Extensional Deformation Triggers for Potential Composite Failure of Bedded Rock Slopes

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Abstract

Composite failure of bedded rock slopes involves development of a rock mass collapse mechanism through a combination of pre-existing rock defects with stress-induced fracturing of rock substance. In open cut coal mines, excavation processes for slopes in bedded rock inevitably induce some rock substance damage. Previous investigations of composite failures have highlighted shortcomings in predictive capabilities based on current rock mass strength models. Stacey's Extension Strain Fracture Criterion appears to be a reliable indicator for the onset of rock substance damage that is the precursor to composite failure development.

A field experiment was undertaken at Coppabella Mine in the Bowen Basin coalfield of central Queensland to investigate whether precursor rock substance fracturing could be detected by microseismic monitoring. The outcomes of this experiment are described, and it was concluded that direct detection of trigger-level fracturing is a promising concept but not practical under field constraints. Movements of the excavated wall were also monitored by radar. Numerical modelling of the slope movements was undertaken using simplified commercial-standard rock mass modelling techniques to demonstrate movement levels associated with joint articulation and rock mass damage processes that do not progress to collapse. Such movement levels should be considered as indicators of stable response, with movements in excess of these levels constituting triggers for potential collapse mechanism development. Systematic slope movement monitoring has to be able to reliably resolve movements of 2mm to 5mm in order to be able to track potential failure mechanism development.

1  Background

Composite failure of a rock slope involves the progressive development of a collapse mechanism through a combination of pre-existing defects with stress-induced fracturing of rock substance. In bedded rock masses, the defects consist of sedimentary and bedding partings, joints, and faults. The rock mass is likely to have variable deformability due to the range of stiffnesses of the component material lithologies and the range of articulation associated with the defect network.

Rock slopes in open cut coal mines are excavated sequentially by a variety of methods, resulting in slope profiles that typically consist of multiple batters and benches. Overall slope profile heights of the order of 300m or greater are either being excavated or planned. Composite failure mechanisms may develop in such slopes at scales ranging from a single batter to the entire profile.

Industry-sponsored research has been undertaken in Australia to investigate composite failures in coal mine open pit slopes. Part of that research has involved reviewing pit slope failures to understand the characteristics of composite mechanisms, and numerical modelling aimed at understanding the roles of rock mass blockiness and extensional deformations. These aspects are briefly reviewed below in order to provide the context for a large-scale field experiment aimed at identifying pre-collapse deformation trigger levels for composite mechanisms.

1.1  Methods available for prediction of composite failures

Predictions involve judgements related to uncertainties but also rely on methods of analysis of the behaviour of rock masses that can be subdivided into two basic approaches: stability assessment based on limit equilibrium techniques applied to forces and collapse mechanisms, and deformation-based assessment based on stress-strain or force-displacement relationships for elements of the rock mass. The spectrum of analytical methods available
to designers ranges from state-of-practice to state-of-the-art. Simmons and Simpson (2006) expressed the opinion, based on a review of open pit coal mine composite failures, that the incidence of such failures had not changed significantly over a generation of open cut coal mining experience that included widespread adoption of sophisticated computational tools for applying analytical methods.

Limit equilibrium stability assessment is the most widespread analytical method used in practice. This is based on simplified, generally 2D potential collapse mechanisms and rock mass strength obtained either from backanalysis (with its associated assumptions) or by empirical determination. Easy accessibility of the Generalised Hoek-Brown (GHB) strength model through the RocLab freeware (RocScience, 2007) has popularised empirical nonlinear shear strength models with simplified linear Mohr-Coulomb (M-C) approximations, but has introduced into practice some significant risks associated with inappropriate application.

It is widely recognised that tension cracking and associated extensional deformation under low stress conditions lead to significant deviations from either the GHB or M-C strength envelopes because of brittle spalling processes (Kaiser and Kim, 2008). Methods for representing the strength of a brittle-spalling zone have been available for some time (Martin et al, 1999) but constitutive models suitable for stress-deformation analysis are not yet available in practice. Simmons and Simpson (2007) established that brittle strength response, with extensional strains much greater than Stacey’s (1981) Extension Strain Fracture Criterion (ESFC), was inevitable in typical open pit coal mine excavations. Stress-deformation modelling incorporating some measure of extensional fracture features has provided important insights into propagation of fracturing to form discrete blocky collapse mechanisms (Stead et al, 2006) and even fundamental geological processes such as joint development in bedded rock (Stefanizzi et al, 2007).

