

Mining of orebodies under shear loading Part 2 – failure modes and mechanisms

F. T. Suorineni^{*1†}, J. J. Mgumbwa^{2‡}, P. K. Kaiser³ and D. Thibodeau^{4§}

Part 1 of the paper defined shear loaded orebodies and showed through case histories that both pillars and excavations are at elevated risks of failure when mining these orebodies. Part 2 of the paper presents new knowledge on the behaviour of pillars and excavations when mining such orebodies. Numerical modelling is used to understand the behaviours of these structures in the orebodies. It is established that pillars in shear suffer confinement loss compared to their equivalents under pure compression. The confinement loss increases with increasing shear loading in pillars with width : height ($W : H \geq 1$). For pillars with $W : H \geq 1$, the Lunder and Pakalnis (1997) empirical pillar design chart should be used with caution. For excavations in eccentrically loaded orebodies, passive and active high stress envelopes are created in the excavation process. The combined effect of the active high stress and tension zones often results in excavation surface sloughing.

Keywords: Orebodies in shear, Numerical modelling, New knowledge, Pillars, Excavations, Confinement loss, Pillar design chart, Active and passive stress envelopes

Introduction

Orebodies subjected to oblique loading by the major far field principal stress (σ_1) are termed orebodies in shear (Suorineni *et al.*, 2011). In Part 1 of the paper, it was shown through case histories that both pillars and excavations are at elevated risks of failure when mining orebodies under shear loading. This loading mechanism has caused catastrophic failures in underground mining as reported in Hedley *et al.* (1984), Whyatt and Varley (2008) and Kvapil *et al.* (1989). Despite these observations little information exists in the literature on why shear loading often leads to catastrophic failures compared to situations in which underground structures such as pillars are loaded in pure compression. Part 2 of the paper is focused on developing fundamental knowledge on the behaviour of hard rock orebodies under shear loading.

Once an explanation is established as to why underground structures subjected to shear loading are often more prone to elevated risks of failure, existing methods

for the design of underground pillars and excavations will be examined for their validity in the design of structures subjected to shear loading. In particular, the empirical pillar design charts commonly used in mining will be examined for their validity in the design of pillars subjected to shear loading. The effect of shear loading on underground excavations such as stopes during mining will also be investigated. While the phenomenon of shear loading may be well established in long-wall mining of coal, it is less recognised and understood in hard rock metalliferous mining. Hence, little attention is paid to this loading mechanism in mine planning and design.

Approach

According to Starfield and Cundall (1988), the purpose of modelling in rock mechanics (a data limited problem field) is to gain understanding of the problem and to generate data where it is lacking at the initial stages of a project. Numerical modelling also assists in the development of improved understanding of the underlying physics of rock mechanics problems (Fairhurst, 1993). Thus, numerical modelling can be used to develop new knowledge to explain why underground structures loaded in shear are often prone to catastrophic failures compared to their equivalents loaded in pure compression in the same geologic environments under identical conditions. This can be achieved through evaluation of various ideas and hypotheses in simple numerical models. Both pillars (Section 'Modelling approach – Pillars') and excavations (Sections 'Modelling approach – excavations (stopes)') in orebodies under shear loading are modelled using two- and three-dimensional numerical modelling codes. The knowledge developed is used

¹MIRARCO – Mining Innovation/Geomechanics Research Centre, Laurentian University, Sudbury, Ont., Canada P3E 2C6

²School of Engineering, Laurentian University, Sudbury, Ont., Canada P3E 2C6

³Centre for Excellence in Mining Innovation, Laurentian University, Sudbury, Ont., Canada P3E 2C6

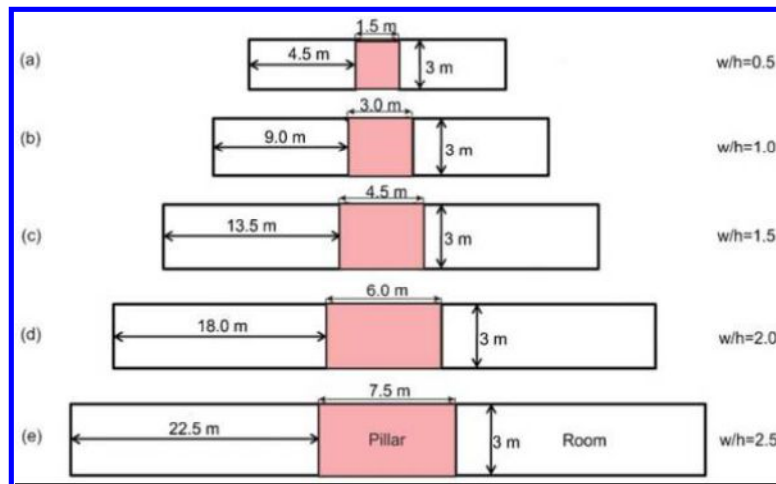
⁴Vale Base Metals Technology Development, Sheridan Park, Sudbury, Ont., Canada

[†]Present address: School of Mining Engineering, The University of New South Wales, Sydney, NSW 2052, Australia.

[‡]Present address: La Mancha Resources Australia, 6A Keegan Street, Kalgoorlie-Boulder, 6432 WA, Australia.

[§]Present address: Stantec, Sudbury, Ontario, Canada.

*Corresponding author, email f.suorineni@unsw.edu.au



1 Layout of pillar modelled with a constant extraction ratio of 75% for an orebody thickness of 3 m

to examine the appropriateness of applying the empirical pillar design chart (Lunder and Pakalnis, 1997) to the design of excavations under shear loading. The stability of excavations such as stopes in orebodies subjected to shear loading is also evaluated and recommendations provided for improved mine planning and design when dealing with orebodies under shear loading.

Numerical modelling

Pillar design and stability assessment are often undertaken by comparing pillar strength to the expected pillar load. Based on a defined magnitude of factor of safety, the ratio of pillar strength to applied load is accepted or refuted. If the resulting factor of safety is refuted, the design parameters are adjusted until an acceptable factor of safety is obtained.

Pillar loads are traditionally estimated from the tributary area method introduced by Bunting (1912). The limitations of the tributary area method and ways of overcoming them are presented in Suorinen (2013). Bieniawski (1981) states that the tributary area approach for calculating average pillar stresses is safe because of its conservative nature and simplicity in practical rock engineering. Hence, it is commonly employed in pillar design. Kaiser *et al.* (2011), however, argue that the tributary area method results in uneconomic pillar design. Both Bieniawski (1981) and Kaiser *et al.* (2011) are right as the required pillar size and safety factor depend on the pillar function. Barrier pillars are designed to be safe for the life of the mine (LOM) while in room-and-pillar type mining, minimum pillar sizes and factors of safety are needed as the latter are temporary pillars and economic extraction ratios must be achieved.

In this paper, both 2D and 3D numerical modelling codes are used to

- Investigate the effects of shear loading on the stability of room-and-pillar type pillars and sill pillars,
- Provide an explanation on the failure mechanisms of pillars and suggest more appropriate pillar design methods,
- Provide an explanation for the effect of the far field principal stress inclination on excavations in the mining of continuous orebodies and
- Examine the validity of using conventional mine planning and design approaches when orebodies are subjected to shear loading.

