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Dynamic Rock Failures Due to "High" Stress at Shallow Depth

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Abstract

High stresses are expected in deep mines, and it is perhaps unusual for rock failure due to high stresses to be observed in shallow mining. In this paper, stress-induced failures of competent rock are described in three shallow mining situations. Two of the rock failures occurred with considerable violence, both involving significant strain bursting seismicity. Rock failures in the third case often occurred with some violence and were preceded by audible noises. The first situation is failure of very competent granite in a dimension stone quarry, in which the depth of the "vertical cut slope" was about 3 m. The second situation is failure of the roof in a coal mine at a depth of about 50 m below surface. The third situation is also failure of the roof in a coal mine, but at a depth of about 25 m below surface. None of these failures can be explained using commonly-used rock strength criteria.

1 Introduction

Violent stress-induced failures of rock would not commonly be expected at shallow depth, in particular in "unjointed" rocks at very shallow depth. Engineering reasoning would simply compare significant rock strength magnitudes with low stress magnitudes, and the immediate conclusion from judgement, even without any calculations, would be that the factor of safety would be high and that the possibility of failure would be remote. In this paper, three mining cases are described in which such "unpredicted" failure occurred violently. The first situation is failure of very competent granite in a dimension stone quarry, in which the depth of the "vertical cut slope" was about 3 m. The second situation is failure of the roof in a coal mine at a depth of about 50 m below surface. The third situation is also in a coal mine, describing very violent failure of the roof at a depth of only about 25 m below surface.

2 Rock failure in a dimension stone quarry

The environment of this case is shown in Figure 1, illustrating a small dimension stone quarry in high quality granite. At the stage of mining, only very limited extraction of rock had taken place, with quarry depths being only a few metres, as shown in Figure 2. Unfortunately, no detailed mapping, no laboratory testing and no in situ stress measurements were carried out. Therefore, this case is purely observational.

The granite is very high quality rock whose strength is unlikely to be less than 200 MPa. The stresses in the rock are unknown, but, taking into account the near surface location and the shallowness of the quarry, are unlikely to exceed say 10 to 20 MPa. Even if higher than these magnitudes, the stresses would be unlikely to be anywhere close to the strength of the rock. After excavation, fractures developed, often with considerable violence, sub-parallel to the surface, as shown in Figure 3. It was reported that the machine used to drill the closely-spaced vertical holes was "thrown" upwards on one occasion. Fracture development was heard as "rifle shots" in the evenings. Sub-vertical fractures developed subsequently, parallel to the cut face of the quarry, as shown in the right hand photograph in Figure 3, and it can be seen that they have extended beyond the boundary of the excavated quarry face. Minor offsets of the vertical drill holes can be seen in Figure 3, and this behaviour was more pronounced at other locations.