Despite recent advances in computational power, the state-of-practice for design is still dominated by assessments based on 2D limit equilibrium analysis, which are not able to directly account for extensional strain fracture and brittle spalling behaviour which can occur with or without leading to collapse.

1.2 Stress and deformation states in bedded rock masses

Composite failure mechanisms in bedded rock masses involve a combination of rock material and rock defect responses to loading or unloading events. It is therefore reasonable to assume that prediction of composite failure requires modelling that recognises stress and deformation states in bedded rock masses that are variably blocky depending on the nature of across-bedding joints and other geological structures. Cundall (2008) described the development of the synthetic rock mass approach to rock mass modelling as an extension of numerically-based discrete or particulate mechanics. While such models are undoubtedly powerful, realistic, and customisable, they are not routinely accessible to practitioners for design purposes.

The commercially available geotechnical modelling code Phase2 (RocScience, 2008) includes a facility for explicitly representing joint networks in rock masses. While Phase2 can simulate brittle strain-weakening strength behaviour it does not have a facility for directly representing brittle spalling response to extensional strain fracturing, in contrast to hybrid codes such as Elfen (Stead et al, 2006). For practical design purposes, Phase2 is the most accessible code that has the capacity to model, by bracketing, the rock mass behaviours that are believed to be essential for composite failure mechanisms.

1.3 Objectives of field-based investigation of pre-failure triggers

There is a gap between conceptual models of extension strain fracture and documented rock slope behaviour for several reasons. Pre-collapse deformations are so small as to be virtually unobservable without precise movement monitoring, and composite failure mechanisms are in practice comparatively rare. Because risk to mine workers is measured as a combination of consequence and likelihood, and the consequences of unexpected collapse are potentially catastrophic, risk assessments for excavating in close proximity to suspected unstable walls often result in requirements for precise movement monitoring. Setting of appropriate trigger levels for alarms can be problematic, and is largely empirical in the absence of reliable information.
Industry-funded research has been conducted in the Bowen Basin coalfield of central Queensland with the aim of providing reliable and consistent alarm trigger information for monitoring of potential composite mechanism hazards. The objectives of the field experiment component of the research were to:

- Obtain evidence for pre-collapse extensional fracture processes by deploying a microseismic array;
- Locate extensional strain fractures in both time and space with respect to the progress of excavation, and in particular seek an explanation for the common observation that a large proportion of composite failures occur during the mining of coal at the base of excavated wall profiles;
- Simultaneously obtain evidence for wall movements during the mining process by deploying a radar;
- Explain the patterns of fracturing and movement observed by modelling the excavated wall profile taking into account rock mass yield processes including extensional strain fracturing.

Figure 1. Location map showing Coppabella Mine within Bowen Basin coalfield of central Queensland.

2 Coppabella mine field experiment

Following earlier phases of the research project it was initially planned to conduct the field experiment at Saraji Mine. Due to the complexities of scheduling installation of a microseismic array in close proximity to operating draglines and shovels it became necessary to relocate to another regional site, Coppabella Mine (Figure 1).

2.1 Geological setting

Coppabella Mine is located within the Bowen Basin coalfield on the south-western limb of a shallow north-plunging regional structure known as the Nebo Synclinorium. The target coal is the Leichhardt Seam of the late-Permian Rangal Coal Measures (Mutton, 2003), which is locally about 10m thick and commonly hosts igneous
sill intrusions of Jurassic or Cretaceous age. At Coppabella mine the coal seam typically dips at between 2° and 8° towards the north-east. Cover materials include up to 40m of Neogene alluvium which is water-bearing and poorly lithified and underlain by weathered and fresh coal measures comprising lower alluvial floodplain sediments of medium and high substance strength. Depth to the base of weathering is between 40m and 50m. The rock mass is jointed as a response to erosional unloading and is also fractured by joints and faults associated with earlier transpressive and transtensional tectonic events.