Modelling approaches

Modelling approach – pillars

In order to understand why pillars in the mining of orebodies under shear loading respond to mining the way they do, it was considered more appropriate to back analyse case histories that were identified in Part 1 of the paper. Phase2 (RocScience, 2010) and Map3D (Wiles, 2010) were used. The brittle Hoek–Brown damage initiation criterion (Martin, 1993) and the bi-linear failure criterion (Diederichs, 2003) are used to investigate the behaviour of pillars under shear loading.

For a geometrically uniform mining layout, the axial pillar stress in a rib pillar is directly determined by the extraction ratio (equation (1)), assuming the tributary area concept. Hence, for valid comparison of stresses in different pillar width to height ($W : H$) ratios one must maintain a constant extraction ratio (E_R)

$$E_R = 1 - \frac{W_p}{W_o + W_p} \quad (1)$$

where W_p = pillar width and W_o = room width.

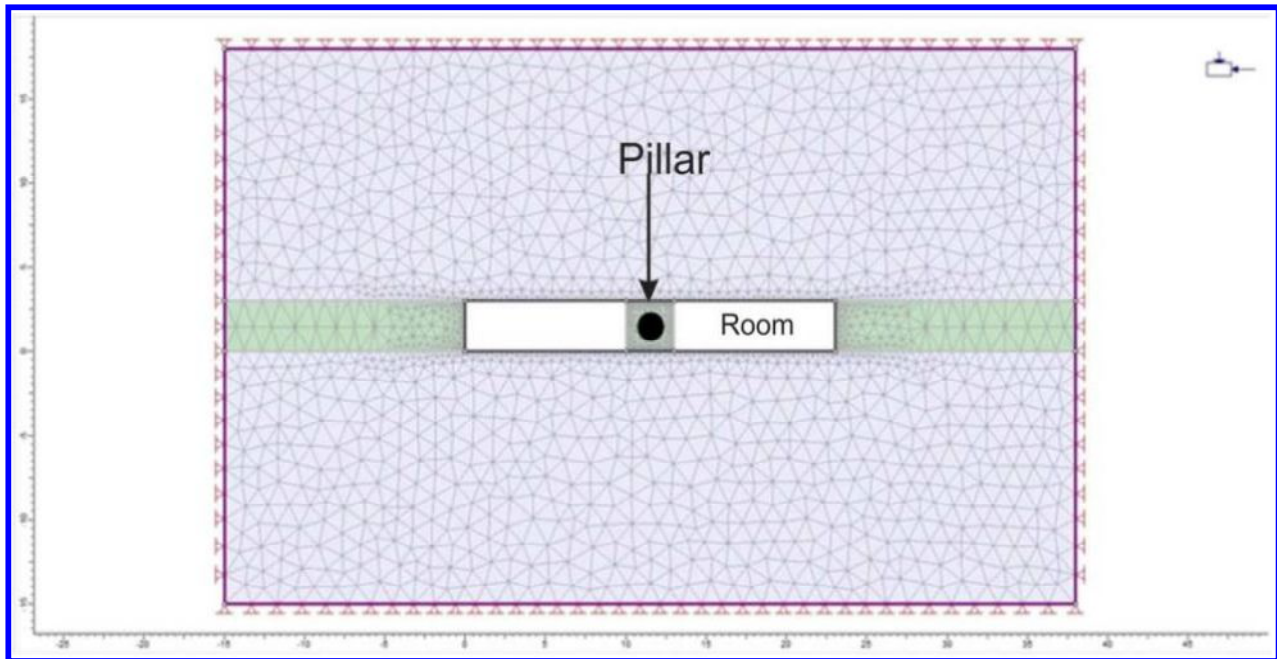
Equation (1) assumes that pillars are square in plan view. Similar expressions can be shown for rectangular pillars. For rib pillars, Hoek and Brown (1980) assume unit length for the pillars. The pillars modelled in this study are rib pillars.

For the cases considered in this study, a pillar height of 3 m (orebody thickness) and an extraction ratio typical of room-and-pillar mining of 75% are assumed. The pillar width to height ratio is varied from 0.5 to 2.5 in increments of 0.5 m (Fig. 1).

Phase2 is used for the modelling. A typical model setup in Phase2 is shown in Fig. 2 and illustrates where the pillar stress is determined.

The accuracy of the results in Phase2 is controlled by the number of discretisation nodes and the expansion factor. The optimum combination of nodes and expansion factor was obtained for 200 nodes and an expansion factor of 9, giving approximately 0% errors.

The model was first run for vertical pillars at the various $W : H$ ratios shown in Fig. 1 for k_o of 1.5 as the base case. This strategy of modelling vertical pillars in pure compression in horizontal orebodies was adopted because abundant knowledge on the behaviour of such pillars exists and will help validate the implemented modelling approach.



2 A typical Phase2 model setup for rib pillars

The intact rock and rock mass parameters in the model are those used by Maybee (1999) as shown in Table 1. In Table 1, the Young's modulus (E) and Poisson's ratio (ν) for the rock mass are given. The brittle Hoek–Brown damage initiation values for s and m for the rock mass are 0.11 and 0, respectively. The brittle Hoek–Brown damage initiation criterion uses one-third of the intact rock uniaxial compressive strength (210 MPa in this case) as the rock mass compressive strength (70 MPa in this case).

The brittle Hoek–Brown failure criterion is known to be the best in describing the performance of pillars and excavations (Martin *et al.*, 1999) in strong, hard, and massive rock masses. For each pillar $W:H$ ratio, the model was run to achieve a strength factor of 1 for a k_o of 1.5 (as in Martin and Maybee, 2000, as base case). The strength factor is the equivalent for factor of safety in Phase2 (McCreath and Diederichs, 1994). The results are superimposed on the Lunder and Pakalnis (1997) empirical pillar design chart as others (Martin and Maybee, 2000; Kaiser *et al.*, 2011) have done (Fig. 3).

The factor of safety curve from the modelling is superimposed in Fig. 3. The curve is obtained for the different pillar $W:H$ values by subjecting them to different stresses while maintaining the same k_o -ratio and rock properties to obtain a factor of safety of unity in each case.

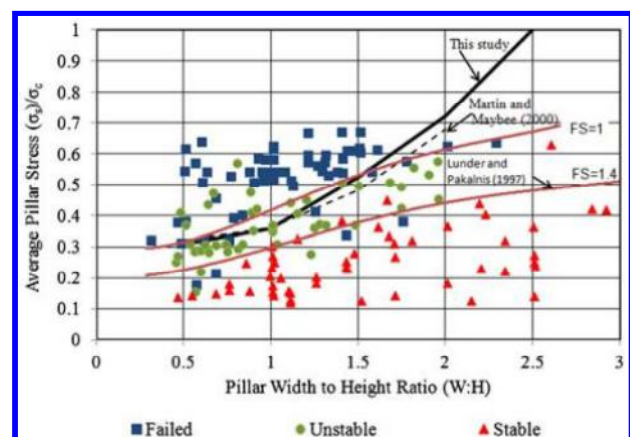
Table 1 Rock properties (Maybee, 1999)

Parameter	Value	Brittle Hoek–Brown parameters
Intact rock strength σ_c	210	210
Geological strength index (GSI)	80	80
Hoek–Brown parameters	m_i	22
	m_b	10.7
	s	0.108
	a	0.5
		0.11
Young's modulus E /GPa	56	0.5
Poisson's ratio ν	0.25	