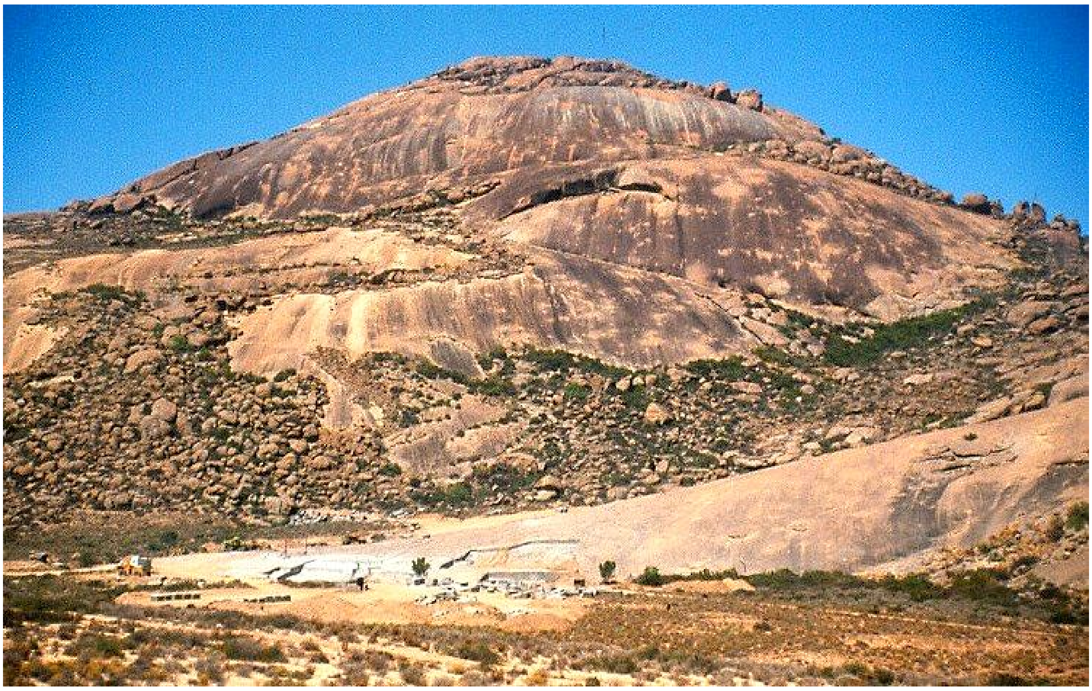

Figure 1 Setting of dimension stone granite quarry

Figure 2 Extent of the quarry

The occurrence of the fracturing is very detrimental to the mining operation, whose aim is to produce large unfractured blocks of granite. The very nature of dimension stone is that, in spite of the fact that it is produced in large blocks, it is essentially rock material rather than rock mass. Realistic values of Hoek-Brown and Mohr-Coulomb strengths for the rock far exceed the likely stresses acting in the rock. Therefore, the conventional methods of prediction of rock failure do not appear to be appropriate in this case.

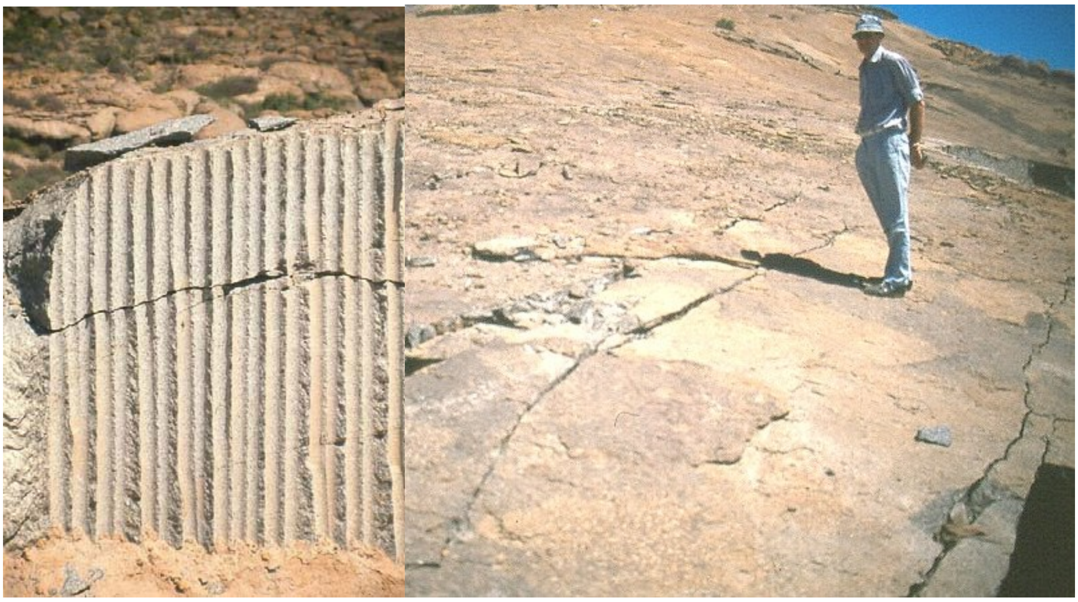

Figure 3 Stress-induced fractures sub-parallel to the surface and to the quarry face