2.2 Geotechnical conditions

Overburden at Coppabella is stripped by a combination of shovel-truck and dragline uncovering of coal in 60m wide strips which are oriented approximately parallel to seam strike. Prestrip excavation consists of multiple 15m high benches while the dragline pass is a single presplit batter approximately 40m high. Material for excavation by dragline is cast-blasted, and this in combination with presplitting of highly structured rock often causes significant damage to the highwall face.

The experiment was located at Johnson Pit South Strip 16 (JPS16) about 200m to the south of a lowwall access ramp. Figure 2 shows the profile of density and rock substance strengths measured on core samples from the fresh Permian rock interval. Above this interval the weathered coal measures and Neogene sediments grade upwards from low strength rock to clayey sands of hard to very stiff consistency.

![Figure 2. Coppabella Mine overburden density and Uniaxial Compressive (UCS) and Indirect Tensile (ITS) strength profiles for fresh rock sequence at Johnson Pit South Strip 16 (JPS16): note coal seam at 85m – 90m and carbonaceous/intrusive unit at 75m – 85m.](image)

Mapping of rock mass defects had been carried out for JPS16 at reconnaissance scale, so there was a reasonable but not precise understanding of thicknesses for lithological components and typical orientations and spacings of joints. Several high-angle faults were present in the vicinity of the experiment but these typically offset bedding vertically by 1.5m or less, and so a significant structure that intersected the experiment site was not confirmed until after the dragline had exposed the highwall batter. Several weakly cemented bedding contact surfaces were observed but bedding-parallel shear surfaces were not identified in logged core.
Rock mass strength and stiffness parameters were assigned based on geophysical data plus the results of core testing, correlations suggested in RocLab, and the application of experience and judgement. Table 1 lists the parameters used for stress-deformation modelling. Of particular note, the rock mass was assumed to be elastic-perfectly plastic in accordance with experience from backanalysis, and this implies that the dilatancy parameter \((D)\) should be set to zero. However, previous modelling experience (Simmons and Simpson, 2007) had shown that significant extension strain fracturing was inevitable under normal, stable excavation conditions. Without a constitutive modelling capability for brittle spalling development, pre-collapse extension strain fracturing was modelled by assuming a modest amount of post-peak strength reduction in combination with limited dilatancy. The parameters shown in Table 1 were determined from modelling simple slopes to achieve confined, dilatant yield without collapse development. Strip mining also involves sequential placement of dumped spoil materials. Strength parameters for dumped spoil were adopted based on industry experience while stiffness parameters were based on judgement including correlations with SPT tests and results of backanalyses.

Table 1. Rock mass and dumped spoil geotechnical parameters for JPS16 excavation modelling.