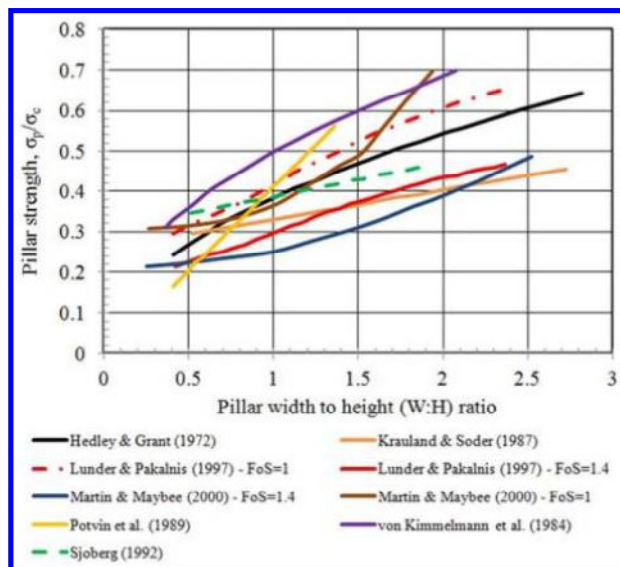
The factor safety increases with increasing pillar width to height ratio and approaches a vertical asymptote after a pillar width to height ratio of between 2.5 and 3. Other authors (Martin and Maybee, 2000; Elmo and Stead, 2010; Kaiser *et al.*, 2011) have found similar behaviour for the factor of safety from numerical modelling. Hence, the procedure adopted for the modelling can be assumed to be validated. The modelling results are presented and discussed in later sections for pillars at inclinations of 0° – 40° at 5° increments for k_o ratios of 1.5 and 2.

In Fig. 3, the curves labelled as factors of safety of 1 and 1.4 were statistically derived (Lunder, 1994) to separate failed and unstable and unstable and stable pillars, respectively. It is therefore doubtful if the degree of conservatism in these curves is known to refer to them as factor of safety curves.

The curves labelled as factor of safety curves of 1 and 1.4 have also been interpreted as strength curves in Kaiser *et al.* (2011), which they compared with a factor



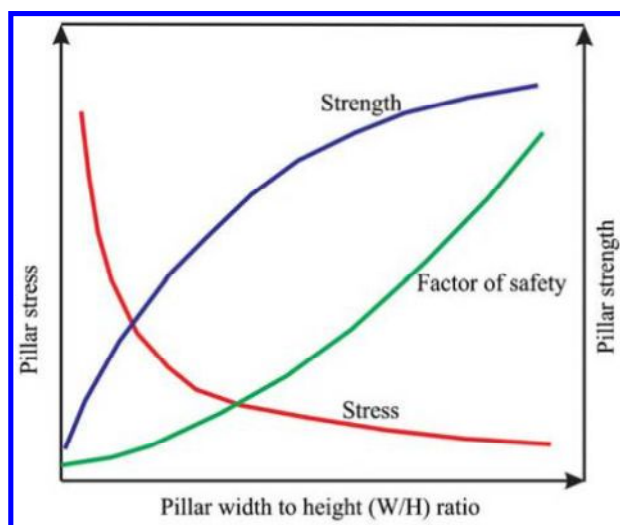
3 Calibration results superimposed on Lunder and Pakalnis (1997) empirical pillar design chart. The calibration model pillars are vertical and subjected to pure compression: $k_o=1.5$



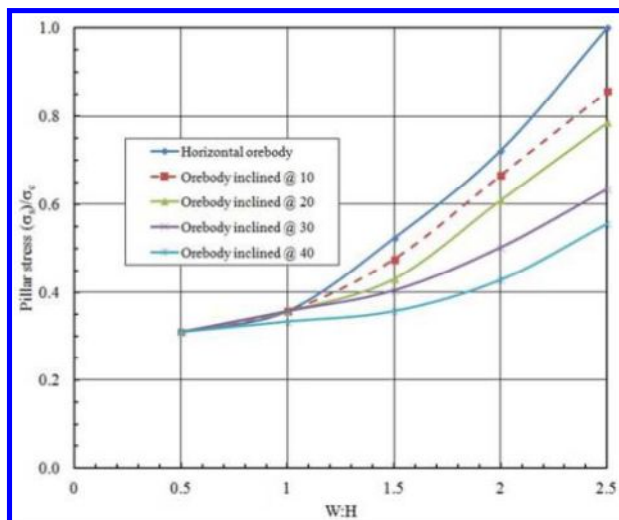
4 Empirical pillar strength curves plotted from the empirical pillar strength equations with the Martin and Maybee (2000) factor of safety curve superimposed

of safety curve developed from numerical modelling. Figure 4 shows a plot of hard rock empirical pillar strength equations. According to Kaiser *et al.*, the pillar strength curves in Fig. 4 and therefore Fig. 3 (they being interpreted as strength curves) should approach a vertical rather than a horizontal asymptote after a pillar $W:H$ of about 2.5. This argument implies that pillar strength curves should behave as the factor of safety curve derived from numerical modelling. However, pillar factor of safety is different from pillar strength.

The suggestion of pillar strengths approaching a vertical asymptote at $W:H$ greater than or equal to 2.5 implies that such pillars have infinite strengths and are indestructible. Because the curves labelled as factor of safety curves in Fig. 3 are not factor of safety curves, they cannot be compared with a factor safety curve. Also, because the curves in Fig. 4 are pillar strength curves, it is inappropriate to compare them with the numerically derived factor of safety curve.



5 Hypothetical relationship for pillar stress, pillar strength and pillar factor of safety curves (Suorineni, 2013)



6 Effect of pillar inclination (increase in shear strength) on pillar stability for $k_0=1.5$

Babcock *et al.* (1981) argue against pillar indestructibility, noting that the assumption of a vertical coal pillar in a flat coal seam being 'infinitely' strong or indestructible is misleading. They contend that such a conclusion is based on laboratory testing of samples between steel platens, which is not the same in the field. Additionally, the idea of pillar indestructibility is based on the assumption of the pillar material being continuous, homogeneous, isotropic and linear isotropic, assumptions that may not be realistic for a pillar rock mass.

To clarify the relationship between pillar stress, pillar strength and pillar factor of safety, Suorineni (2013) provides a hypothetical relationship for these pillar behaviour characteristics as shown in Fig. 5. In this figure, the factor of safety for pillars increases with increasing $W:H$ in a convex manner tending to a vertical asymptote at some $W:H$ ratio. This behaviour corresponds to a decreasing pillar stress and increasing pillar strength as $W:H$ increases. This observation contradicts the behaviour of the curves labelled as factors of safety of 1 and 1.4 in Fig. 3 and supports the findings of Kaiser *et al.* (2011), Elmo and Stead (2010) and Martin and Maybee (2000).

