses are thus similar to a laboratory pillar model with the "stiff" country rock representing the "stiff" steel loading platen.

These simulations revealed that the pillar strength did indeed increase with increasing width to height ratio. The simulated laboratory stress - deformation curves for the pillars are shown in Figure 7.10. As can be seen, the peak strength of the pillars increased with width to height ratio, as is commonly reported from laboratory tests e.g. Figure 7.11 (Wagner & Madden, 1984, Hudson,
Figure 7.10 Simulated laboratory stress/deformation curves for pillars
Figure 7.11 Stress/strain curves for pillars (after Wagner & Madden, 1984)
Brown & Fairhurst 1971). The simulated stress-deformation curves also bear a close resemblance to those derived from laboratory scale tests. The most notable difference between the curves shown in Figures 7.10 and 7.11 is that in the simulated curves, the shape of the curve is more sensitive to width to height ratio. The reason for this is that the numerical curves are derived for a two-dimensional plane strain situation, whereas the lab test results shown in Figure 7.11 are derived from axisymmetric pillars which are inherently less confined. The numerical model thus gives good qualitative agreement with laboratory data.

The most significant conclusion arising from this series of simulations is that the "draping" effect which occurs in the case of pillars which have elastic moduli the same as the host rock (as in a gold mine for example) results in a lower peak yield stress for increasing width to height ratios. This suggests that it is possibly dangerous to extrapolate results derived from laboratory tests conducted using relatively stiff loading platens to a real underground mining situation in cases where the pillars and host rock consist of similar material.
9. CONCLUSIONS

Investigation of the fracture zone by means of underground mapping revealed that on a macroscopic scale, the processes which occur there are relatively simple to understand. The high edge stresses induced by the mining cause the rockmass to fracture in such a way that the high vertical stresses are relieved. The most natural way for this to occur is for inclined shear fractures to form, which permit the rock to dilate horizontally into the mined area. This horizontal dilation is facilitated by the horizontal bedding or parting planes which occur at 0.3 – 1.0 m spacing. Near the mining face, low confinement or extension fractures form, similar to those observed in a uniaxial compression test.

The relatively regular spacing of the shear fractures results in an apparent "banding" of fractures ahead of the mining face, as has been reported by previous researchers.

In the fracture zone, the reef rock shortens vertically but dilates horizontally in such a way that the rock volume is almost constant (since the strains are large compared to those easily simulated in the laboratory). Based on laboratory data it must be inferred that some volumetric increase occurs as the fractures first form. The deformations which occur are substantially larger than can be easily obtained during laboratory testing of triaxial, plane strain or extension test specimens, and it was thus necessary to develop a constitutive law based on certain
assumptions about the post-failure behaviour of rock. The constitutive law developed in this thesis is in agreement with observed rock behaviour in the laboratory, but extends the range of available data far into the nonlinear zone.

The overall deformations of the fractured rock as described above manifest themselves on a small scale as discrete shear displacements on shear fractures and as horizontal slip on bedding planes. Since these processes were observed to occur in a region of rock whose vertical extent is usually greater than the actual stoping width, the existence of an "Effective Stope Width" was postulated. This Effective Stope Width can be identified in suitable boreholes as the vertical height between the highest and lowest parting planes on which substantial horizontal slip occurs.

The dilation of the fractured zone must influence the stope hangingwall and footwall, and the closure of footwall dip gullies and occasional upward buckling of footwall strata are proposed as consequences of the Effective Stope Width concept.

The most important feature of the geology of the near reef rock is its bedded nature. The parting planes, from underground observations and from numerical model studies, appear to exert a strong controlling influence on the deformation processes which occur near the edges of the tabular excavations.

A relatively simple numerical model which is capable of capturing
the essential features of the observed phenomena has been developed. This model realistically describes the elastic behaviour of the host rock by means of boundary element (Displacement Discontinuity) techniques. The reef horizon is regarded as a yielding seam. The vertical stress which each seam element can support is related to the minimum horizontal confinement provided by adjacent elements. In this way, relatively large deformations are permitted to occur when the seam elements fracture and deform. Horizontal dilations predicted by the model are in good quantitative agreement with observed dilations. The model also yields fracture zone depths and vertical stress profiles which agree with available underground observations.

The numerical model was applied to the analysis of the load-deformation behaviour of pillars of various width to height ratios. A potentially very significant result was obtained from this series of simulations. It was found that the "draping" or "sag" effect which occurs when a pillar is stressed by country rock of comparable modulus to that of the pillar, significantly reduces the peak strength of the pillar. This implies that it may be dangerous to extrapolate pillar strengths derived from model tests carried out in the laboratory to the underground mining situation. This effect is probably not significant in soft seam or coal mining geologies, but can be extremely significant where hard-rock pillars are left.
The algorithm describing yield of the fractured rock is potentially sufficiently simple and general enough for it to be incorporated into a three-dimensional tabular mining stress analyzer. This will enable rock mechanics engineers to design three-dimensional mine layouts for tabular excavations using analytical tools which simply but realistically account for the way in which the rock near the edges of their excavations fractures and deforms.

The numerical model provides a satisfactory explanation for the energy "released" by mining. An element on the face, if mined or removed, contains relatively little strain energy. Its removal however, disturbs the equilibrium established between it, the elements ahead of it, and the elastic rockmass. In re-establishing the equilibrium condition, the elastic rockmass deforms the seam elements and does work on them. This work is equal to the Energy Release Rate, less the energy removed in the mined element. In most cases, virtually all of the "released" energy will be expended in crushing the rock ahead of the mining face, and virtually none of this energy will be actually "removed" in the mined rock, nor will a balance of energy necessarily be available for release in the form of crippling seismic events or rockbursts.

Certain mining parameters are controllable. It was established by means of the numerical model that it should be possible to influence the size of the fracture zone by reducing the friction
coefficients on the parting planes or shear fractures (for example by means of water infusion). Altering the Effective Stope Width should also produce a change in size of the fracture zone. This is not necessarily true if one attempts to alter the actual stope width. The practice of cutting dip slots in the hanging and footwall as is routinely done on some mines, will almost certainly produce a change in size of fracture zone. This practice (i.e. initiating "caving") should tend to move the stress peak further ahead of the face, as has been proposed by various researchers.