Geohazard Mechanics 1 (2023) 18–31

Contents lists available at ScienceDirect

Geohazard Mechanics

journal homepage: www.keaipublishing.com/en/journals/geohazard-mechanics

Hardrock burst mechanisms and management strategies

Graham Swan [a], Charlie C. Li [b,*]

[a] Private Consultant, Ontario, Canada
[b] Norwegian University of Science and Technology (NTNU), Trondheim, Norway

ARTICLE INFO

Keywords:
Rockbursts
Yielding support
Seismic energy
Managing design uncertainty

ABSTRACT

Rockbursts occur as a direct consequence of underground mining or civil excavation. The general scale of their seismic disturbance and consequences depend upon known factors. However, uncertainty remains as to exactly when and where rockbursts will occur, as well as the effectiveness of ground support measures to fully mitigate their consequences. While the uncertainty in when and where is a dilemma shared with earthquake prediction, that associated with ground support capability is both a design and a management concern. Following a brief review of the known mechanisms that produce rockbursts, the paper explores the sources and scales of energy demands that characterize the risk of their damaging consequences upon underground excavations. We note that some of this risk continues to be associated with uncertainty with respect to rockmass properties and *in situ* stress, particularly in the context of deep mining. A review is presented of all available yielding ground support systems and their necessary design requirements, identifying practical weaknesses and limitations where these are known. The paper concludes with some suggested areas where further study and development could provide the ways and means to reduce the design uncertainty in managing rockbursts.

1. Introduction

Rockbursts, like earthquakes, from the perspective of public awareness and disturbance, are the rare exception to an otherwise relatively stable and equilibrium state of the earth's near-surface rock mass. Unlike earthquakes, rockbursting and its associated seismicity derives from relatively shallow mining activity and the removal of a rock volume to form an underground excavation of some kind. The depth, act and scale of such an excavation itself provides the potential energy which, in the extreme case, can trigger the release of sufficient kinetic or seismic energy to cause serious damage to both underground and surface installations in proximity to the event or events. Descriptions and studies of such events are well represented in the literature as case studies, based upon records dating back to the 1930s [1,2]. Again, unlike earthquakes, the mechanisms which are known to trigger rockbursts typically involve more than fault (i.e. shear) rupture type failures – by virtue of the existence of the noted underground excavations. Adding to the complexity is the dominant role played by the rockmass geology, geological structures – faults, slips, joints – as well as *in situ* (or pre-mining) and mining-induced stresses.

This paper explores the various design challenges that have been or still need to be addressed by practitioners in mining and civil engineering works where the risk of rockbursting can be anticipated: What/Where is the risk and how should it best be managed in any given situation? The authors use various, real-life examples drawn from their years of practical experience, to illustrate their thoughts on this difficult and complex subject. An introduction is also presented on the state of the art of dynamic ground support (DGS) technology and the most commonly used elements in the current DGS systems.

2. Mechanisms and the mechanics of rockbursts

2.1. Knowledge from field observations

As already noted, the factors necessary to trigger and mitigate rockbursts are various and complex and in most cases defy easy, deterministic methods of engineering analyses. With respect to failure *mechanism*, Hedley [1] classed rockbursts to three broad categories, namely: fault-slip, pillar and strain bursts. These categories exist, in the first place, to describe the relative *seismic energy scale* of an event, ordered here in *decreasing* scale and thereby of expected overall damage to excavations: fault-slip, for example, posing the greatest risk of damage to nearby infrastructure, on surface and underground, compared to pillar or strain. However, this would only be true if any rockburst mechanism acted

* Corresponding author.
E-mail address: charlie.c.li@ntnu.no (C.C. Li).

https://doi.org/10.1016/j.ghm.2022.11.001
Received 19 September 2022; Received in revised form 3 November 2022; Accepted 11 November 2022
Available online 19 November 2022

completely independently of the other two.