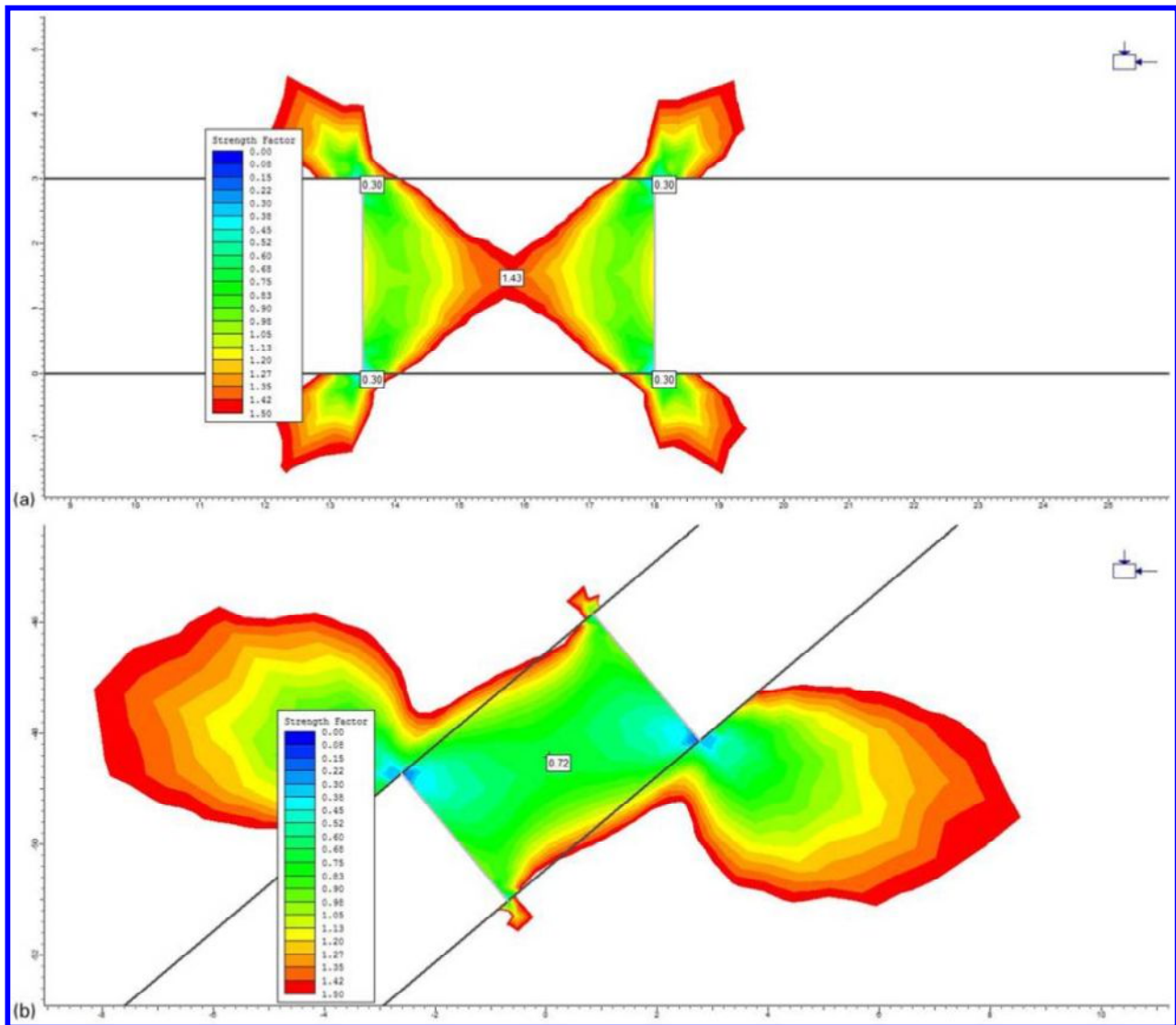
Results for inclined pillars – shear loaded pillars

Typical results for k_0 of 1.5 for pillars at various inclinations are shown in Fig. 6. The figure shows that as the pillar inclination increases its stability is drastically reduced when compared with that of a vertical pillar under pure compression.

The factor of safety contours at the core of a vertical pillar and one inclined at 40° are shown in Fig. 7 for a pillar $W:H$ of 1.5 at 900 m depth. The two pillars have the same rock properties and are subjected to the same far field stress state ($k_0=1.5$). The figure shows that while the vertical pillar is stable (FS=1.43), the 40° inclined pillar failed (FS=0.72).

Comparison of stress distributions in horizontal and inclined pillars

The behaviour of a rock pillar is controlled by its stress distribution. Pillar stress distribution is governed by its shape, size and loading mechanism. Coates (1981) provides pillar shapes that give optimal stress distributions for inclined pillars. Figure 8 shows the stress



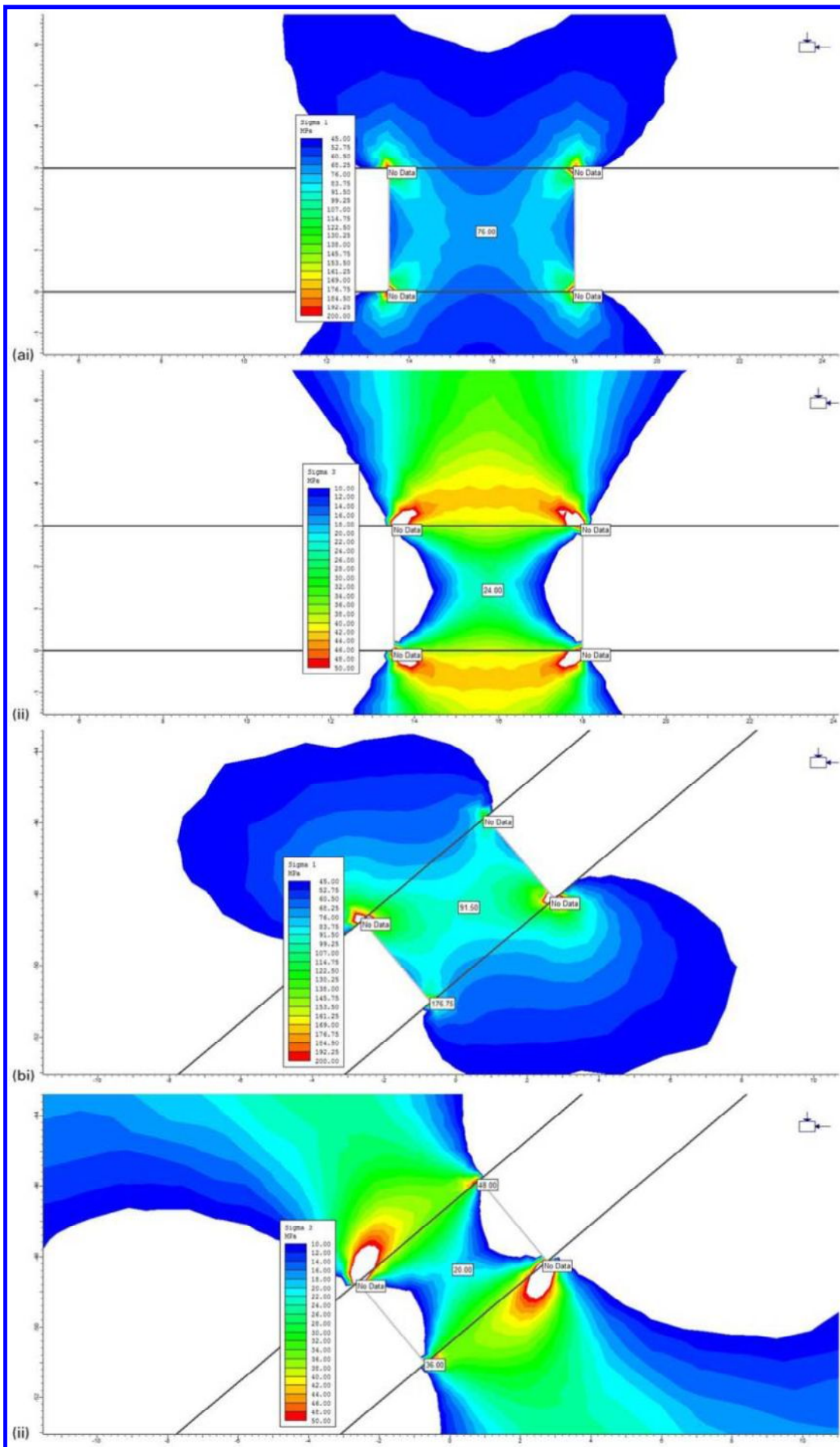
7 Comparison of stability states of pillars loaded in pure compression *a* and shear at 40° inclination *b*: $k_0=1.5$ and $W:H=1.5$