<table>
<thead>
<tr>
<th>Material</th>
<th>Unit</th>
<th>(\gamma) (MN/m(^3))</th>
<th>(E) (MPa)</th>
<th>(\nu)</th>
<th>(t^{(1)}) (MPa)</th>
<th>(c_{p,r}^{(2)}) (MPa)</th>
<th>(\phi_{p,r}^{(2)})</th>
<th>(D^{(3)})</th>
</tr>
</thead>
<tbody>
<tr>
<td>Tertiary Alluvium</td>
<td></td>
<td>0.022</td>
<td>200</td>
<td>0.25</td>
<td>0.005</td>
<td>0.075, 0.075</td>
<td>30.0°, 30.0°</td>
<td>0°</td>
</tr>
<tr>
<td>DW CM Rock(^{(4)})</td>
<td></td>
<td>0.024</td>
<td>2500</td>
<td>0.25</td>
<td>0.050</td>
<td>0.120, 0.120</td>
<td>35.0°, 35.0°</td>
<td>0°</td>
</tr>
<tr>
<td>DW CM ESFC(^{(5)})</td>
<td></td>
<td>0.024</td>
<td>2500</td>
<td>0.25</td>
<td>0.010</td>
<td>0.120, 0.067</td>
<td>32.0°, 32.0°</td>
<td>5°</td>
</tr>
<tr>
<td>Fr CM1 SST-SLST(^{(4)})</td>
<td></td>
<td>0.024</td>
<td>4000</td>
<td>0.25</td>
<td>0.075</td>
<td>0.450, 0.450</td>
<td>42.0°, 42.0°</td>
<td>0°</td>
</tr>
<tr>
<td>Fr CM1 (ESFC)(^{(5)})</td>
<td></td>
<td>0.024</td>
<td>4000</td>
<td>0.25</td>
<td>0.015</td>
<td>0.450, 0.225</td>
<td>42.0°, 42.0°</td>
<td>5°</td>
</tr>
<tr>
<td>Fr CM2 Carb.SLST(^{(4)})</td>
<td></td>
<td>0.024</td>
<td>2500</td>
<td>0.25</td>
<td>0.050</td>
<td>0.350, 0.350</td>
<td>35.0°, 35.0°</td>
<td>0°</td>
</tr>
<tr>
<td>Fr CM2 (ESFC)(^{(5)})</td>
<td></td>
<td>0.024</td>
<td>2500</td>
<td>0.25</td>
<td>0.010</td>
<td>0.350, 0.175</td>
<td>35.0°, 35.0°</td>
<td>5°</td>
</tr>
<tr>
<td>Fr Coal(^{(4)})</td>
<td></td>
<td>0.015</td>
<td>1500</td>
<td>0.15</td>
<td>0.025</td>
<td>0.035, 0.035</td>
<td>35.0°, 35.0°</td>
<td>0°</td>
</tr>
<tr>
<td>Fr Coal (ESFC)(^{(5)})</td>
<td></td>
<td>0.015</td>
<td>1500</td>
<td>0.15</td>
<td>0.005</td>
<td>0.035, 0.025</td>
<td>35.0°, 35.0°</td>
<td>5°</td>
</tr>
<tr>
<td>Spoil Cat.3U(^{(6)})</td>
<td></td>
<td>0.018</td>
<td>100</td>
<td>0.25</td>
<td>0.010</td>
<td>0.050, 0.050</td>
<td>30.0°, 30.0°</td>
<td>0°</td>
</tr>
<tr>
<td>Spoil Cat.3S(^{(6)})</td>
<td></td>
<td>0.020</td>
<td>75</td>
<td>0.25</td>
<td>0.005</td>
<td>0.020, 0.020</td>
<td>25.0°, 25.0°</td>
<td>0°</td>
</tr>
</tbody>
</table>

Notes: (1) \(t\) = rock mass tensile strength
(2) \(c, \phi\) = equivalent Mohr-Coulomb strength parameters for (p)eak and (r)esidual conditions
(3) \(D\) = dilatancy angle as defined in Phase2 code documentation
(4) \((C)\)oal \((M)\)easures rock subdivided into DW = distinctly weathered, SST-SLST = interbedded siltstone-sandstone, Carb SLST = carbonaceous siltstone, and Coal
(5) ESFC = parameters adjusted to simulate effects on rock mass of confined extensional fracturing
(6) Strength and stiffness parameters for spoil materials based on Simmons and McManus (2004).

The geotechnical model for JPS16 also incorporated joint networks consisting of bedding surfaces and cross-joints. Strength parameters for these interfaces were based on limited shear strength testing from other sites in similar materials. There was no information on interface stiffness parameters but field observations typically indicate relatively very stiff pre-peak response of both bedding and cross-joints. These parameters were selected based on numerical experiments to simulate typical “lipping” movements that are observed in excavated coal measures rock walls. Interface defect strength and stiffness parameters are listed in Table 2.

No site information was available regarding initial stress conditions and groundwater pressure distributions within the rock mass surrounding the pit. Based on a regional understanding of virgin rock stress conditions (Hillis et al, 1999) and accepted practices, initial principal horizontal stress ratios from 2.0 to 0.5 were considered. Groundwater pressures were modelled by 2D finite element analysis with far-field head levels at the
base of weathering drawn down to the pit floor and restored to about 5m above the pit floor in spoil backfill (Simmons and McManus, 2004). Reasonable assumptions were made regarding rock mass permeability characteristics based on experience with backanalyses of multiple-level piezometers.

Table 2. Rock interface geotechnical parameters for JPS16 excavation modelling.

