Ground Support 2026 - D Cumming-Potvin & J Wesseloo (eds.)
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doi:10.36487/ACG_repo/2625_47

The rock in rock reinforcement: linking fracture mechanics, structural geology and ground support

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Abstract

A significant requirement for ground support design in dynamic situations is an understanding of how the rock fractures due to induced stress and seismic wave impact from a burst. This is often referred to as the bulking factor, although this paper will expand on the concept. The bulking factor governs the deformation experienced by the support system and is what the system is designed to manage by limiting deformation or controlling it. The bulking factor is defined by 2 mechanisms. The first is the extensional fracturing or spallation observed on the surface of the exposure, and the second is the apparent shearing of the rock adjacent to the spallation. Both these 'mechanisms' occur in a biaxial stress field where the induced stress is parallel to the surface and a response to the shear wave motion emanating from the burst. This story is incomplete and fails to capture the field observations of the remnant support bending before shearing. Surface-wave-induced block rotation provides one possible explanation for the observed bolt bending. Surface waves cause most of the damage from earthquakes in buildings and infrastructure. Surface-wave behaviour underground has received comparatively little attention, however. There is the observation that 80% of failures attributed to bursts occur at depths below the surface within the first 50 to 70% of the minimum opening dimension. Deeper failure, which is often more extensive along strike, is sometimes referred to as 'shakedown'. It may be attributed to the presence of structural control where the extent of failure is related to the size of the burst event, the rock shear wave velocity, the wavelength and the dominant frequency of the event. Without a comprehensive understanding of the structural geology, the location of these events is considered random. This paper will show that knowing how rock fails is a necessary component of ground support design and implementation.

Keywords: *biaxial, spallation, extensional failure, shakedown, stress measurement, strainburst, rockburst, shear wave velocity, Rayleigh wave, bulking, rock mechanics*

1 Introduction

This paper does not attempt to present a complete theory of strainbursting. Rather, it proposes an integrated framework linking field observations, laboratory testing, fracture mechanics, seismic response, structural geology and support behaviour. Each topic is briefly introduced to demonstrate how these components interact in support design. The purpose is not to provide a comprehensive treatment of each discipline but to illustrate how these disciplines interact during support design.

Dynamic support is used when seismic conditions are expected or already have been experienced. Seismicity is considered to arise from stress changes. Seismicity is also more prevalent as mines go deeper, although it has been observed at shallow depths (e.g. Dight et al. 2025). Hence a knowledge of the in situ stress conditions should be a prerequisite for examining such conditions.

In situ stress can be measured by several methods including Hollow Inclusion Cell, US Bureau of Mines cell, Hydraulic Fracture (HF), Compact Conical-ended Borehole Overcoring, acoustic emission (AE), deformation rate analysis (DRA), Sigra cell and Australia New Zealand Inflatable cell, to name a few. An example of an

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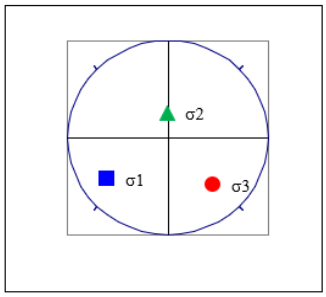
unbiased comparison between tests was conducted by de Andrade Penido et al. 2022 which reviewed 3 methods of HF, DRA and Diametrical Core Deformation Analysis to ascertain the better approach.

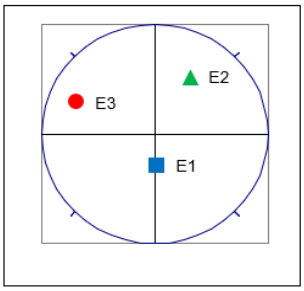
The stress changes induce deformation. As all our mechanical properties are directly or indirectly obtained from deformation, it stands to reason deformation should be considered in design. This can be achieved with numerical modelling.

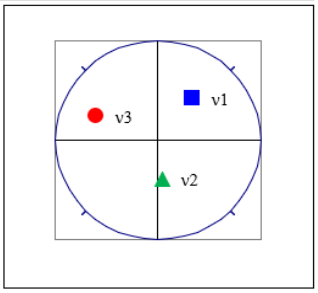
With numerical modelling we need realistic boundary conditions and geometry of openings and structure. These dominate any analysis.

Material strength should then be considered. In isotropic materials (common assumption) it is much simpler to obtain intact mechanical properties (strength, modulus, Poisson's ratio, Fracture Toughness, Brazilian tensile strength, P-wave and S-wave velocity). It requires more effort to obtain the 3D properties if the rock is anisotropic/orthotropic with most focus just on the strength characteristics. The latter condition is much more pervasive, around mineralised systems where metamorphism and foliation are often observed. Even intrusive rocks like dacite and diorite can show orthotropic behaviour. Figure 1 illustrates the point that assuming isotropic behaviour ignores reality.

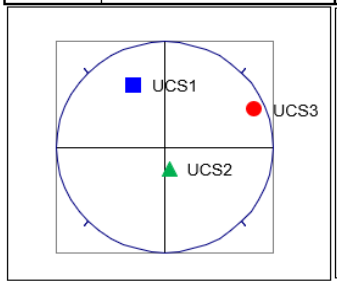
Stress	Trend	Plunge	MPa	Modulus	Trend	Plunge	GPa	Poisson's Ratio	Trend	Plunge	
σ1	234	17	46	E1	175	57	99	ν1	40	33	0.27
σ2	358	61	38	E2	31	27	83	ν2	173	46	0.23
σ3	137	22	25	E3	292	17	47	ν3	292	25	0.10
σ1:σ2:σ3	1.83 : 1.51 : 1.00			E1:E2:E3	2.10 : 1.76 : 1.00			ν1:ν2:ν3	2.72 : 2.30 : 1.00		

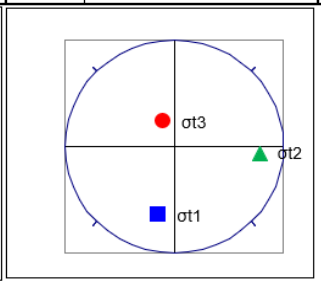


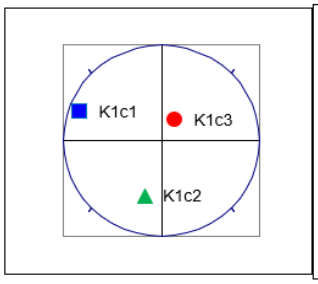


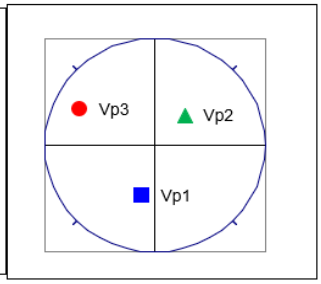


Strength	Trend	Plunge	MPa	BTS	Trend	Plunge	MPa	Fracture toughness	Trend	Plunge	MPa.m$^{1/2}$	V_p	Trend	Plunge	m/s
UCS1	334	24	285	σt1	192	23	43	K1c1	290	8	2.26	Vp1	193	38	6077
UCS2	170	66	261	σt2	95	14	19	K1c2	196	28	1.75	Vp2	45	47	5974
UCS3	67	6	171	σt3	337	62	17	K1c3	34	61	1.45	Vp3	296	16	4924
UCS1:UCS2:UCS3	1.67 : 1.53 : 1.00			σt1:σt2:σt3	2.53 : 1.12 : 1.00			$K1c_1$:$K1c_2$:K1c	1.56 : 1.20 : 1.00			Vp1:Vp2:Vp3	1.23 : 1.21 : 1.00		









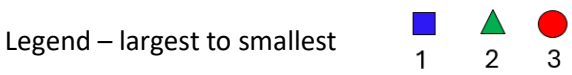


Figure 1 Example of some intact properties of a dacite. Note the modulus ratio is~2:1

2 Observations

This study is a work in progress. There are several field observations which warranted further explanation as identified in Table1.

The project was initiated when it was brought to our attention that ~ 80% of the damage from rockbursts experienced in the Sudbury basin were shallow as illustrated in Figure 2. Note that many of the bolts are bent and/or sheared. This raises a question: Are the bolts being subjected to flexural bending rather than 'shear' where the latter is assumed to comprise a defect surface only sliding? All grouted bolts and to a much lesser extent full contact bolts, bend during shearing (unlike a rivetted connection). However, if the bending is in random directions as shown in Figure 2, then the movement is not a consistent shear direction and possibly the movement is not 'shearing'.