In practice, the most likely rockburst scenarios involve *two* if not all three mechanisms, depending upon the available potential energy within the system and the near-failure state of each mechanism. As an example, take the deep underground excavation illustrated in Fig. 1 which depicts the reduction in the dimensions of a central, brittle ore pillar excavated inwards from two abutments. The two stopes (or tunnels) advance from the abutments of a brittle, flat-lying, tabular orebody to form a central pillar. At some stage, given deep mining and soft wall rock conditions, the pillar fails – in this case violently, driven by the convergence of the soft floor and roof – producing a pillar burst. As a direct consequence of the pillar failure, a large increment in the convergence of the roof and floor occurs – denoted by the shift in the dashed lines in Fig. 1. The pillar failure leads to the prospect of a shear rupture on existing faults or slips in the immediate back and floor of the excavation – a fault-slip failure mechanism. The failure of the pillar is essentially driven by the high strain in the pillar, whilst the pillar failure leads to the occurrence of secondary fault-slip bursts. Therefore, both strain and fault-slip bursts could occur in a pillar failure event.

As already noted, the distinction between these rockburst mechanisms is in fact more one of geometry and damage *scale* rather than the actual failure process *mechanism* since both involve shear failure. Pillar failures are typically unconfined and so a very high proportion of the released strain energy at failure is converted to seismic energy, giving rise to violent failure. This has been demonstrated in the laboratory, Fig. 2. The slender rock specimen was tested on a very soft, 14 MN capacity loading machine. Prior to final shear failure of the specimen, multiple small-scale, surface strain "burst" failures were observed. A rock pillar burst is similar to the failure of the rock specimen. However, a fault-slip event can occur over a much larger geometric scale and so, while their seismic efficiency, defined as a ratio of the kinetic energy to the released energy and expressed as a percentage, is low compared to a pillar or a strain burst, their overall radiated seismic energy can still be very large [1]. In fact, the largest known mining-induced seismic event was reported by a Norilsk nickel mine with a Richter magnitude of 5.0 [3].

Of the three mechanisms, when they manifest separately and distinctly, the most easily contained by an appropriate selection of ground support is the strain burst, Fig. 3. This is simply because most of the potential energy W available to such events is on a scale that can be absorbed by yielding support systems through the energy relationship. The kinetic energy W_k observed by a microseismic system is a portion of the potential energy W according to Salamon [4] and Hedley [1], that is:

$$W_k = W - (U_c + W_{gs}) \tag{1}$$

where U_c is the strain energy and W_{gs} is the energy absorbed by deforming the ground support.

Here the energy scales are on the order of kJ/m^2 of excavation surface, whereas for the larger pillar and fault-slip mechanisms they can be several orders of the magnitude greater. As a direct consequence of this, larger scale mine design strategies rather than ground support solutions alone are required to mitigate the consequences of these types of rockbursts. In *all* cases the most effective ground support solutions must have *yielding* or deforming characteristics, which when combined with a high yield load serves to provide their energy absorbing property commonly referred to as *toughness*. Some of these strategies will be reviewed and discussed below.

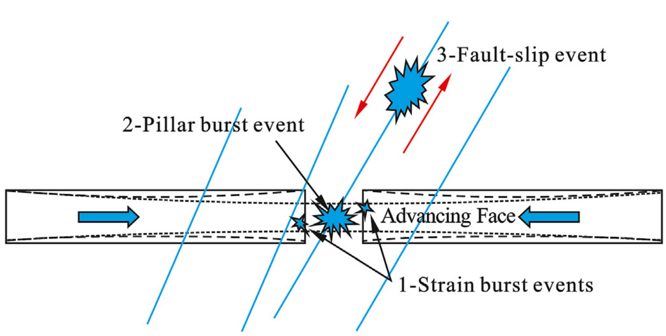


Fig. 1. A sketch illustrating a scenario whereby three related rockburst failure mechanisms may occur as a consequence of an advancing production sequence in an underground mine.