A compressive stress in the near surface rock will induce extension strains in the rock, normal to the minimum principal stress. This is approximately sub-vertical prior to excavation, and then also sub-horizontal, normal to the quarry face, after excavation. With compressive stress magnitudes in the range of 10 to 20 MPa, the resulting extension strain magnitudes can be expected to be in the range of 100 to 200 microstrains. Stacey (1981) suggested an extension strain criterion to predict the initiation of rock fracture, and critical strain magnitudes identified for brittle rock were of this order. The fractures formed in the quarry appeared to be extension in nature and therefore this extension strain criterion satisfactorily predicts the initiation of such fractures. From reports of the time of occurrence of fracturing, temperature effects, due to heating during the day and cooling at night, also appear to have had an influence. The measurements of strains that develop in exposed rock surfaces due to temperature variations, carried out by Dunner et al. (2007), indicate that strain magnitudes can be significant. They measured compressive strains of about 200 microstrains sub-parallel to the surface and extension strains of about 100 microstrains perpendicular to the surface. Such temperature-induced strains would add to the strains resulting from the in situ stresses acting in the rock, and would clearly contribute to the development of extension fracturing.

The development of extension fracturing in the quarry is likely to have led to the formation of incipient slabs, and a buckling mechanism was probably involved in the ultimate violent failure observed, i.e. the mechanism violent failure of the roof at a depth of only about 25 m below surface.

3 Rock failure in the roof of a shallow coal mine

Coal production in South Africa is mainly from bord and pillar mining, which accounts for about half of the total production and 90 per cent of the total underground production. A problem associated with board and pillar mining of coal is roof instability. One such cause of roof instability is roof guttering, in which the roof rock fractures, falls, and leaves a “groove” in the roof along a roadway.

The occurrence of guttering results directly in roof instability, but in addition, may interact with natural weakness planes in the roof rock, resulting in more extensive instability. Roof guttering has proved to be a safety and production constraint. It exposes miners to the hazard of rock falls, and these falls may cause damage to machinery and mine workings. In some instances endangered workings are abandoned, leading to loss of valuable mineral reserves. Rock falls are the single largest contributor to safety related fatalities in the South African mining industry (MHSC 2004/2005 Annual Report).

Frith et al. (2002) noted that in most situations guttering is a result of high horizontal stresses. Van der Merwe and Madden (2002) acknowledged that the occurrence of guttering in South African coal mines is stress-related. To eliminate, control, or minimise the risk of rockfalls, it is necessary to understand the failure mechanisms associated with roof guttering. It may then be possible to determine ways of containing or obviating such failures. The consequence would be safe and maximised extraction of the coal reserves.

3.1 Observations of roof guttering

Mapping carried out in a shallow coal mine at a depth of about 50 m below surface showed that surface features observed along a guttering roof included the "upturned boat-shaped" geometry of a gutter, interlocking fracture surfaces towards the centre of a gutter, thin fractured slabs from centre of gutter going outwards and a heavily fractured surface ahead of a gutter (Ndlovu and Stacey, 2007; Ndlovu and Stacey, 2005). Figure 4 illustrates such geometry, the fractured surface and the tip of a gutter.