distributions in the pillars in Fig. 7. The two pillars have the same rock properties, $W:H$ ratio and subjected to the same stress state. The pillar loaded in pure compression has symmetrical stress distribution while the inclined pillar is characterised by asymmetrical stress distribution. In Fig. 8a, the maximum induced principal stress at the core of the pillar is 76 MPa and the corresponding confining stress is 24 MPa. In Fig. 8b, where the pillar is inclined at 40°, the pillar core major principal stress is 91.5 MPa with a confining stress of 20 MPa. The inclined pillar core principal stress in Fig. 8b is higher and the confining stress is less than that in the vertical pillar, making it more potentially unstable and prone to brittle failure. While the pillar core confinement difference between the two pillars may just be 4 MPa, this is sufficient to suppress stress-induced fracturing as shown by [Martin et al. \(1997\)](#) and [Read et al. \(1997\)](#). This explains the factor of safety of 1.43 for the vertical pillar and 0.72 for the inclined pillar in Fig. 7.

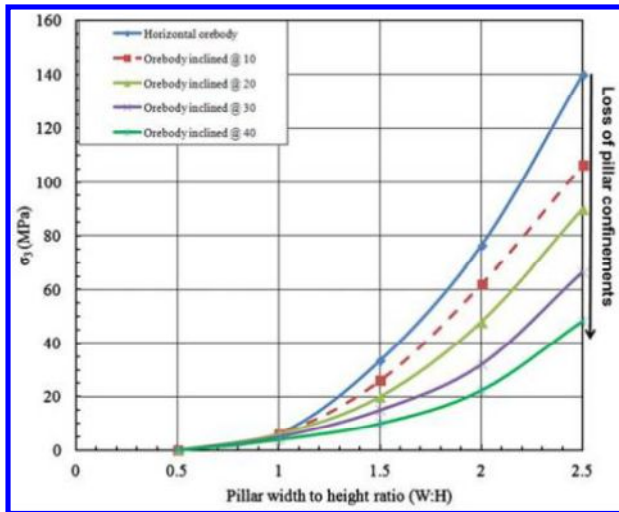
Figure 9 provides a plot of pillar core confining stresses against $W:H$ for pillars at various inclinations and $W:H$ ratios. Figure 9 is developed by subjecting each pillar at a given $W:H$ and inclination to a stress

state (same k_0 -ratio) that results in a factor of safety of unity. The figure shows that as pillar inclination increases it loses confinement with increasing potential brittleness and instability.

Figures 10 and 11 show the maximum induced principal stress distributions across a vertical ($\theta=0^\circ$) and inclined pillar ($\theta=40^\circ$), respectively for $k_0=1.5$ and $W:H$ of 1.5. The stress distributions are determined from lines at mid-heights across the pillar width and two diagonals for each pillar. For the vertical pillar, the maximum stress distributions are symmetrical across the two diagonals and the horizontal line at mid-height. However, for the inclined pillar, the maximum principal stresses are asymmetrical about the two diagonals and the line at the pillar mid-height. Hence, for the vertical pillar, the concept of stress superposition and ignoring the corner stresses to determine average pillar stress ([Hoek and Brown, 1980](#)) is acceptable. On the contrary, because the principal stresses in the inclined pillar are highly skewed for each axis, making the distribution very complex and non-uniform, the pillar corner stresses play a critical role in its performance and cannot be ignored in determining its average stress.



a Pillar loaded in pure compression and b pillar loaded in shear, $k_o=1.5$ and $W:H=1.5$
 8 Comparison of stress distributions (σ_1 and σ_3) for the pillars in Fig. 7

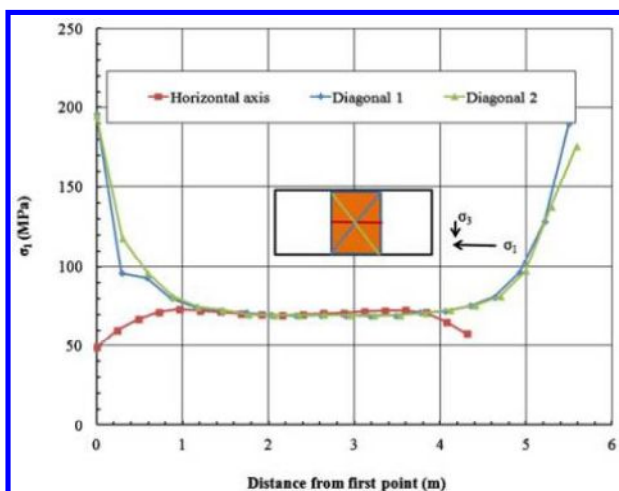


9 Effect of pillar inclination (increasing shear stress) on pillar core confinement: $k_o=1.5$

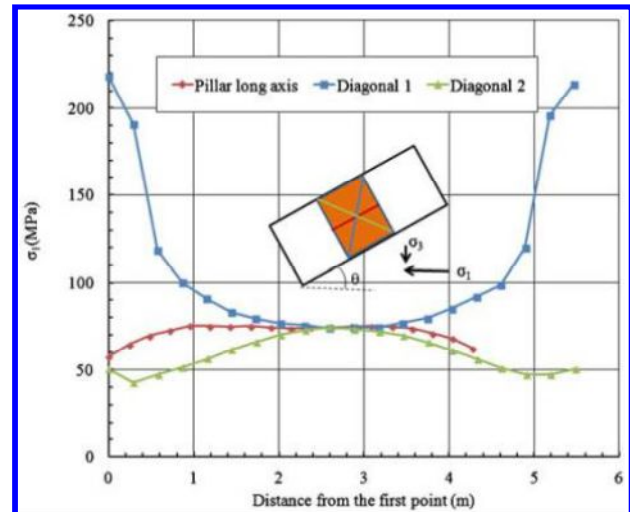
Mode of failure of inclined pillars

The effect of shear loading on pillar modes of failure was investigated by plotting the pillar core confinement (σ_3) to major principal stress (σ_1) ratio (vertical axis) against the $W:H$ ratios for pillar inclinations of 0° , 10° , 20° , 30° , 40° and $k_o=1.5$ (Fig. 12). The figure shows change in pillar failure mode depending on the $\sigma_3:\sigma_1$ ratio and inclination. Figure 12 can be interpreted in terms of the brittle ductile transition phenomenon (Mogi, 1966) that is elegantly demonstrated schematically in Kaiser *et al.* (2000) using the bi-linear failure criterion concept and related to the modes of failure of the pillars. Depending on the pillar core induced stress ratio they may fail in tension (static spalling), shear or ductile deformation as shown in the figure. Thus, the failure modes of pillars depend on the amount of confinement, and as pillar inclination increases, confinement loss increases. Since pillar strength depends on its confinement (Sheorey *et al.*, 1987; Wilson, 1972), loss in confinement results in less stable pillars.