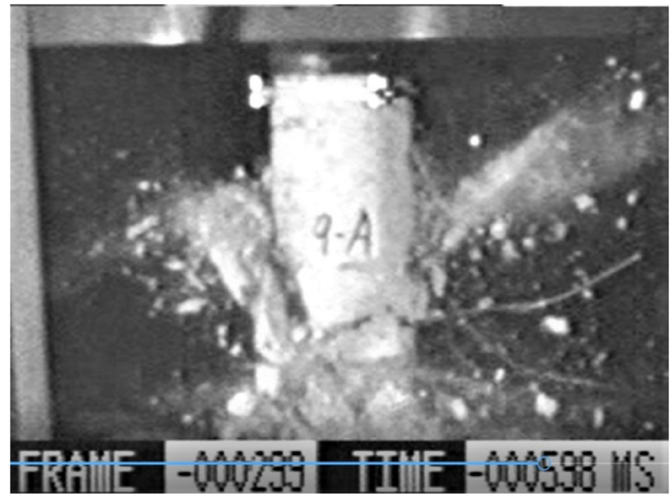


Fig. 2. High speed camera frame obtained in the laboratory on a brittle cylindrical rock specimen, equivalent to a brittle rock pillar, tested in compression.

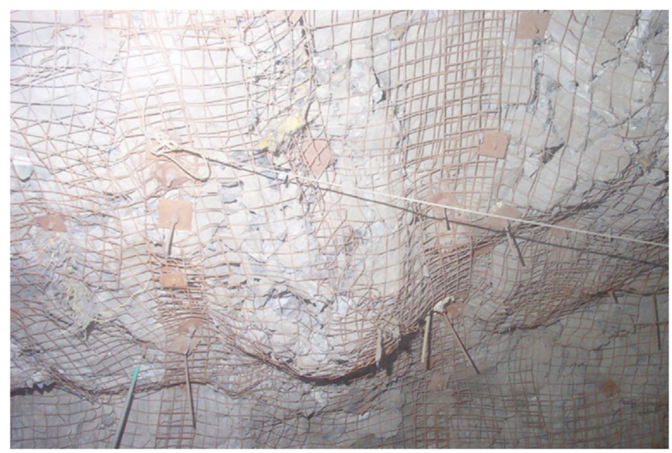


Fig. 3. Example of a strain-type rockburst mechanism, showing surface damage contained by the installed bolts and mesh ground support system in the roof of a mine excavation.

2.2. A concept model of energy conversion in a rockburst event

It is assumed that the bursting rock is completely disintegrated after the burst event and it is not reinforced by any ground support element. The potential energy available in the event, W, is the strain energy stored in the bursting rock prior to bursting, W_b, the strain energy released from the surrounding rockmass in proximity of the burst, W_m, and the seismic energy W_s, that is,

$$W = W_b + W_m + W_s \tag{2}$$

The term W_s is zero in the case of self-initiated strain burst. The above energy components are illustrated in Fig. 4. In the diagram, point A marks the ultimate load of the bursting rock at failure. The thinner solid line

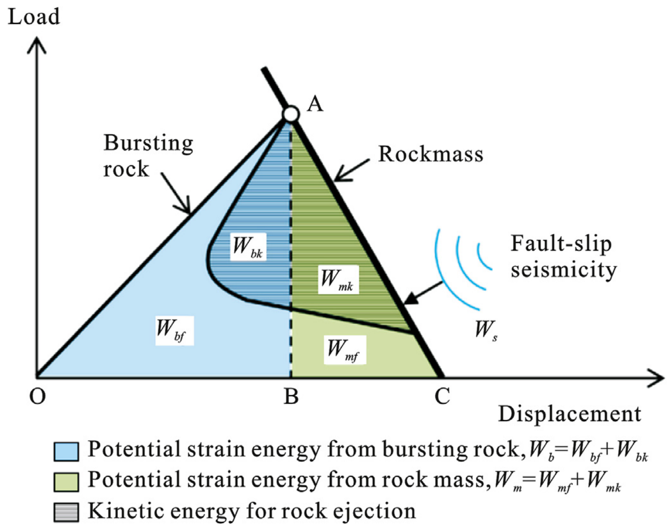


Fig. 4. A conceptual model for the energy conversions in a rockburst event. Modified from Li et al. [5].