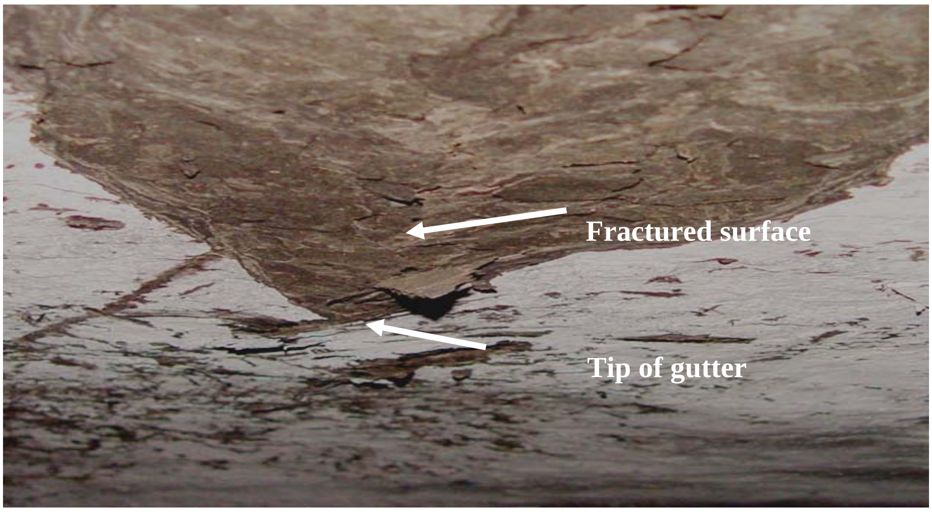

Figure 4 Upturned boat-shaped nature of guttering occurrence

Figure 5 illustrates the fractured surface ahead of a gutter. On scaling down the fractured surface ahead of the gutter, roof falls occurred and led to the gutter developing further longitudinally, laterally, and in depth. After a few hours or days another fracture surface may develop ahead of the gutter. The process repeats itself until the gutter stabilises. The observed size of gutters ranged from about 2 m in length, 0.5 m in width, and 0.1 m in depth to about 70 m in length, 3.5 m in width and 0.5 m in depth. About 80% of gutters were observed to occur towards the centre of the roadways, the remaining 205 being located at the pillar-roof contact.

Figure 5 Stress induced fracturing ahead of a gutter. The roof gutter occurred towards the centre of a roadway

Figure 6 shows the extent of guttering as mapped in one of the sections of the mine. Guttering was observed to occur predominantly in two perpendicular directions. These corresponded approximately with the horizontal principal stress directions (note that the two principal stress magnitudes are similar) determined from in situ stress measurements carried out in the area (Munsamy, 2003; Walker and Altounyan, 2001).

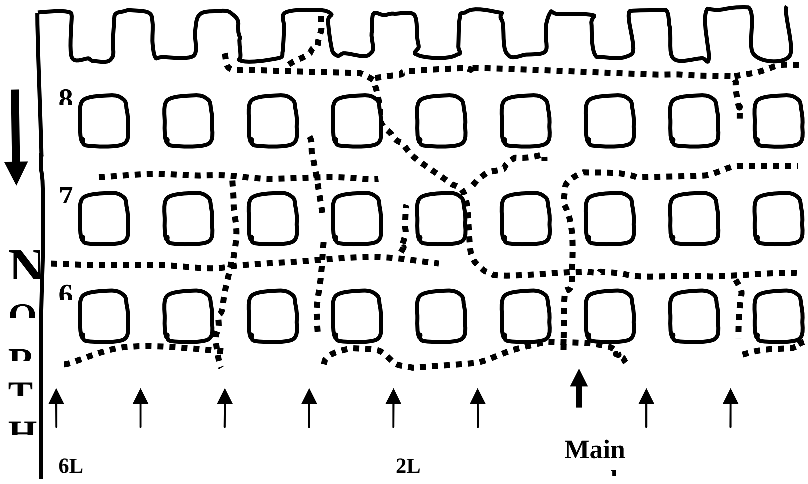

Figure 6 Occurrence of guttering in part of a section. Dotted lines represent the gutters. 2 L, 6 L etc. denote roadways and the numbers 6, 7, and 8 denote splits (development from the roadways)

In some areas, initiation of guttering took place within seconds of blasting while in others it occurred a few days after mining of the particular area. It often occurred violently, with an explosive sound, and with little warning. It was also observed that the extent of guttering progressed with time. In most instances it took several roof falls over several days for the gutter to stabilise. Guttering progression was most evident from the mining faces to about three splits back. Stresses could possibly have re-equilibrated in the back area after the guttering process, as evidenced by lack of active guttering in this area.

3.2 Possible mechanisms involved in the formation of roof gutters

The occurrence of guttering is clearly associated with high stress levels. In situ stress measurements show that the major horizontal stress is substantially higher than the vertical stress. From the available results, horizontal to vertical stress ratios of 5.7 and 4.2 were interpreted.