This study shows pillars with $W:H$ ratios greater than 1 are more affected by the adverse effects of shear



10 Variation of the induced major principal stress (σ_1) across the mid-height and along two diagonals for a pillar loaded in pure compression: $k_o=1.5$ and $W:H=1.5$



11 Variation of the induced major principal stress (σ_1) across the mid-height and along two diagonals for a pillar loaded in shear: $k_o=1.5$ and $W:H=1.5$

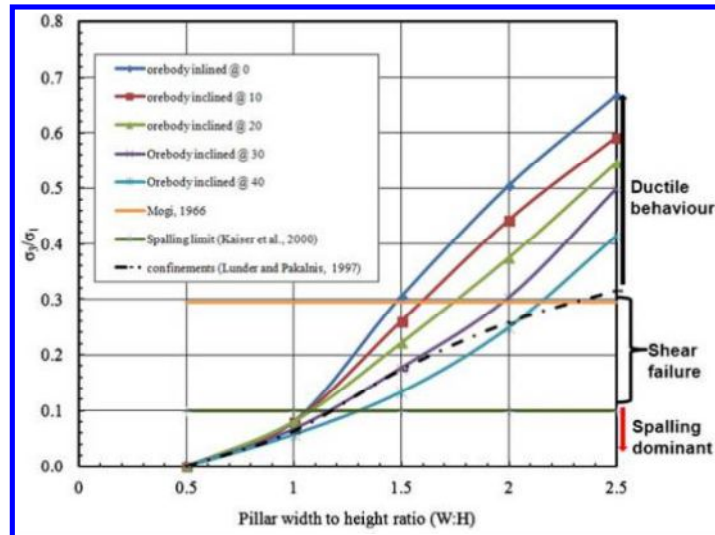
loading and that this effect increases with increasing pillar inclination. It appears that the effect of combined compression and shear loading is a reduction in pillar core confinement. This finding is significant with important implications for mine planning and design. While the role of confinement on pillar stability has been recognised by others (Lunder and Pakalnis, 1997; Sheorey *et al.*, 1987; Wilson, 1972) its critical role in the performance of inclined pillars has rarely been mentioned. Inclined pillars are more sensitive to changes in confining stress, which explains why pillars subjected to combined compression and shear loading often fail more catastrophically compared to their equivalent vertical pillars in the same environments. The consequences of this finding on the use of the Lunder and Pakalnis (1997) empirical pillar stability chart for the design of inclined pillars are discussed later in the paper.

Effect of k_o -ratio on shear loaded pillar stability

The k_o -ratio effect on shear loaded pillars is evaluated by plotting the induced major principal stress in the pillar core normalised by the pillar intact rock uniaxial compressive strength against pillar width to height ratio ($W:H$) for k_o -ratios 1.5 and 2 for pillars inclined at 30° (Fig. 13). The figure shows that there is a significant decrease in pillar stability with increasing k_o ratio. The drop in stability between $k_o=1.5$ and 2 increases as pillar $W:H$ increases. For $W:H$ less than about 0.7, there is no difference in pillar stability state for the two k_o ratios.

Rockbridges in discontinuous orebodies

Orebody offsets may be ore or waste rock and are more common in narrow vein orebodies. These structures occur in various geometries, sizes and composition and can be regarded as pillars. Suorineni *et al.* (2011) provided examples of offsets and showed that they often form the seeds for rockbursts as they are often locations of overstressing. Map3D boundary element code (Wiles, 2010) was used to simulate the behaviour of the various types of offsets. The results show that for the offset sizes, geometries, assumed stress and rock mass conditions used, the offsets are often highly stressed, which explains why they are often sources of rockbursts during mining. It is suggested that where offsets are present during



12 Pillar failure modes determination based on the bilinear failure criterion classification (Kaiser et al., 2000)

mining, the extraction of such offsets should be integrated into the stoping sequence or a distressing strategy formulated to dissipate the stresses at these locations.

Evaluation of the suitability of the empirical pillar design chart for design of inclined pillars

It has been demonstrated in the previous sections of this paper that inclined pillars are subjected to both compressive and shear loading, and that such pillars get increasingly brittle and weaker as the dip increases and with increasing k_o -ratio, compared to vertical pillars under pure compression. The loss in strength of inclined pillars subjected to shear loading is shown to be the result of loss in confinement. Confinement loss increases with increasing pillar dip (i.e. increasing shear). This section evaluates the appropriateness or validity of using Lunder and Pakalnis (1997) hard rock pillar design chart for the design of inclined pillars.

Figure 14 shows the Lunder and Pakalnis (1997) pillar design chart with factor of safety curves of unity for pillar inclinations of 0° and 30° superimposed for $k_o=1.5$ and 2. By comparing the lines of factor of safety for various k_o ratios, it can be seen that as k_o increases, pillars considered unstable and stable in the original pillar design chart fall into the fail zone, particularly for pillars with $W:H$ ratios greater 1. It is concluded from

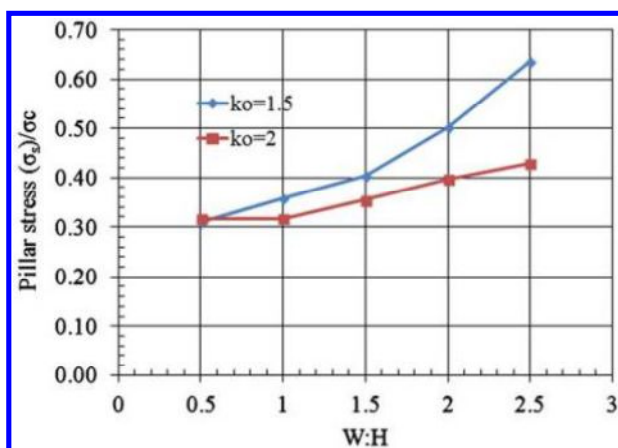
Fig. 14 that the Lunder and Pakalnis (1997) empirical pillar design chart should be applied with caution when using it to design inclined pillars with $W:H$ ratios greater than 1.

Modelling approach – excavations (stopes)

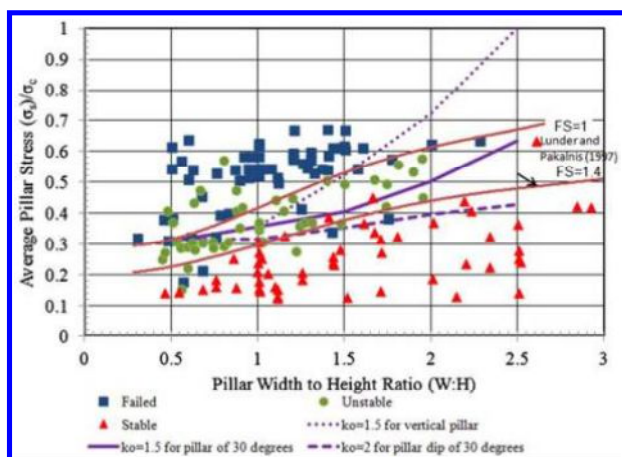
In Part 1 of this paper (Suorineni et al., 2011), it was shown that an inclined stress field at Lac Shortt Mine resulted in either footwall or hanging wall degradation depending on the mining direction relative to the major far field stress inclination to the orebody. This observation was back analysed with Map3D (Wiles, 2010). The approach adopted was to compare where and how the high stress envelope is located and migrated with the mining front for two different mining directions in the orebody.