Walls, backs, floors and faces in the near surface are loaded biaxially. For example, for a wall, the stresses to consider are the vertical stress and the stress acting parallel to the wall.

The stress setup is illustrated in Figure 3. If we take a prism of material from the sidewall, then we can load it biaxially in the laboratory. Such an example is presented in Figure 3. Here a mortar sample representing an isotropic material that had been tested is shown in Figure 4. Note that there is a free face where σ_3=0. The lettering on the sample was used to identify where shards of material were ejected during the test.

Table 1 Field observations warranting further investigation

	Field
1	Apparent depth of failure in 80% cases ~10% of minimum opening dimension
2	Bolts bend in near surface and do not shear
3	Holes offset in near surface
4	Near surface ejection >3 m/s
5	Shakedown – unexpected location/s

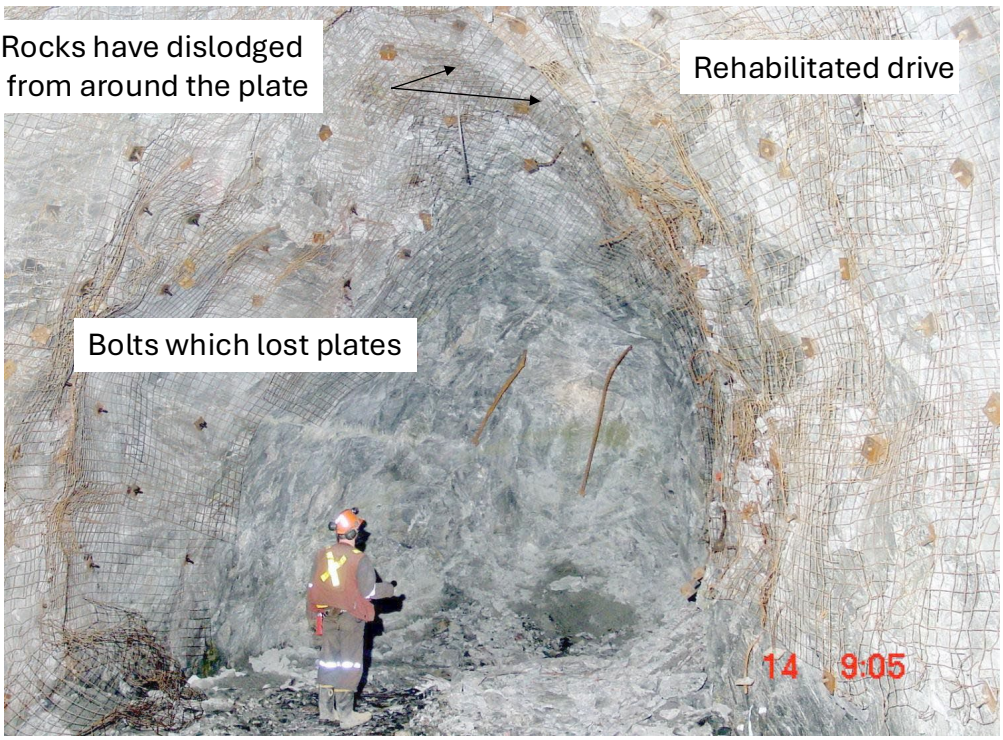


Photo courtesy Glencore Sudbury

Figure 2 Depth of visible damage typically <10% of minimum opening dimension

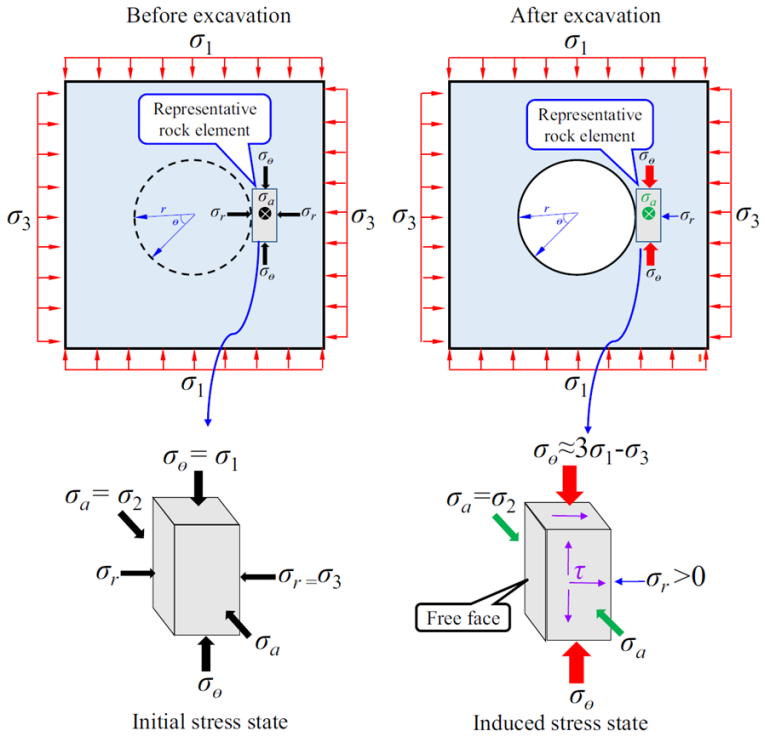


Figure 3 Biaxial stress condition acting on a wall

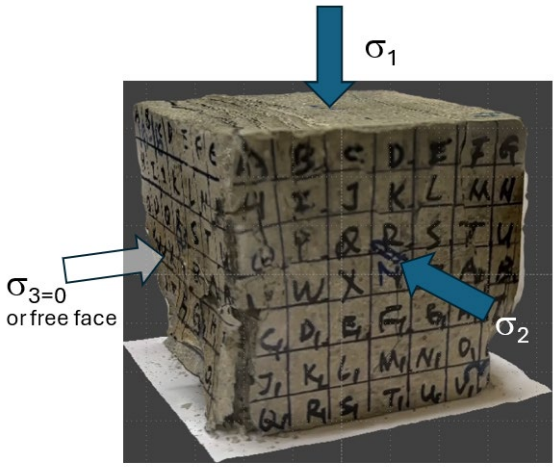


Figure 4 Stresses on a sample simulating a biaxial stress field with σ_3=0

The next piece of the puzzle was to identify a possible reason for the shallow depth. We believe this can be attributed to the locus of the minimum fracture toughness which had been identified by Sahouryeh et al. (2002) when testing prismatic samples of resin containing an induced flaw. Wang et al. (2021) then demonstrated that σ_2 need only be ~10% of the maximum induced stress for the crack to initiate all the way through the sample.

The set up and some results are shown in Figure 5 where the key elements are labelled. Figure 5a is the biaxial frame in an Instron 8035 120 t testing frame. An example of the spontaneous crack all the way through a resin sample is shown in Figure 5b. The key is the biaxial stress acting on the sample. The hackled surface of the crack morphology is shown in Figure 5c. The crack location is located at a depth behind the face corresponding to approximately 1/10th of the face dimension. For example, the crack would initiate at about 0.5 m behind a 5 m high drive. Independent research conducted by CSIRO found the crack at a depth of 0.63 m behind the face (Hoehn & Spearing 2023). This location coincides with the lowest fracture toughness in an isotropic material.