To study the possible mechanisms involved in the formation of roof gutters, fracture surfaces were observed with the aid of a microscope. Thin slabs obtained from different areas in the guttering roof were used. One hundred photo images were captured during this analysis. Two of these photographs are illustrated below. Figure 7 shows a fracture surface, which is believed to be of the extension type. There are no visible shear effects. Extensional type fractures were observed in about 90% of the images. Shear type fractures were observed in a few samples (about 10%), mainly at the edges of the slabs. It is possible that these apparent shear effects were due to secondary shear failure involved in the falling of rock fragments during the process of guttering. Figure 8 shows a fracture surface which is believed to be of the shear type. The visible white surfaces appear to be caused by relative movement during the shearing process.

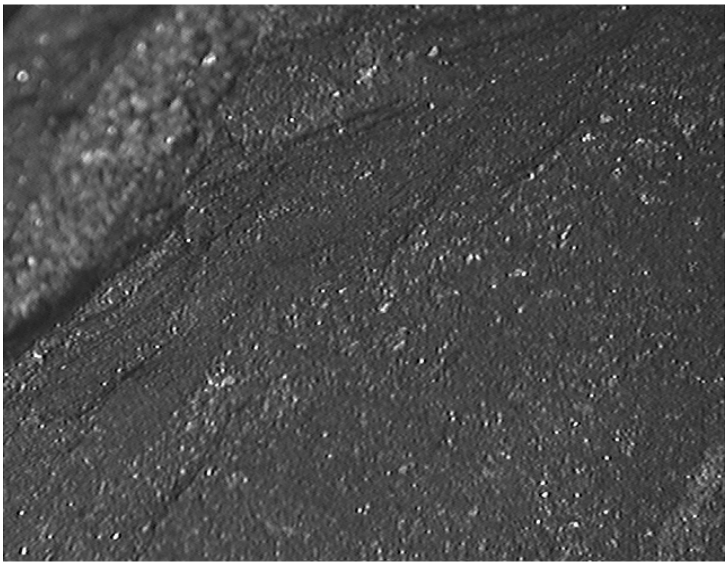

Figure 7 Photograph of an extension fracture surface

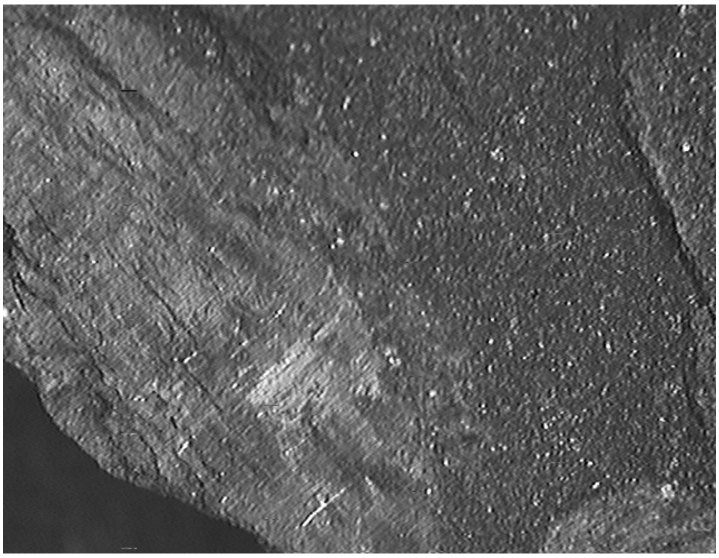

Figure 8 Photograph of a possible shear fracture surface

From underground observations it was evident that guttering involved progressive failure of brittle rock. The observations of the failure of rock during the process of guttering in the coal mine show that the behaviour is similar to the progressive brittle failure of rock discussed by Martin (1990, 1997), Martin et al. (1997, 1998), Martino and Chandler (2004). Similarities are derived from the following observations:

- Guttering involved failure of intact rock which may be explained by stresses in the region of failure being greater than the strength of the rock in this region.
- The high horizontal compressive stresses in combination with a weak, but unjointed mudstone roof and low compressive stresses in a vertical direction could be the main cause of the observed extension type of fractures. Extension fractures could be initiating the process of roof guttering.
- Guttering cannot be a result of high stresses alone; if this was the case it is expected that guttering would initiate in the corners of the roadways, where induced stresses are greatest.
- Occurrence of guttering was not a once-off event. Several falls of crushed rock were necessary for the gutter to stabilise. It is possible that the mechanism of confinement described by Martin (1997) caused the stabilisation of gutter progression.