Behaviour of stopes under inclined loading

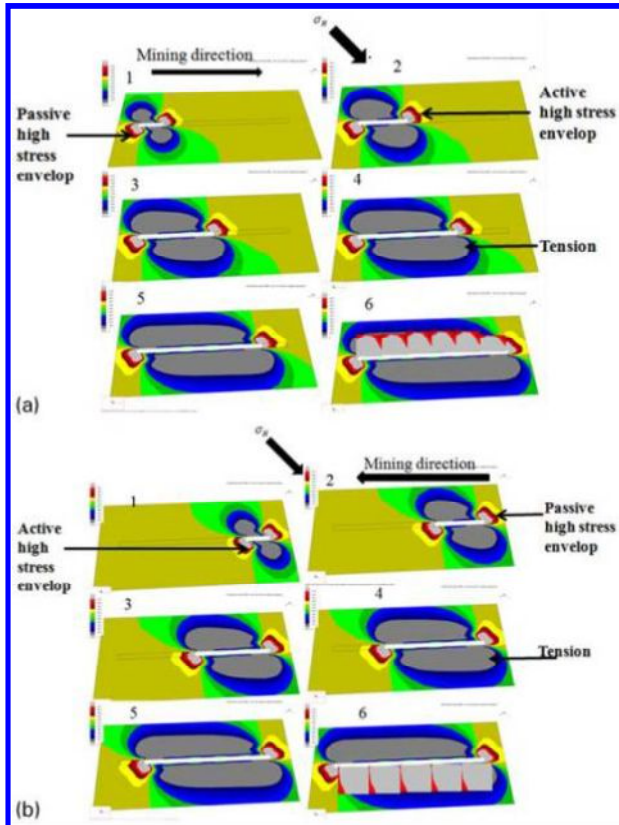
The results of the numerical modelling are shown in Fig. 15 for two mining directions. In Fig. 15a, the mining direction is in the driving direction of the inclined major far field principal stress (west to east). For this case, the induced-stress damage envelopes are at the lower left and upper right hand corners in the footwall and hanging wall of the excavation, respectively. The induced stress damage



13 Effect of k_o -ratio on shear loaded pillar stability



14 The Lunder and Pakalnis (1997) pillar design chart with factor of safety curves of unity for pillar inclinations of 0° and 30° superimposed: $k_o=1.5$ and 2



15 Effect of mining direction relative to major far field principal stress orientation: a along dip with damage location in hanging wall and b against dip with damage location in footwall

envelope at the lower left hand corner is passive (stationary) while the one in the upper right hand corner in the hanging wall is active (moves) and migrates with the mining front (increasing mining step). The migrating high stress envelope in the hanging wall accounts for the high density micro-seismicity in that wall (see Suorineni *et al.*, 2011) because of its damaging effect. Note that the sweeping damaging high-stress envelope in the hanging wall is followed by an envelope of tension. The combination of a high stress induced damaged zone and tension is a recipe for caving (sloughing). This phenomenon, while well known in long-wall coal mining, appears unrecognised in hard rock metalliferous mining. Suorineni *et al.* (2011) showed that caving was a dominant stope failure mode at Lac Shortt Mine.

In Fig. 15b, where mining is from east to west and against the inclination of the major far field principal stress, the location of the damage zone is in the footwall, which is the exact opposite of the damage location when mining from west to east.

For comparison, the major far field principal stress was made perpendicular to the strike of the orebody. In this case, the envelope of high stress ahead of the mining front moves in the orebody. Limited and equal damage is then observed in the hanging wall and footwall. In this case, the result was shown to be the same independent of mining direction. Similar results were obtained when the major principal stress is parallel to the orebody strike except that this time the damaging high stress zone is more out in the hanging wall and footwall with little effect in the mining front in the ore. The latter two modelling results are not included here.

Discussion of modelling results of stopes under inclined loading

The complex geometries of orebodies, the occurrence of orebodies in multiple lenses within a mine, and inaccuracies in the determination of actual *in situ* stress orientations imply that there are more orebodies under shear loading than generally assumed.

The modelling results unambiguously show that the magnitudes of the far field principal stress alone are not sufficient for mine planning and design. The orientation of the major far field principal stress relative to the orebody axis controls the locations of potential damage around the excavation or stope. It is therefore critical to take the major far field stress direction into account in the planning and design of stopes for the safe and economic extraction of orebodies. Understanding the failure mechanisms and modes of failure of orebodies under shear loading will assist in the reliable prediction of dilution and intelligent damage control procedures such as support type and system location. In the case of the Lac Shortt Mine, if the impact of the mining direction on the location of rock mass damage was anticipated, a better handle on dilution or support measures in the footwall developments could have been put in place to manage the mining process.

Conclusions and recommendations

Suorineni *et al.* (2011) presented case histories to show that orebodies subjected to oblique loading by the major far field principal stress are often more prone to catastrophic failures than would normally be expected under known factors that cause rockbursts. It was concluded that the orientation of the major far field principal stress relative to the orebody strike or dip is another factor that causes rockbursts and hence must be considered during mine planning and design.

Part 2 of the paper uses numerical modelling to explain the failure mechanisms and modes in pillars and excavations in shear loaded orebodies. The research developed new knowledge on why orebodies in shear are more prone to catastrophic failures than would normally be expected. The following are the major conclusions in the study:

- Pillars under shear loading lose strength through loss of confinement. The confinement loss increases with the following:
 - Pillar inclination,
 - k_o -ratio, and
 - Pillar width to height ratio.
- The mode of failure in shear loaded pillars is dependent on the ratio of the minimum to maximum induced principal stresses; and depending on the amount of confinement loss, may be
 - tensile splitting for slender pillars,
 - shear failure, or
 - ductile deformation.
- Rockbridges between discontinuous orebodies are often highly stressed and burst-prone. The amount of overstressing depends on the geometry, size, k_o -ratio, and orientation of the major far field principal stress relative to the orebody strike or dip.
- Recent publications of numerical modelling results have compared empirical pillar strength with factors of safety curves, resulting in the conclusion that

empirical pillar strength curves, which are concave and approaching a horizontal asymptote with increasing $W : H$, are wrong. However, pillar strength is different from pillar factor of safety.

- The suggestion of pillar indestructibility after a $W : H$ ratio of about 2.5 is debatable. This interpretation implies that pillars with $W : H$ ratios greater than or equal to about 2.5 have infinite strengths.