<table>
<thead>
<tr>
<th>Interface</th>
<th>Region(1)</th>
<th>$k_N$ (2) (MPa/m)</th>
<th>$k_S$ (2) (MPa/m)</th>
<th>$t$ (3) (MPa)</th>
<th>$c$ (4) (MPa)</th>
<th>$\phi$ (4)</th>
<th>GW (5)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Bedding A(6)</td>
<td>1 - 6, 8</td>
<td>250000</td>
<td>100000</td>
<td>0.100</td>
<td>0.050</td>
<td>30.0°</td>
<td>Y</td>
</tr>
<tr>
<td>Cross Joint B</td>
<td>1 - 6, 8</td>
<td>250000</td>
<td>100000</td>
<td>0.025</td>
<td>0.025</td>
<td>30.0°</td>
<td>Y</td>
</tr>
<tr>
<td>Bedding C</td>
<td>7</td>
<td>250000</td>
<td>100000</td>
<td>0.025</td>
<td>0.035</td>
<td>30.0°</td>
<td>Y</td>
</tr>
<tr>
<td>Cross Joint D</td>
<td>7</td>
<td>250000</td>
<td>100000</td>
<td>0.005</td>
<td>0.025</td>
<td>30.0°</td>
<td>Y</td>
</tr>
<tr>
<td>Bedding T</td>
<td>9</td>
<td>250000</td>
<td>100000</td>
<td>0.005</td>
<td>0.000</td>
<td>25.0°</td>
<td>Y</td>
</tr>
<tr>
<td>Cross Joint U</td>
<td>9</td>
<td>250000</td>
<td>100000</td>
<td>0.005</td>
<td>0.000</td>
<td>25.0°</td>
<td>Y</td>
</tr>
</tbody>
</table>

Notes: (1) Region = joint network region as assigned to the different material units based on mapping data  
(2) $k_N$, $k_S$ = interface normal and shear stiffness respectively  
(3) $t$ = interface tensile strength  
(4) $c$, $\phi$ = interface Mohr-Coulomb sliding strength parameters  
(5) GW = indicator (Y)es or (N)o for inclusion of groundwater pressure in strength model for interface

2.3 Experimental design

The microseismic array was designed to locate local source events spatially to within a radius of 1m to 2m, taking into account the constraints of slope geometry and access restrictions during installation and operation. Layout of the array is shown in Figure 3 and a schematic cross-section is shown in Figure 4. A combination of 14 Hz and 4.5 Hz uniaxial and triaxial geophones were grouted into boreholes drilled after the presplit and cast-blasts had been fired. Event data was recorded with timing referenced to GPS signals. Four calibration test shots were fired which provided average P- and S-wave velocities of 2900±360 and 1350±270 m/s respectively. For comparison purposes, sonic log P-wave velocities for the rock materials ranged from 4920-3390 m/s for sandstone-siltstone, 2650-2170 m/s for carbonaceous siltstone, and 2250-1960 m/s for coal. Clearly the rock material velocities were attenuated by rock mass structure and this impacted on event interpretation.

While it would have been very informative, it was not practical to measure movements of the highwall face during all stages of mining. A Slope Stability Radar® unit (SSR, Groundprobe, 2004) was deployed to measure movements of the highwall commencing as soon as dragline stripping had cleared the highwall block containing the array. Movement measurement was therefore restricted to the last stage of the mining cycle: coal recovery.

2D finite element stress-deformation modelling was undertaken for a simplified JPS16 pit cross-section based on Figure 4. In practice the seam structure dip in JPS16 varied from about 5° to almost 0°. Modelling was therefore undertaken for two sections to evaluate the effects of these limiting dip values. Because of expected load-path dependent elastoplastic responses and the sequential nature of strip mine development, a total of eight loading stages were simulated in the models culminating in stripping and then coal recovery from JPS16.

3 Results from microseismic monitoring

3.1 Array performance

The array was operational from early December 2008 to mid-January 2009 with events recorded when triggered by four or more of the eight sensors. On 12 December a failure of the highwall batter occurred where a fault
intersected the highwall, removing the two geophones at the south front of the array and causing degradation of spatial resolution of detected events. From 2 January there were intermittent faults in data recording, so interpretation of array data was concentrated on the period between 2 December and 2 January. A total of 15303 events were recorded over this period, of which 14510 were attributed to noise from mining activity including 5 events due to blasting. All these events were removed from the interpreted set by band-pass filtering. While the four test shots were located well away from the array to achieve separation of P- and S-waves, the single booster shots in open holes resulted in poor discrimination of S-waves.