- It is possible that guttering initiated in the region of maximum compressive stress concentration and propagated away from it. This is because no clear pattern of the geometrical location of the occurrence of guttering was observed. In some areas, within a roadway, guttering was observed to occur on one side of the roadway, then suddenly crossed the centre of the roadway and moved to the other side of the roadway.
- Scaling down favoured the progression of gutter formation. Removal of loose material, such as material previously held in place by the steel mesh support, usually helped guttering to develop further in depth, width and length.

3.3 Laboratory testing

Laboratory testing of the roof rock was carried out to obtain material strength and deformation properties for numerical modelling, with the object of attempting to explain the guttering process. Tests included the following:

- Ultrasonic tests on both cylindrical and cubic samples with loading perpendicular and parallel to bedding.
- Uniaxial compression tests on both cylindrical and cubic samples with loading perpendicular and parallel to bedding.
- Brazilian tensile tests on cylindrical cores with loading perpendicular and parallel to bedding.

Table 1 Results of laboratory tests

	Cylindrical specimen		**Cubic specimen**	
	Parallel	Perpendicular	Parallel	Perpendicular
Density, ρ (kg/m^3)	2350			
Dynamic E (GPa)				
range	16.7–19.8	10.6–11.4	19.0–20.7	9.3–9.8
mean	18.4	11	20	9.5
Static E (GPa)				
range	9.17–13.59	1.6–2.0	-	-
mean	11.3	1.8	-	-
Static v				
range	0.17–0.43	0.10–0.18	-	-
mean	0.27	0.16	-	-
UCS (MPa)				
range	23.28–28.15	37.18–40.95	23.81–28.01	36.89–39.25
mean	25.31	39.15	25	37.8
Tensile strength (MPa)				
range	0.76–1.42	2.35–3.11	-	-
mean	1.07	2.81	-	-

From the laboratory testing programme it is clear that the rock is anisotropic. The following mean values were used for numerical modelling: 11.3 GPa for Young’s modulus, 0.27 for Poisson’s ratio, 25 MPa for UCS and 1.07 MPa for tensile strength. Other input parameters, for example, friction angle, dilation angle and cohesion were estimated.

3.4 Numerical Analyses

Numerical stress analyses were carried out using a FLAC 3D model (Ndlovu, 2006). The bord and pillar widths modelled were 7 m and 9 m, respectively, and the modelled pillar height was 2 m. The modelled depth was 52 m below ground surface. Two model geometries were considered for the analyses:

Model 1 - a quarter of the bord and pillar geometry was used for modelling, taking advantage of the symmetry of the excavation geometry and the loading.

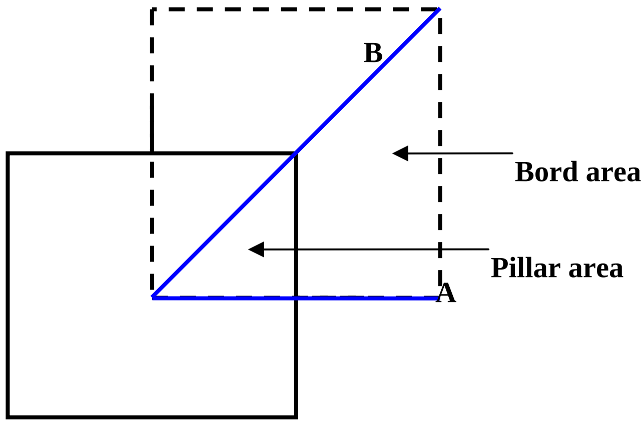

Figure 9 Plan view showing the bord and pillar quarter geometry

Model 2 – the geometry shows a coal heading. Loading is symmetrical. The model is shown in plan view in Figure 10.