The authors wish to make the following comments on pillar indestructibility:

- Pillar factor of safety curves are not the same as pillar strength curves and should not be compared.
- There is no field data to support pillar indestructibility.
- Data in the Lunder and Pakalnis (1997) empirical pillar design chart cannot be used to support pillar indestructibility. The lack of failed pillars after $W : H$ of about 2 is a reflection of the database limit rather than evidence of pillar indestructibility.
- The curves labelled as factors of safety of 1 and 1.4 in the Lunder and Pakalnis (1997) pillar design chart are lines separating failed and unstable pillars and unstable and stable pillars, respectively. The degree of conservatism in these pillars is unknown for them to be assigned factors of safety.
- The empirical pillar design chart by Lunder and Pakalnis (1997) should be used with caution when applied to the design of shear loaded pillars with width to height ratios greater than unity.
- When mining stopes in orebodies under shear loading, the location of stress-induced damage is dependent on the direction of mining relative to the dip of the major far field principal stress.
- The severity of damage is exacerbated by the fact that in the mining process, the moving envelope of damaging high stress in the hanging wall or footwall with the mining front is followed by a zone of relaxation. This is a recipe for sloughing (caving or excessive dilution).
- It is recommended that because of the large uncertainty with stress measurements data, in particular, stress orientation, it will be beneficial to regularly perform sensitivity analysis to account for potential shear loading effects in pillars and excavations in order to take proactive measures to mitigate potential seismicity effects and excessive dilution.
- Mine planning and design should always take into account the orientation of the major far field principal stress relative to orebody strike or dip.

Acknowledgement

The authors are grateful to Vale Ltd and the Natural Sciences and Engineering Research Council of Canada (NSERC) for funding the research. The authors also wish to thank Dr J. G. Henning for providing useful suggestions during the preparation of the manuscript.

Reference

Babcock, C., Morgan, T. and Haramy, K. 1981. Review of pillar design equations including the effects of constraint, Proceedings First Conference on Ground Control in Mining, 23–34, Morgantown, ICGCM.

Bieniawski, Z. T. 1981. Improved design of coal pillars for U.S. mining conditions, Proceedings First Annual Conference on Ground Control in Coal, 13–22, Morgantown, ICGCM.

Bunting, D. 1912. Chamber-pillars in deep anthracite-mines, *Trans. Am. Inst. Min. Eng.*, **42**, 236–245.

Coates, D. F. 1981. Rock mechanics principles, Ottawa, Canadian Government Pub Centre.

Diederichs, M. S. 2003. Rock fracture and collapse under low confinement conditions, *Rock Mech. Rock Eng.*, **36**, (5), 339–381.

Elmo, D. and Stead, D. 2010. An integrated numerical modelling-discrete fracture network approach applied to the characterization of rock mass strength of naturally fracture pillars, *Rock Mech. Rock Eng.*, **43**, 3–19.

Fairhurst, C. 1993. Analysis and design in rock mechanics: the general context, in *Comprehensive rock engineering*, (ed. J. A. Hudson), 1–29, Oxford, Pergamon Press.

Hedley, D. G. F., Roxburgh, J. W. and Muppalaneni, S. N. 1984. A case history of rockbursts at Elliot Lake, Proceedings Second International Conference on Stability in Underground Mining, (ed. C. O. Brawner), 210–234, New York, American Institute of Mining, Metallurgical and Petroleum Engineers Inc.

Hoek, E. and Brown, E. T. 1980. Underground Excavations in Rock, London, Institution of Mining and Metallurgy.

Kaiser, P. K., Diederichs, M. S., Martin, C. D., Sharp, J. and Steiner, W. 2000. Underground works in hard rock tunnelling and mining, Proceedings GeoEng2000, 841–926, Melbourne, ICMS.

Kaiser, P. K., Kim, B., Bewick, R. P. and Valley, B. 2011. Rock mass strength at depth and implications for pillar design, *Min. Technol. (Trans. Inst. Min. Metall. A)*, **120**, (3), 170–179.

Kvapil, R. L., Beaza, J. R. and Flores, G. 1989. Block caving at El Teniente mine, Chile, *Inst. Min. Metall. (Trans. Inst. Min. Metall. A)*, **98**, (6), A43–A56.

Lunder, P. J. 1994. Hard rock pillar strength estimation: an applied empirical approach, M.A.Sc. thesis (unpublished), Vancouver, University of British Columbia.

Lunder, P. J. and Pakalnis, R. 1997. Determination of the strength of hard-rock mine pillars, *CIM Bull.*, **90**, (1013), 51–55.

Martin, C. D. and Maybee, W. G. 2000. The strength of hard-rock pillars, *Int. J. Rock Mech. Min. Sci.*, **37**, (8), 1239–1246.

Martin, C. D. 1993. The strength of massive Lac du Bonnet granite around underground openings, PhD thesis (unpublished), Winnipeg, University of Manitoba.

Martin, C. D., Read, R. S. and Martino, J. B. 1997. Observations of brittle failure around a circular test tunnel, *Int. J. Rock Mech. Min. Sci.*, **34**, (7), 1065–1073.

Martin, C. D., Kaiser, P. K. and McCreath, D. R. 1999. Hoek–Brown parameters for predicting the depth of brittle failure around tunnels, *Can. Geotech. J.*, **36**, (1), 136–151.

Maybee, W. G. 1999. Pillar design in hard brittle rocks, Master's thesis (unpublished), Sudbury, Laurentian University.

McCreath, D. R. and Diederichs, M. S. 1994. Assessment of near-field rock mass fracturing around a potential nuclear fuel waste repository in the Canadian shield, *Int. J. Rock Mech. Min. Sci. Geomech. Abstr.*, **31**, (5), 457–470.

Mogi, M. 1966. Pressure dependence of rock strength and transition from brittle fracture to ductile flow, *Bull. Earthq. Res. Inst. Jpn.*, **44**, 215–232.

Read, R. S., Martino, J. B., Dzik, E. J., Oliver, S., Fall, S. and Young, R. P. 1997. Analysis and interpretation of AECLs heated failure tests, Tech. Rept., 06819-Rep-012000-0070 R00, Toronto, Canada.

RocScience. 2010. Phase2, ver. 7.0 finite element analysis for excavation (RocScience), Toronto, RocScience.

Sheorey, P. R., Das, M. N., Barat, D., Prasad, R. K. and Singh, B. 1987. Coal pillar strength estimation from failed stable cases, *Int. J. Rock Mech. Min. Sci. Geomech. Abstr.*, **6**, 347–355.

Starfield, A. M. and Cundall, P. A. 1988. Towards a methodology for rock mechanics modelling, *Int. J. Rock Mech. Min. Sci. Geomech. Abstr.*, **25**, (3), 99–106.

Suorinen, F. T. 2013. Pillar design in underground mining, Proceedings Twenty-third World Mining Congress (CD-ROM), 15, Montreal, CIM.

Suorinen, F. T., Mgumbwa, J. J., Kaiser, P. K. and Thibodeau, D. 2011. Mining of orebodies under shear loading Part 1 – case histories, *Min. Technol. (Trans. Inst. Min. Metall. A)*, **120**, (3), 138–147.

Whyatt, J. and Varley, F. 2008. Catastrophic failures of underground evaporite mines, in Proceedings Twenty-seventh International Conference on Ground Control in Mining, 113–122, Morgantown, ICGCM.

Wiles, T. 2010. Map3d Manual, Victoria, Map3D Pty Ltd.

Wilson, A. H. 1972. An hypothesis concerning pillar stability, *Min Eng.*, **131**, (141), 409–417.