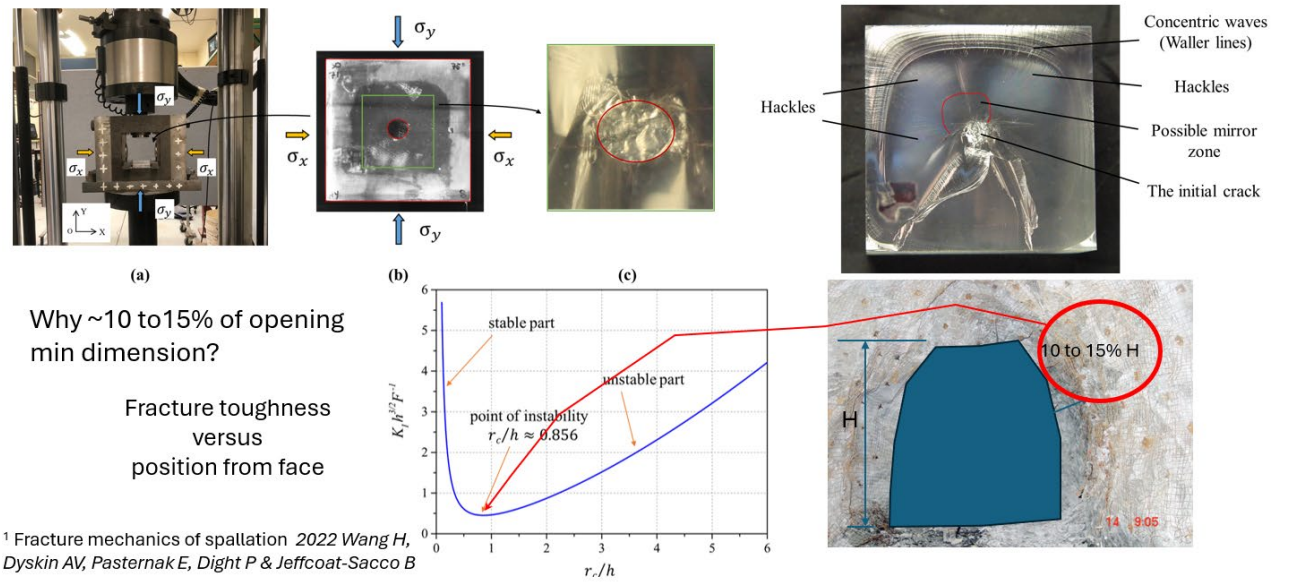


Figure 5 The location of the minimum fracture toughness is located between 10 and 15% behind the free face

The next challenge was to explain the observation that in strainbursts the bolts which remain appear bent and not sheared. A question was posed earlier whether the shearing is limited to planes of sliding. Clearly that is not the case, but the definition of shearing is loosely used. The bending mechanism is actually included in the rockbolt shear algorithm (Dight et al. 2025).

The bolts shown in Figure 1 are bent and not 'sheared'. The bending orientation is irregular – this observation has also been observed in the laboratory as shown in Figure 6.

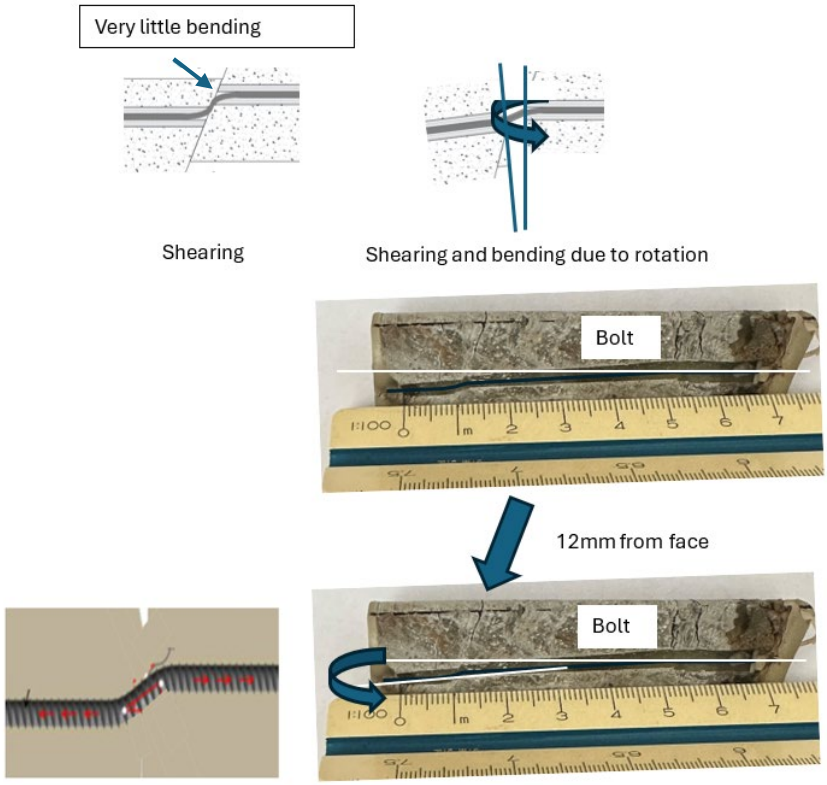


Figure 6 Example of shearing on the left and a bolt bending in the biaxial test on the right

Using Kirsch theory, the distance from the surface to where the radial stress is sufficient to induce shearing is about 30% of the minimum opening dimension. For a 5 m high drive, this would be approximately 1.5 m. Figure 7 shows the relationship using the Kirsch equations.

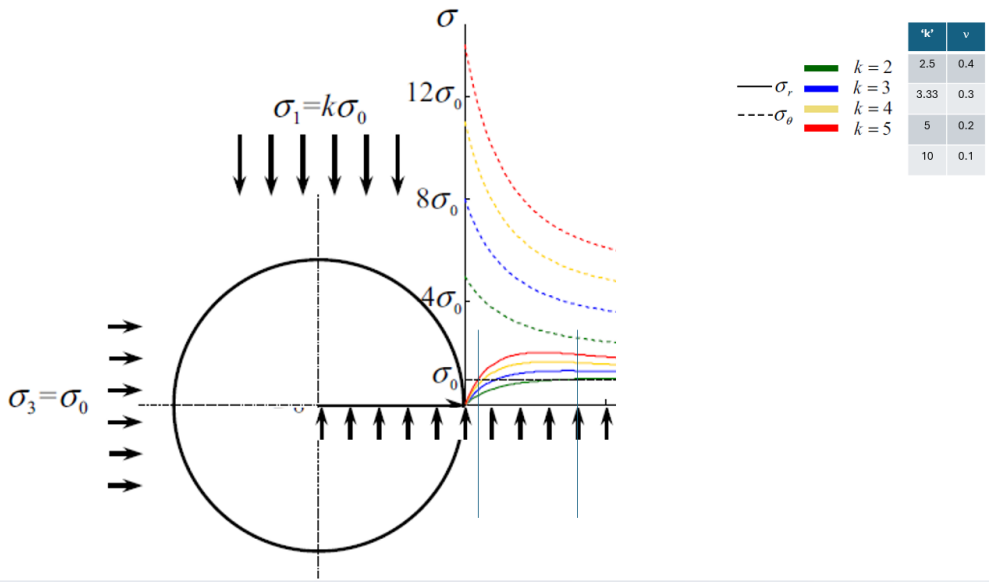


Figure 7 Distance behind the face to where shearing can occur

A possible explanation for the bending is that the surface materials rotate during failure. In a rockburst, this could be attributed to a surface seismic wave rather than the commonly assumed body wave. A Rayleigh wave travels at 0.9 times the shear wave but causes the most damage as witnessed in the damage occurring from an earthquake.

Figure 8 shows a picture of an offset on a drillhole. Initial consideration would be shearing but rotation of the blocks cannot be ruled out.



Figure 8 Block shearing or could it be block rotation and shearing (courtesy P Mikula 2026)

Another observation in the field is the poor collar encapsulation observed in fully encapsulated bolts. This is a significant problem for operations but is not replicated in the laboratory tests.

Similar ejection velocities appear in laboratory and field studies. These can be more than 3 m/s. In the laboratory the ejection at this speed is only near the surface and does not extend to depth. The mechanics of spallation may provide a suitable explanation (Wang et al. 2021).

3 Fracture mechanics and structural control

Mines which experience seismicity often install microseismic systems to record event location. The data is used to identify hazardous areas and improve ground support. In the laboratory AE can be used to serve a similar function (Figure 9). The sample size is 70 by 70 by 70 mm. The source location of the events recorded are shown in red.