![Plan view of JPS16 field experiment site showing microseismic array and other features. Each mining block is 100m long by 60m wide.](image)

The 793 events generated by the rock mass were classified typically as low-frequency, low stress-drop responses and the effective detection limit was $M_L = -0.8$ with the maximum event having $M_L = 1.8$. Modes were predominantly extensional at lower magnitudes and shear at higher magnitudes. All events with $M_L \geq 1.0$ were located at a relatively large distance from the array. The JPS16 area is transected by several faults and the larger and more distant events were interpreted as caused by pit-scale stress changes on pre-existing structures.

### 3.2 Interpretation of array results

Figure 5 shows the history of all events over the interpretation period, and is annotated to show the correlation with mining activity, while Figure 6 shows the history for events detected within 80m of the array sensors. Dragline stripping commenced in front of the array on 2 December, initially by a shallow pass to chop the upper half of the batter and define the crestline and then a key pass to expose the full height of the highwall and about 15m of coal on the highwall side. The final or “blocks” pass exposed the full 60m width of coal in the strip.

Figures 5 and 6 show the close correlation of event frequency with stress change caused by excavation. It is also clear that the highwall instability occurred after about two days of precursor events. Locations of events close to
the array are shown in an oblique view in Figure 7, which shows that the location of event activity shifted after the failure along the fault. This is interpreted as further evidence that the microseismic events were generated by stress changes, associated in the case of the fault with collapse of a section of the highwall.

Figure 4. Cross-section view of JPS16 excavated highwall profile showing layout of array sensors.

Figure 5. Microseismic activity from all rock mass events over period 2 December 2008 to 2 January 2009.
Figure 6. Microseismic activity from rock mass events within approximately 50m of array over period 2 December 2008 to 2 January 2009.

Figure 7. Microseismic events within 100m of array showing clustering in front of fault before 12 December failure (blue-green) and behind fault after failure (lime-red).
4 Movement measurement results

4.1 Movement records

The SSR was deployed on 12 December at the time when the highwall failure was occurring and immediately before the start of coal recovery. Unfortunately reliable data collection was only possible over the periods 16-18 December and 23 December - 11 January due to hardware faults. Scanning of the highwall face required location of the radar on a dumped spoil lowwall bench. Radar discrimination of millimetre-scale highwall movements requires designation of a “stable” zone within the scanning field. Significant diurnal movements of the spoil bench, similar-scale movements of the stable zone, and the line-of-sight nature of radar scanning all introduce complexities into measurement interpretation.

In the immediate aftermath of the highwall failure the radar detected ongoing movements amounting to 25mm. The rate of movement accelerated and then stabilised. Return signal coherence maps were used to discriminate between rock mass movement and apparent movements due to seepage flows or dribbling of fine debris.

4.2 Interpretation of movement data

Radar measurements over the period of coal recovery were corrected for the effects of background movement rates which amounted to 13.7 mm/day. This is a substantial figure when scanning for highwall movements of a few millimetres.

Corrected daily movement rates for various regions of the highwall were extracted from the radar data. Figure 8 shows movement rates for the lower third section of the highwall. To the south-east side of the array, the radar detected larger movement rates in the region corresponding to the concentration of microseismic events after the highwall failure. Over the period of coal extraction the corrected radar movements ranged between 30mm on the north-west and 100mm on the south-east and were concentrated at the time when the last coal was removed from
the toe of the highwall. A second highwall failure developed about 150m to the south-east of the array within two days of full coal extraction.

5 Patterns of movement attributable to stable highwall yielding

Finite element modelling was carried out for dips of 0° and 5° and for a horizontal principal stress ratio $K = 2.0$. Model scenarios were (1) elastic and (2) elastoplastic but without joint networks, plus (3) elastic and (4) elastoplastic with joint networks. Additionally, the effects of (5) extension strain fracturing within a joint network were modelled together with varying principal stress ratio values of (6) $K = 1.0$ and (7) $K = 0.5$.

Modelled movements in response to coal mining are summarised in Table 3 and Table 4 for seam structure dips of 0° and 5° respectively. Coal seam dip has a controlling influence on movement magnitude, which is to be expected of a rock mass with continuous bedding surfaces and multiple cross-bedding joints. The predicted movements are very similar to those measured by the radar. Between 15% and 50% of the wall movements occurred in response to coal mining due to stiffness and strength contrasts. Models including extensional cracking based on ESFC concepts show a significant contribution to total movement.