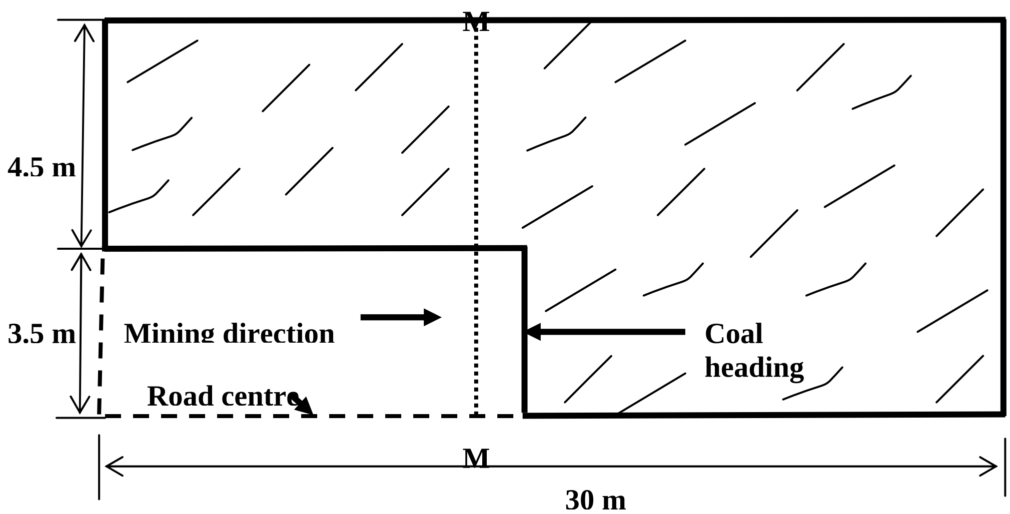

Figure 10 Plan view of model geometry with a coal heading

Three model analyses were carried out for each of several different material behaviours, as described below. These model analyses were the two models above and model 1 with an interface at the coal-shale contact. The numerical modelling results indicated that there were insignificant stress concentrations towards the centre of the roadway for elastic and transversely isotropic elastic models. Stress concentrations were always predicted at the roof-pillar contact area, and it therefore could be expected that failure should initiate and occur in this area. However, this does not correspond with underground observations of the guttering. Mohr-Coulomb and Mohr-Coulomb strain softening models predicted shear failure at the roof-pillar contact area. The predicted depths of failure ranged from about 0.2 m to about 1.5 m while the predicted widths of failure ranged from about 0.4 m to about 1.1 m. The observed underground failures ranged from about 0.1 m to 0.5 m in depth and from about 0.5 m to 3.5 m in width. The two models, therefore, over predicted the depth and under predicted the width of failures. Extension strain analyses predicted failure mainly at the roof-pillar contact area. The depths and widths of failure were successfully predicted. In contrast with the predictions from model 1, in model 2, high extension strain values were predicted at the centre of the road position *ahead* of the coal face. It is therefore probable that fracture initiation could be taking place in the roof rock ahead of the coal face. The mechanism of failure could then be as follows: on blasting, the rock that has been fractured due to the induced extension strains fails and, in the process, a gutter is formed at the centre of the road. Predictions of both location and extents of failure are therefore much more credible using the extension strain criterion than using the commonly-used rock mass strength failure criteria.

4 Dynamic failure of the roof in a very shallow coal mine

A case of dynamic failure of roof rock in a coal mine that can rightly be termed a rockburst, owing to the violence of its occurrence, has been described by Ortlepp and Moore (1987). As in the previous two cases, rock failure was associated with a loud noise. In this case mining was at a depth of only 22 m. The roof rock involved was very fine-grained and anisotropic carbonaceous shale. Its UCS normal to the stratification was about 65 MPa and its tensile strengths normal and parallel to the stratification were, respectively, 2 and 4 MPa.

The geometry of the failure is shown in the plan and section in Figure 11. The failure took place during top-coaling (removal of the top slice of the seam as can be seen in Figure 11), the primary board and pillar extraction having been completed some months before. A roofbolt hole was being drilled at the time of the failure.

Figure 12 shows a photograph of the failure area after it had been resupported with rock bolts. Figure 13 shows the detail of a near-perfect arc which defines the transition from an extension fracture spreading smoothly at an increasing propagation velocity to fracturing of a hackly nature. The latter occurs to provide more surface area to accommodate the excess energy release – radial traces are formed along which two or more fractures develop out of the plane of the original fracture.