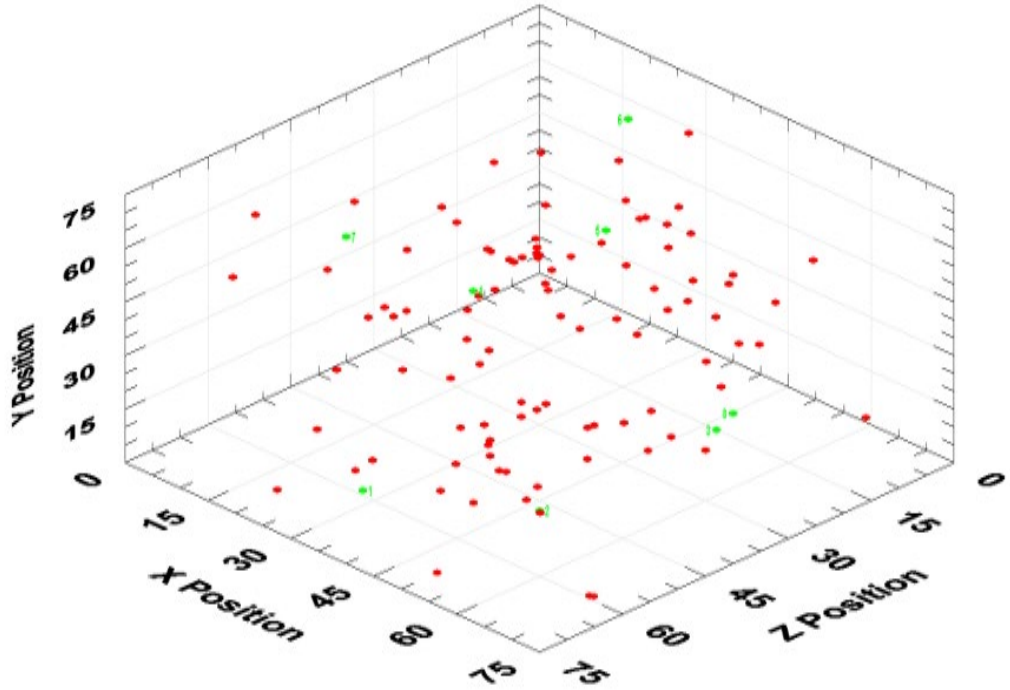


Figure 9 Acoustic emission response to loading for a mortar sample

The fracturing of the block is by extension failure; an example of a mortar sample is shown in Figure 10.

In this case the fracture patterns can be identified in the AE response as shown on the YZ elevation in Figure 11. The close approximation between the laboratory test and the AE response is encouraging, but it is based on a structural interpretation not using a density clustering algorithm as illustrated in Figure 12.

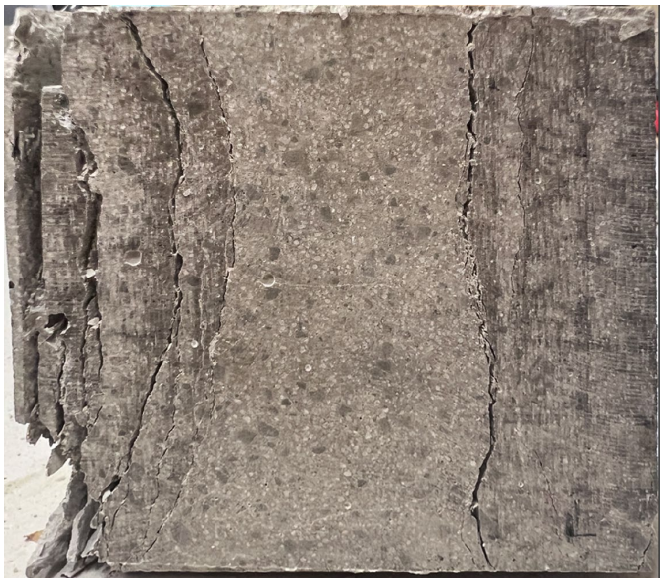


Figure 10 Fracture planes of a mortar sample showing near the free surface vertical fracturing while towards the centre there is often a conjugate set of fractures dipping at ~70°

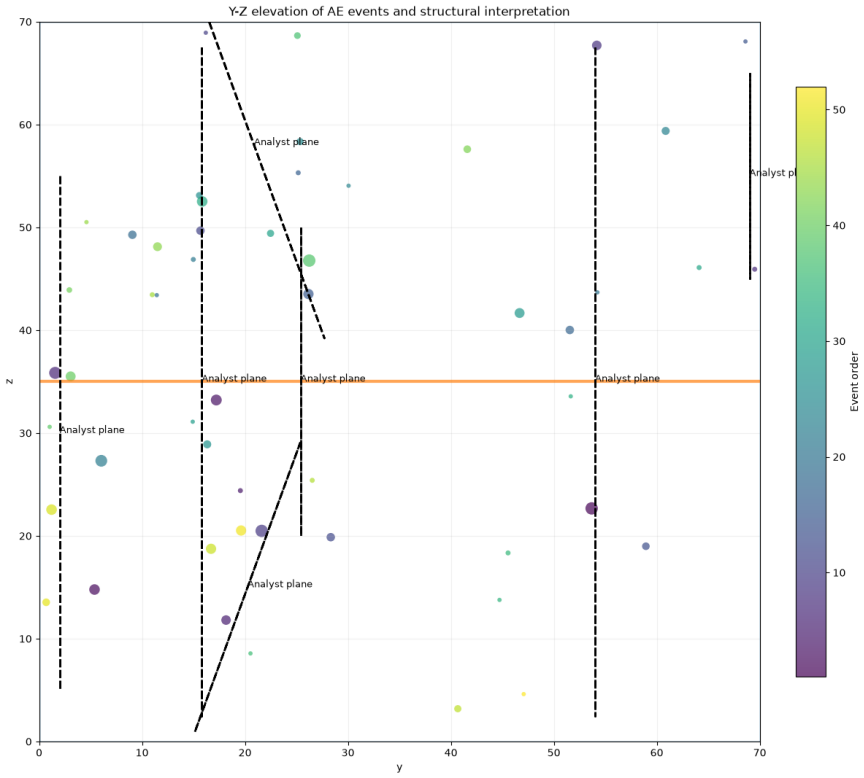


Figure 11 Interpreted fracture pattern from the acoustic emission (AE) response in mortar

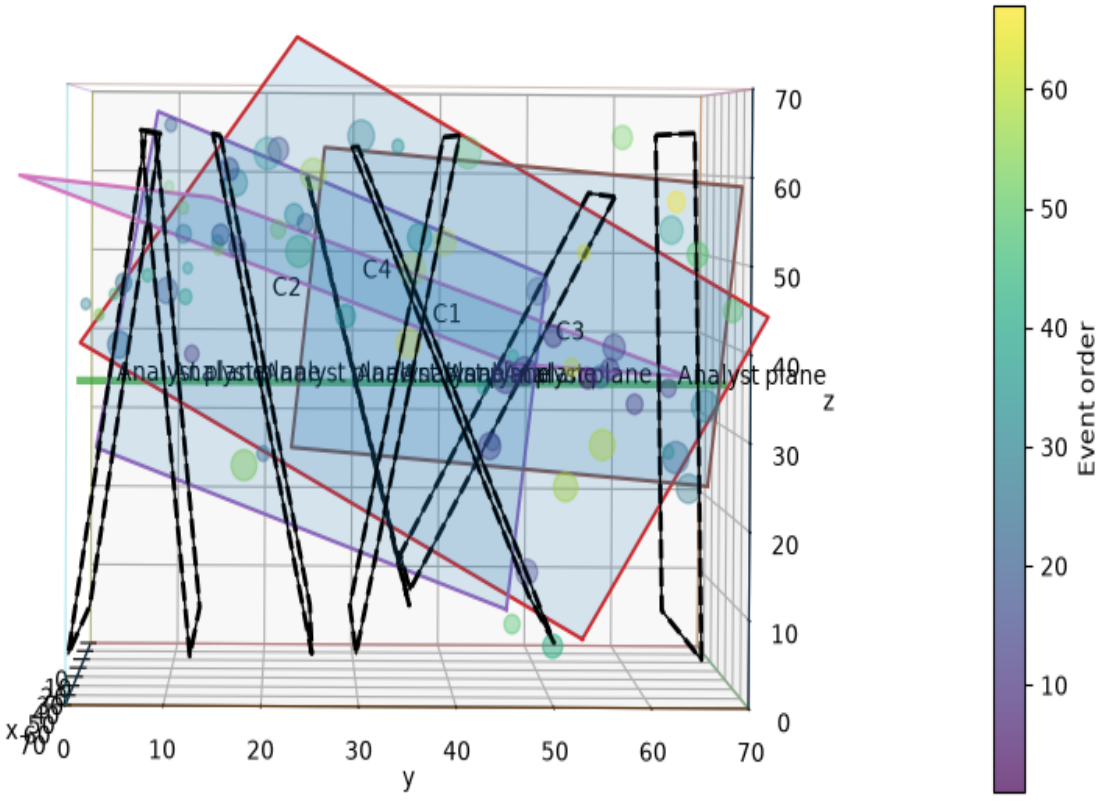


Figure 12 Acoustic emission results clustered based on a density algorithm. Conventional density-based clustering did not recover the fracture geometry observed in these laboratory specimens, whereas a structurally informed interpretation did

As recognised by Potvin & Cumming-Potvin (2024), structure needs to be considered when interpreting seismic data.