Table 3. Highwall total face movements in response to coal mining, Dip=0°.

<table>
<thead>
<tr>
<th>Scenario (listed in text above)</th>
<th>Crest</th>
<th>Middle</th>
<th>Coal Roof</th>
</tr>
</thead>
<tbody>
<tr>
<td>(1) Rock mass, elastic, $K_0=2.0$</td>
<td>0.017</td>
<td>0.023</td>
<td>0.052</td>
</tr>
<tr>
<td>(2) Rock mass, elastoplastic, $K_0=2.0$</td>
<td>0.023</td>
<td>0.029</td>
<td>0.043</td>
</tr>
<tr>
<td>(3) Joint network, elastic, $K_0=2.0$</td>
<td>0.020</td>
<td>0.030</td>
<td>0.073</td>
</tr>
<tr>
<td>(4) Joint network, elastoplastic, $K_0=2.0$</td>
<td>0.069</td>
<td>0.066</td>
<td>0.079</td>
</tr>
<tr>
<td>(5) Joint network, ESFC-e/plastic, $K_0=2.0$</td>
<td>0.158</td>
<td>0.126</td>
<td>0.146</td>
</tr>
<tr>
<td>(6) Joint network, ESFC-e/plastic, $K_0=1.0$</td>
<td>0.017</td>
<td>0.014</td>
<td>0.033</td>
</tr>
<tr>
<td>(7) Joint network, ESFC-e/plastic, $K_0=0.5$</td>
<td>0.000</td>
<td>0.000</td>
<td>0.003</td>
</tr>
</tbody>
</table>

Table 4. Highwall total face movements in response to coal mining, Dip=5°.

<table>
<thead>
<tr>
<th>Scenario (listed in text above)</th>
<th>Crest</th>
<th>Middle</th>
<th>Coal Roof</th>
</tr>
</thead>
<tbody>
<tr>
<td>(1) Rock mass, elastic, $K_0=2.0$</td>
<td>0.015</td>
<td>0.019</td>
<td>0.055</td>
</tr>
<tr>
<td>(2) Rock mass, elastoplastic, $K_0=2.0$</td>
<td>0.003</td>
<td>0.004</td>
<td>0.007</td>
</tr>
<tr>
<td>(3) Joint network, elastic, $K_0=2.0$</td>
<td>0.020</td>
<td>0.032</td>
<td>0.076</td>
</tr>
<tr>
<td>(4) Joint network, elastoplastic, $K_0=2.0$</td>
<td>0.004</td>
<td>0.006</td>
<td>0.010</td>
</tr>
<tr>
<td>(5) Joint network, ESFC-e/plastic, $K_0=2.0$</td>
<td>0.022</td>
<td>0.024</td>
<td>0.036</td>
</tr>
<tr>
<td>(6) Joint network, ESFC-e/plastic, $K_0=1.0$</td>
<td>0.025</td>
<td>0.020</td>
<td>0.029</td>
</tr>
<tr>
<td>(7) Joint network, ESFC-e/plastic, $K_0=0.5$</td>
<td>0.021</td>
<td>0.023</td>
<td>0.032</td>
</tr>
</tbody>
</table>
6 Summary and conclusions

While the microseismic array was successful in providing clear evidence of extensional fracturing being a direct result of unloading by excavation, it is not a practical tool that can be deployed routinely to provide warning of instability. Based on the detected events, seismically-generated movements were concentrated towards the base of the highwall and occurred for up to two days prior to visually observable cracking and collapse.

Radar monitoring of highwalls also demonstrated the direct relationship between unloading and movement and confirmed at least two days of detectable response prior to visually observable cracking and collapse.

Pre-failure highwall movements include a significant contribution from extensional strain fracturing for which Stacey's ESFC is an invaluable predictive model. Small movements can be detected using radar but there will always be uncertainties associated with typical background movement rates versus movements accelerating to a failure process. Tables 3 and 4 above provide guidance for the rate-corrected movements that can be expected for a stable highwall. These movements, or more site-specific values, form the basis for appropriate risk management trigger levels. Because the potential movements are relatively small and undetectable by routine visual observation, measurement tools that can discriminate changes of 2mm to 5mm are required if movement triggers are to be monitored reliably.

7 Acknowledgements

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8 References