Ortlepp and Moore (1987) studied the mechanics of the failure process and estimated a value of 1.1 MPa for the tensile stress normal to the crack plane at the time the crack front reached the uniform-hackly boundary. This was reasonably close to the tensile strength normal to the stratification measured in the laboratory.

Ortlepp and Moore (1987) stated that the fact that there appeared to be several initiating flaws was difficult to explain since they expected a brittle fracture to have a unique starting point. An explanation might be given, however, by referring to the second case study above - the numerical analyses carried out showed that there will be a relatively uniform distribution of extension strain in the roof above an intersection, and that the strain levels are likely to be high enough to initiate fracturing. Fractures may therefore have initiated parallel to the stratification at several localised “weaker” areas.

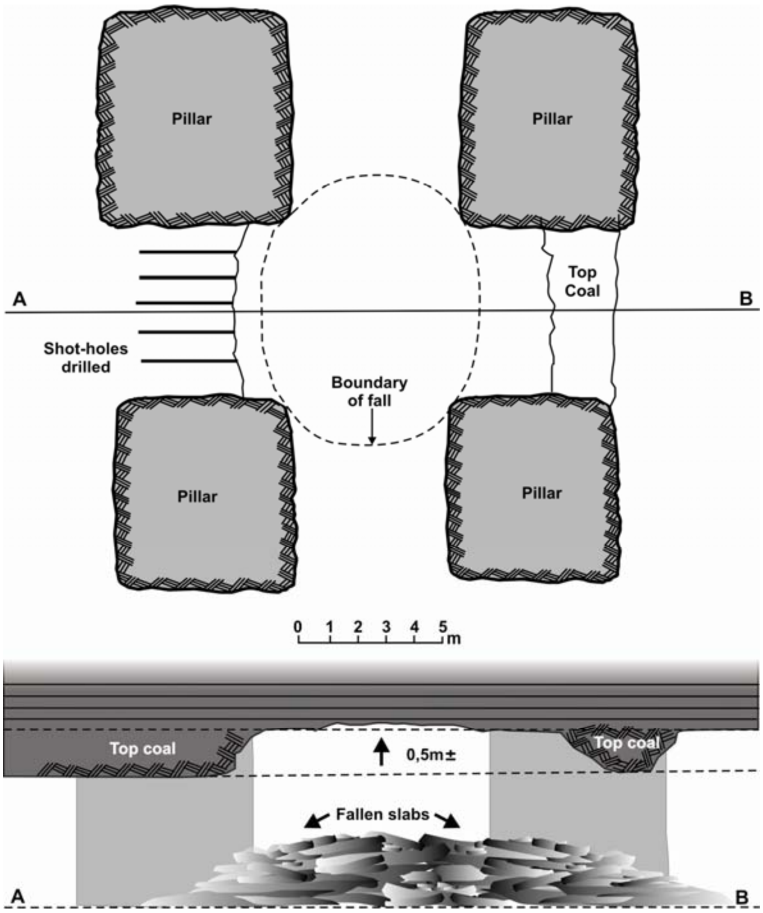

Figure 11 Plan and section of roof failure

Figure 12 General view of the roof failure

Figure 13 Detail of fracture propagation

5 Discussion and conclusions

The three case studies presented in this paper have shown that depth is not a must for high stress conditions. The uniaxial compressive strengths of the rocks, as would commonly be measured in laboratory tests, ranged from about 40 MPa to more than 200 MPa. In two of the cases the rocks were anisotropic, with the result that both compressive and tensile strengths were about half the values that would commonly be measured on core drilled normal to the stratification. Depths below surface varied from a few metres to a maximum of about 50 m. In all cases, owing to the shallow depth and "massive", unjointed rock involved, normal engineering intuition would probably lead to the immediate conclusion that no problem of stability would be present. However, all three cases involved violent failure at the very shallow depth conditions. In all cases the fracturing observed in the rocks was extensile in nature.

The three case studies show that it is essential to take into account the "correct" mechanisms of failure initiation and development, and then appropriate failure criteria, if such failures are to be predicted correctly.

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