The response of a test on a highly anisotropic rock such as Urquhart Shale is shown in Figure 13. In this YZ elevation the position of the bolt is shown. The fracture mechanics is similar to the mortar sample (isotropic) which is welcome. What was of interest was the minimum Brazilian test result was normal to the minimum fracture toughness results. As such the expected depth of failure may not be the same as in the isotropic material. This requires further study.

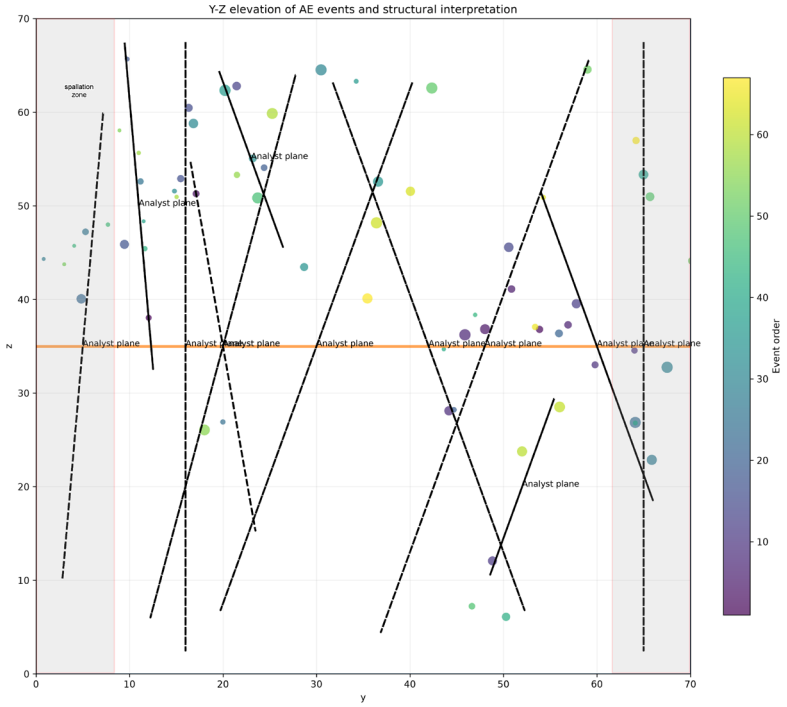


Figure 13 Acoustic emission (AE) response of Urquhart Shale tested with the free surface parallel to the fabric

The structural control on the failure mechanism discussed here is the extensional strain. First introduced by Stacey (1981), it was extended it into 3D (Dight 2022). The benefit is that the minimum extensional strain required for fracture (based on critical crack strain) could be identified using numerical modelling as shown in Figure 14. Recognising the location and applying the appropriate support allows operators to manage the risk of strainburst. It was shown in Dight (2022), that when located in the floor, and no action taken, the floor heaved. As floor support is considered unacceptable the next best action is to undertake precondition blasting.

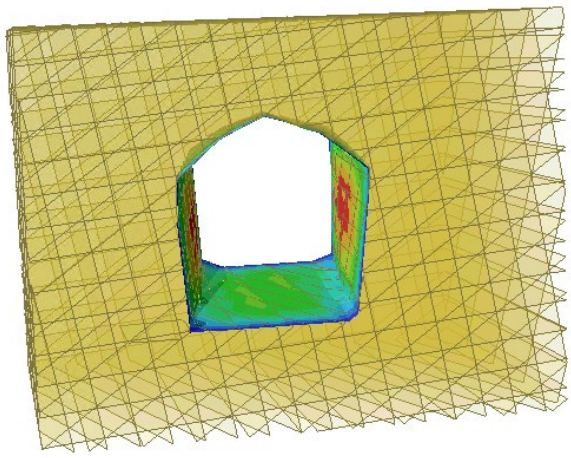


Figure 14 Area of extensional strain shown in red, Rocscience RS3® (Rocscience 2026b)

In 2004, Dight presented a paper on how to recognise the conditions of when to change the support, because a bigger failure could be anticipated (Dight 2004). This was based on the principal structural controls identified controlling the (gold) mineralisation.

The location of these structures should be mapped but can often be interpreted by examining the plan of the oredrives – the geologists follow the ore. Such an example is shown in Figure 15. Today, the approach would be to use a discrete fracture network (DFN) for these major structures which are shown in the stereonet in Figure 16.

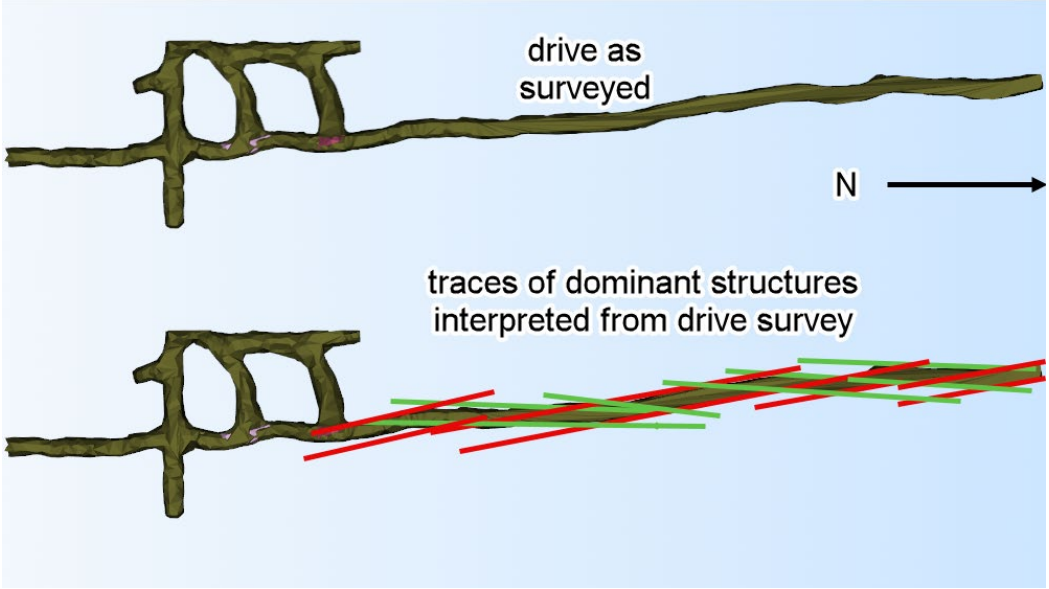


Figure 15 Showing how to use the oredrive to locate major structure with the traces shown in red and green

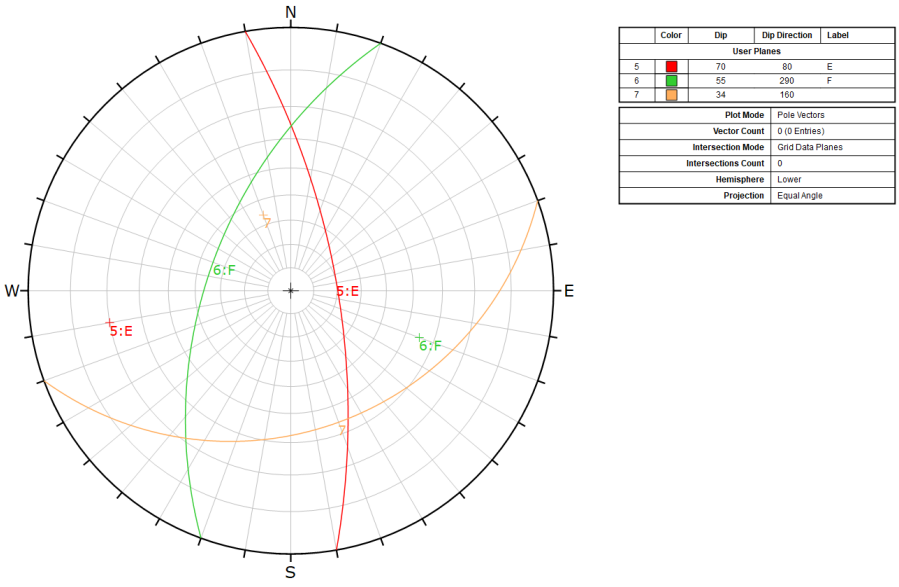


Figure 16 Structural controls for the gold mineralisation for a mine in Western Australia. Set 1 has a spacing of 20 to 40 m, set 2 a spacing of 15 to 50 m and set 3 a spacing of 3 m, Rocscience Dips® (Rocscience 2026c)

The support for the wedges defined by these structures will depend on the stripping planned for the drive. It could be determined by using Rocscience Unwedge® (Rocscience 2026a) as shown in Figure 17.

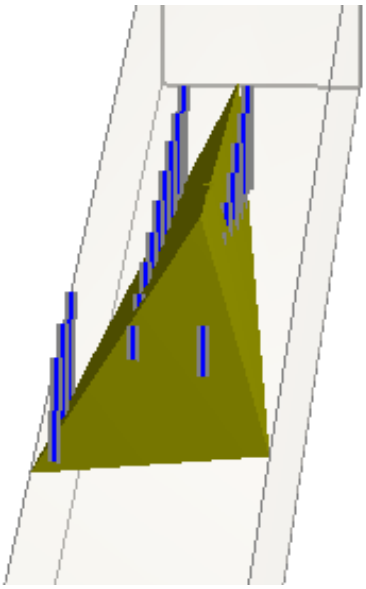


Figure 17 Maximum wedge formed based on the development width

In static circumstances this wedge could be considered stable. That is for the drive. However, if an intersection is created then the possible size of the wedge can be greatly increased changing the assumptions. Based on the spacing of the major structures in the example shown in this case, there is at least one wedge that has an exposed length of >40 m. This is shown in Figure 18 using Rocscience RS3®(Rocscience 2026b) and a DFN comprising the 3 structures. Such modelling is extremely valuable when undertaken with an understanding of the structural regime.

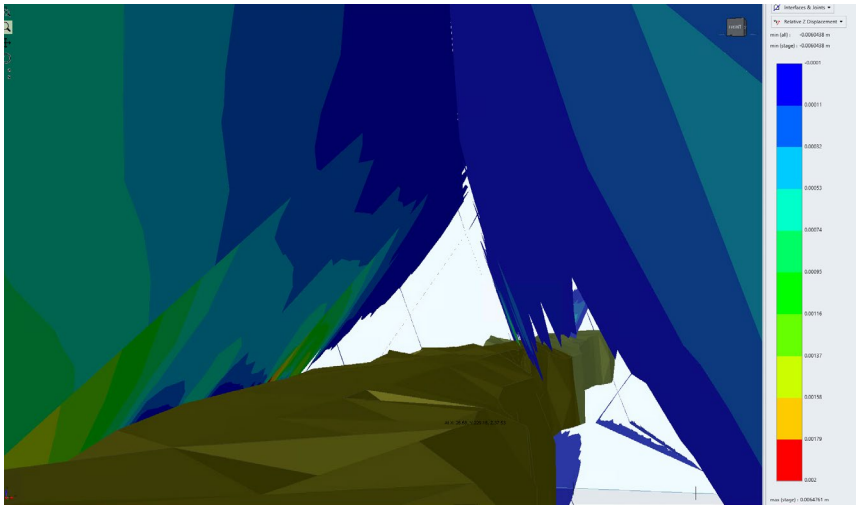


Figure 18 Looking at the shear stress on the surfaces of a 40 m long wedge

The strike length of the wedge is about the size that a remote seismic event such as rockburst could mobilise with a surface wave.

A simple example is used to illustrate the point. Let the Rayleigh wave velocity = 3,000 m/s and the corner frequency (*f)* of the rockburst is assumed to be *f* = 15 Hz. The wavelength (λ) would be 233 m (= 3,500/15). In blasting (e.g. Dowding 1996) it is well known that a block between 1/8th of the wavelength to ¼ of the wavelength can be excited. In this case 1/8 × λ = 29 m and 1/4 × λ = 58 m. There are a lot of conditions to meet here, but using the major structure may be sufficient to identify probable locations for when to change the support for ‘shakedown’. It is worth noting that these ‘lengths’ of potential shakedown are not uncommon.

Using the data produced from the traces and the major structure orientations shown in Figure 16, the true spacings of the shears were determined. For the drive, 3 possible wedges have been determined as summarised in Table 2.

Table 2 Large possible wedges intersecting drive shown in Figure 15

Rank	Chainage	Length	Height	Width	Volume m^3	Probability %	Corner frequency Hz	Wave velocity V_R (m/s)	Wavelength λ (m)	λ/8	λ/4
1	127.3 m	41.8	9.1 m	6.4 m	1,580	4	15	3,500	233	29.2	58.3
2	64.2 m	37.6	8.0 m	5.7 m	1,180	7					
3	181.5 m	34.9	7.3 m	5.3 m	940	11					

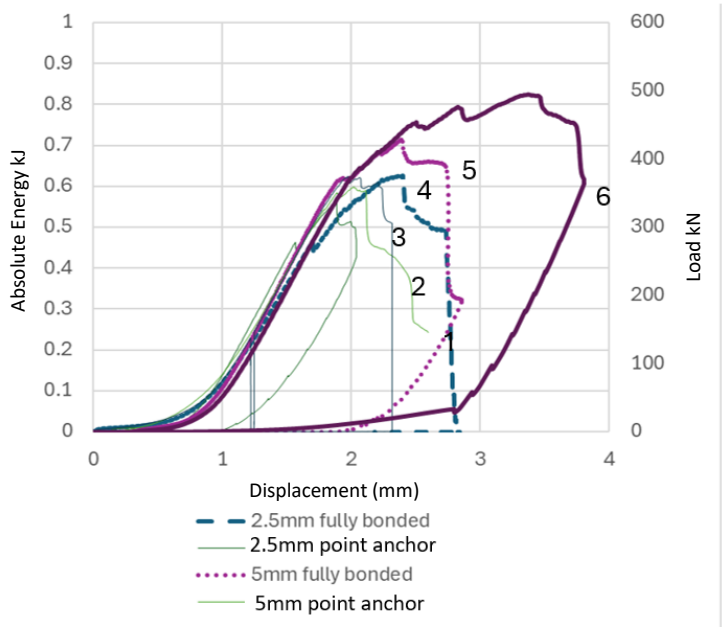
For the conditions considered, all 3 ranked wedges would be susceptible to mobilisation by the design seismic event. Identifying these wedges a priori would lead to modified support installation.

4 Support

In the laboratory, steel bolts with weld mesh are installed in the blocks to simulate the field behaviour.

It would be expected that the bolts would not contribute to the block strength until failure is imminent. The types of bolt installations (point anchor, resin grouted, debonded resin grouted, hybrid bolts [Split Set with point anchor]) and effect of doubling the bolt diameter have been simulated with the results shown in Figure 19.

The increase in strength can be simulated numerically (Dight 2023; Dight et al. 2025) as shown in Figure 20. This is achieved using the principal of superposition.



Increasing shear strength:
1. point anchor/unreinforced *
2. debonded *
3. fully bonded*

*All 2.5 mm diameter

4. split set (roll pin)
5. fully bonded 5 mm diameter
6. 10mm bolt

Note that the performance of the Split set in these graphs would not hold for larger seismic events (i.e. >M_w 2)

Figure 19 Strength displacement plots for the different bolt configurations and the impact of the maximum load the material can take before catastrophic failure. The 10 mm diameter bolt is highlighted as this 4 times the diameter of the point anchor bolt

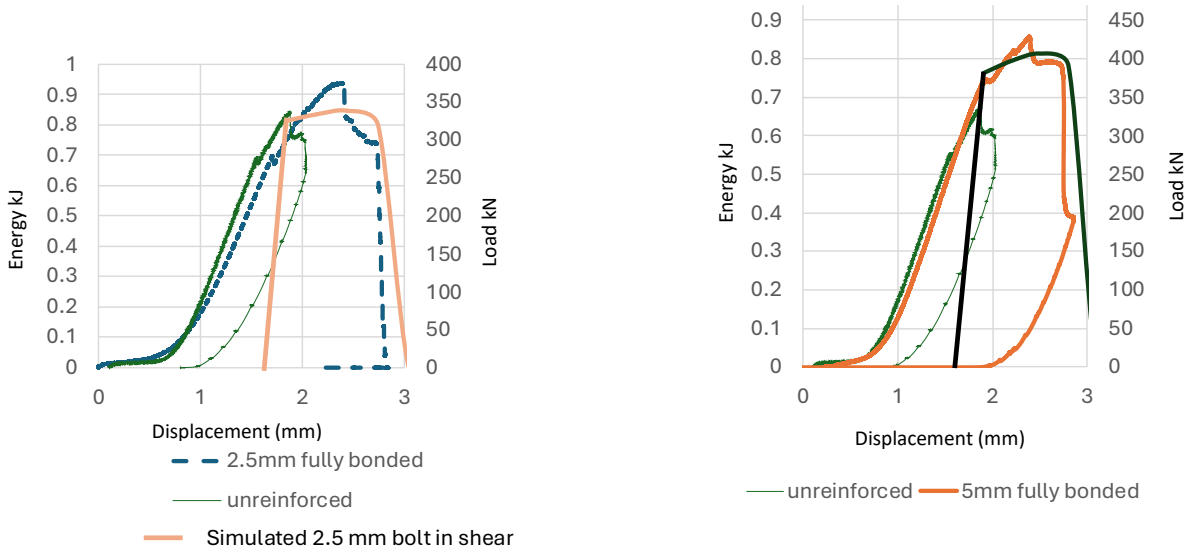


Figure 20 Superposition of the results from analysing the bolt characteristics and then the contribution to block strength

5 Design for strainburst-rockburst

Greenfield/brownfield projects should get stress measurements and rock testing in 3D at the earliest possible stage. Tests should be done in major lithologies and those showing precursors to bursting (breakout, disking). From field observation, this is typically a massive rock issue not a fractured rock mass issue.

- From the stress measurement and 3D properties the compliance matrix and the 3D extensional strain can be determined.
- Use mine plans to examine each stage of mining to ascertain stress changes and proneness to strainburst (3D DXF files).
- Consider seismic events normal to development and parallel to development as 2 extremes (Gibowicz & Kijko 1994).
- Where microseismic activity is being recorded, look at projected event size depicting amplitude and hence damage potential. Corner frequency, wavelength and 3D shear wave velocity need to be assessed.

For events subparallel to development (say from shears [gouge filled]) or abutments of stopes), the extent of damage can be much more extensive because the development can allow for a much longer wavelength, event size, and lower frequency. There is then a need for the shear/Rayleigh wave velocity estimate preferably in 3D as well.

For strainburst, the damage zone appears to be constrained to a depth of ~ 10% of the minimum dimension. This dimension is generally less than the point at which the dynamic bolts are likely to be activated. Could the dynamic bolts be compromised when the rock failure is this shallow? The answer is not yet known.

Observation indicates that full contact bolts (like Split Sets) perform quite well in retaining rock fracture resulting from local seismic damage. Certainly, a weakness is the collar ring of the Split Set, but the support element can absorb rotation. A hybrid bolt comprising a Split Set and a resin anchored bolt should be considered in strainburst prone ground. The resin anchor is considered better than a barrel and wedge anchor as it can be used in more variable rock conditions.

Weld mesh works with these smaller events retaining the bulking which is made up of material that is fractured subparallel to the development. The mesh breaks down when the bolt collars fail and the mesh unravels. This is because the bolts are located on the overlaps of the mesh installation.

For rockbursts, the issue of shakedown needs to be considered. Here the chain-link mesh works better to retain the greater depth of failure, more bulking and still provide restraint with the loss of bolts. The mesh is much less likely to unravel because the mesh connections are well engineered for dynamic loading. Bolt length becomes important so cable bolts would be recommended.

6 Discussion and summary

The field observations are critical to the assessment of mechanisms of rock failure. Table 1 looked at the field observations that would benefit from closer study. A summary of that investigation is then presented in Table 3 where the field observations are correlated to the laboratory observations with an explanation.

The laboratory work should be undertaken to simulate aspects of the field work where the physics can be better controlled and understood. Ignoring the rock mechanics in devising the ground support in the laboratory is counterproductive. Table 3 summarises the 5 aspects covered in this paper showing how the field observations can be linked through good laboratory simulation. This can then be backed up with the appropriate physics.

The paper presents ideas that need further consideration. Getting out into the industry is the first step.

Laboratory testing of intact/massive material shows that the influence of support is only seen as the sample approaches maximum load (Dight et al. 2025).

Mortar was used to simulate an isotropic material. In this material the crack initiation stress occurred at ~50% of the maximum load accompanied by a burst of AE. The AE activity then diminished until failure was imminent, as shown in Figure 21. This behaviour was not replicated in the Urquhart Shale. The reasons are not yet known.

Table 3 Observations of field and laboratory work

	Field	Laboratory	Theory/explanation
1	Apparent depth of failure in 80% cases ~10% of minimum opening dimension	Function of σ_2 >10% σ_1; minimum fracture toughness and minimum Brazilian tensile strength align in isotropic materials; they tend to be normal to each other in anisotropic materials	Location of minimum fracture toughness.
2	Bolts bend in near surface and do not shear	Same observation in isotropic and anisotropic materials	From Kirsch equations the radial stress σ_3 >=1/8 σ_1. In a 5 m high drive this is >~1.5 m behind the face.
3	Holes offset in near surface	Same observation	Could be shearing or result of block rotation.
4	Near surface ejection >3 m/s	Same observation	Spallation.
5	Shakedown – unexpected location/s	Structural control and issues with clustering	Reasonable correlation with expected structural controls and simplistic connection with wavelength, corner frequency and wave velocity.

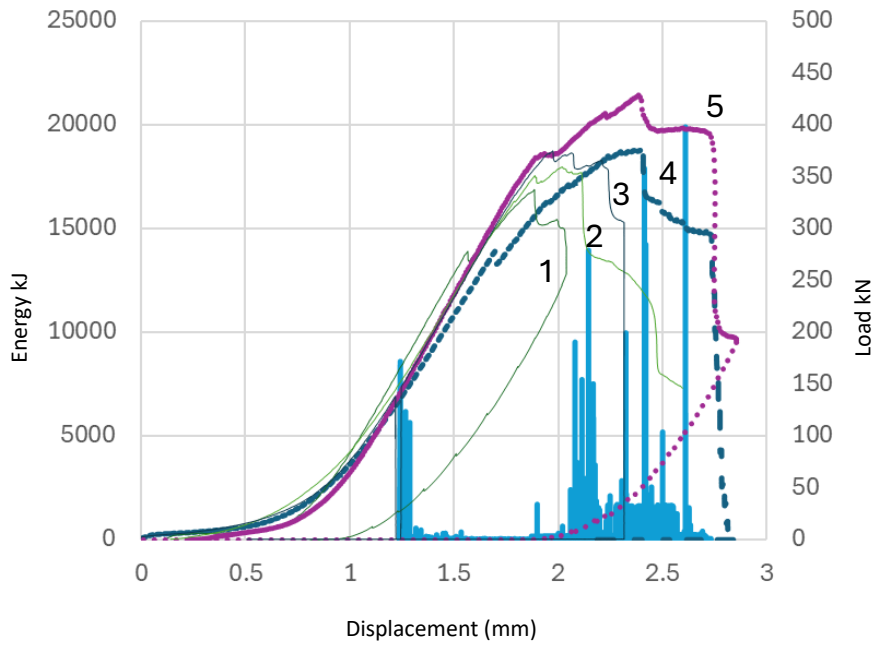


Figure 21 The acoustic emission activity shown during sample loading. At about 50% of the maximum load there is a sudden burst, attributed to the sample cracking through at about a depth of 10% behind the face

1-point anchor, 2–2.5 mm diam debonded resin anchor, 3–2.5 mm diameter fully bonded resin , 4-split set and point anchor, 5–5 mm resin anchor

Tests on Urquhart Shale, which is highly anisotropic, showed minimal AE until very close to maximum load. The influence of the bolts on increasing the maximum load was similar to the isotropic case. The difference was that the post yield strain was minimal.

In all cases the weld mesh defected in a similar way in both the isotropic and anisotropic cases.

The only bolt not to influence the maximum load achieved was the point anchor bolt.

A possible explanation for bolts bending rather than shearing could be the influence of a surface wave such as a Rayleigh wave. A Rayleigh wave differs from a body wave (such as a shear wave) in that it's action involves body rotation not elastic rebound. The scientific literature does not look at surface waves in the underground environment; however, they could be used to explain or understand the occurrence of drive damage remote from a rockburst event with the waves being directed or channelled along drives.

The failures experienced are often called shakedown. The extent of failure is often significant (~30 to 50 m). It is then further conjectured that such damage may coincide with the intersection of conjugate shears, with the extent of failure related to the corner frequency and V_R (~$0.9V_S$). The shear wave velocity can be measured in the field or laboratory and the likely corner frequency obtained from the microseismic record. The expected event size would be obtained from the b-value intersection with event magnitude based also on the microseismic record. An example is shown for the AE sample in Figure 22.

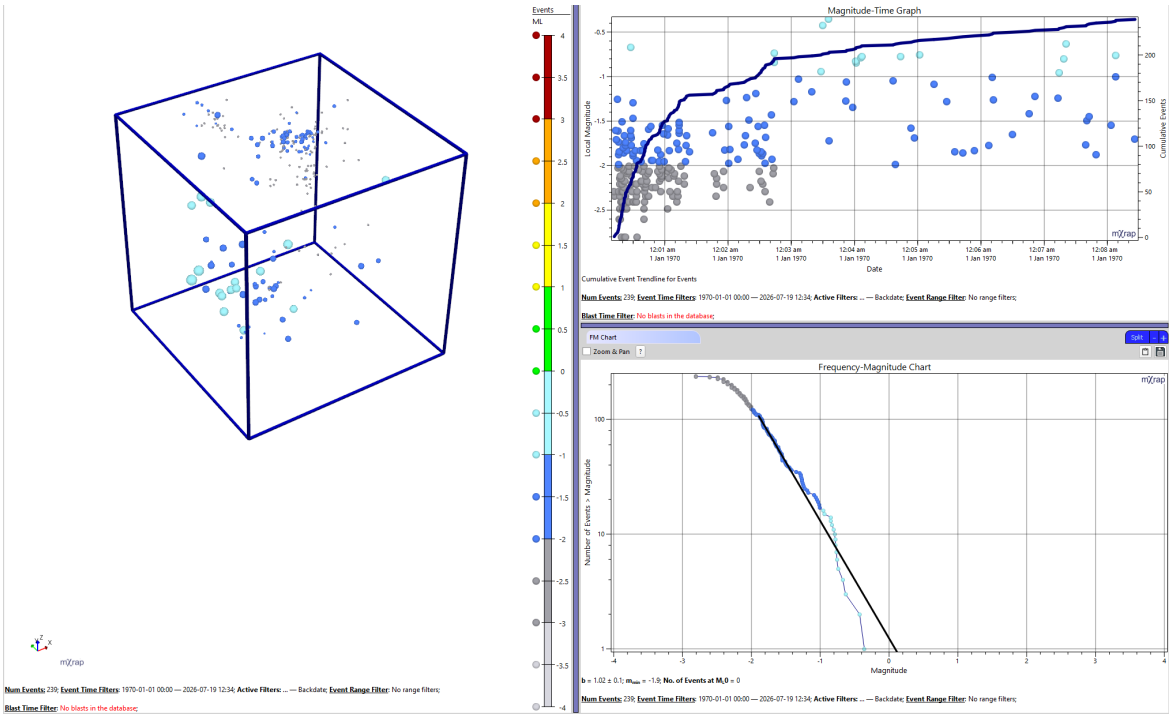


Figure 22 Event location, Moment Time and Moment Magnitude

The length of failure would be in the range of 1/8 the wavelength to ¼ times the wavelength (e.g. Dowding 1996) commonly observed in material response to blasting.

Let's assume that it could be bending or shearing, then the impact on full contact dynamic support could be profound.

The rock fracturing is extension failure in the nearfield biaxial stress regime. This can also be observed in the AE if the data is interpreted with a knowledge of the fracture system. If the same data is interpreted using conventional density-based clustering algorithms alone, the resulting fracture geometry may not represent the structural interpretation observed in the laboratory. The advantage we have is we can disassemble the samples afterwards which is not possible in the field. There is a dearth of literature in laboratory testing that looks at the samples from a structural mode. There are examples in the literature where the clustering interpretation for major seismic events also do not align with the structural model (Dineva & Gospodinov 2022).

It is already observed in the field that grouted bolts are rarely fully encapsulated to the collar. As such this QA/QC issue leads to poor load transfer and may, because of bending, prevent load transfer to the debonded section.

In response to low amplitude events, one support system appears to be ok. That is the humble Split Set. It then stands to reason that a hybrid bolt comprising a Split Set and a point anchor bolt may be the solution. This is observed also in the laboratory. To be fair, in the laboratory we can be confident about the anchorage which is often cited as one of the problems of point anchor bolts with mechanical anchors. This would be overcome with a resin anchor, however.

To manage the ejecting shards, weld mesh was used which was anchored in the middle of the span. However, this is rarely the case in the field where the mesh is typically anchored on the overlaps. Bolt failure (nut, plate pull through or yielding) can lead to the overlaps unravelling.

The process outlined would follow a flow chart as shown in Figure 23. This framework links observations that are normally treated independently. Rather than designing support solely from seismic demand, the approach begins with understanding how the rock fractures, how those fractures evolve, how they are expressed in AE and microseismic data, and finally how support should respond. Each component is individually established; the novelty lies in integrating them into a single design philosophy.

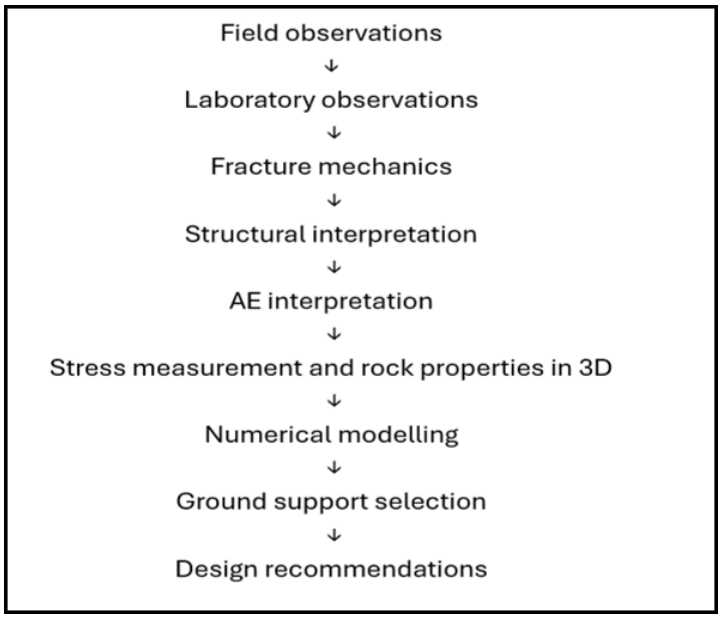


Figure 23 Flow chart for design of support

7 Conclusions

The principal conclusions of this study are:

- Dynamic support design should begin with understanding the rock failure mechanism rather than the support system.
- The shallow damage zone has a mechanical explanation.
- Laboratory and field observations consistently show bolt bending preceding shear failure.
- Examination of the structural geology may be useful in guiding the interpretation of shakedown.
- AE should be interpreted using fracture mechanics rather than relying solely on clustering algorithms.
- Support selection depends on the failure mechanism, not simply on event magnitude.

Acknowledgement

The work described here is the result of many hours underground and in the laboratory talking and working with experts. I wish to identify them: all the staff at the Australian Centre for Geomechanics – Yves Potvin, Ai Li, Sheehan Gabriel, Stuart Tierney, Richard Battison, Paul Harris, Ariel Hsieh, Broadus Jeffcoat-Sacco and Hongyu Wang. At The University of Western Australia, Arcady Dyskin and Elena Pasternak and the workshop, and in the field Brad Simser, Paul Brenchley, Peter Mikula, Roland Bucher, Simon Bruce and William Power. Lastly to the personnel at Instron, Physical Acoustics and Rocscience who have made it possible.

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