represents the load-displacement behavior of the bursting rock (notice that microcracking damage in the pre-peak stage is not taken into account). The thicker declining line to the right represents the response of the surrounding rockmass during bursting. The triangle OAB represents the potential strain energy in the rock to burst, W_b , and the triangle ABC the potential strain energy contributed by the rock mass, W_m . The strain energy is dissipated for rock fracture and rock ejection, that is, rockburst, by neglecting a small portion for heat and vibrations. Therefore, we have $W_b = (W_{bf} + W_{bk})$ and $W_m = (W_{mf} + W_{mk})$ where subscripts *f* and *k* refers to the portions of the energy dissipated by rock fracture and rock ejection, respectively. In addition to the two energy sources above, the seismic waves also contribute energy, denoted as W_s , which is similarly dissipated by rock fracture (W_{sf}) and rock ejection (W_{sk}), that is, $W_s = (\mathrm{W_{sf}} + \mathrm{W_{sk}})$. The total kinetic energy for rock ejection, W_k , is the sum of the three components from the bursting rock, the rockmass and the seismicity, that is, $W_k = (W_{bk} + W_{mk} + W_{sk})$.

In a small-scale self-initiated strain burst, the bursting intensity is only dependent on the kinetic energy converted from the bursting rock, W_{bk} , because the rockmass releases little energy and no fault-slip seismicity is involved. In a large-scale rockburst, the kinetic energy can be from all three sources with a major portion from the rockmass (W_{mk}) or/and the fault-slip seismicity (W_{sk}). In a fault-slip rockburst, the potential energy from the fault-slip seismicity could be enormous, leading to vast damage in underground works. The energy for rock fracture and rock ejection in such an event is dominantly contributed by the fault-slip seismicity (W_s).

3. Applicable data

It is commonly supposed that *in situ* stress measurements and standard laboratory rock strength determinations should be made as a necessary starting point to any study on rockburst potential and the corresponding mitigation opportunities while operating an underground mine or civil work. This approach would certainly be valid if the above three noted rockburst failure mechanisms could be well-described using pure fracture modes (i.e. shear, anti-plane shear, and tensile) and the rock volume was a brittle, perfectly elastic continuum with uniform properties and with no history of past tectonic activity. Clearly none of these is true and in most cases cannot be assumed under any but the rarest of circumstances in the context of *deep* mining. The challenge here is to face the uncertainty by somehow obtaining a statistical description of the *in situ* strength and stress populations and using these in the determination of excavation and ground support design purposes.

3.1. In situ measurement data

Much effort has been expended in the past on the determination of *in situ* stress, using a variety of indirect measurement techniques. Unfortunately, all suffer from the same limitation of being point measurements – in time and space [6,7]. While not always the case, this limitation is most striking in the context of mining, where orebodies have been placed by, and have subsequently experienced, significant tectonic events in the past. Today this history has contributed a spatially complex grouping of different rock types, rock properties, geological structures and stress concentrations such that point measurements only reveal variations of magnitude/sense and only then when taken over the entire population representing the mine. Some appreciation of the uncertainty of point stress measurement data may be seen in Fig. 5, taken from a mine in the "stable" Canadian Precambrian Shield comprising such rocks as volcanics, meta-sediments and granites. What shown in Fig. 5a is the orientation of borehole breakout, which is essentially the orientation of the minor principal stress in the cross-sectional plane to the borehole. The orientation of the major principal stress is perpendicular to the orientation of the breakout. Here an overstress condition is typically initiated at depths in excess of 1 km, but because of the complexity of past tectonics, there is no real certainty of predicting the *local in situ* stress state with depth, magnitude or orientation. Taking NRD12 as an example, the breakout orientation varies in the range from 120° to 160° in the same depth of approximately 1300 m. This is aside from any consideration in the variation of rock strength. A typical phenomenon is corediscing, as shown in Fig. 5b, at the same place where breakout occurs. For a given rock type, the disc thickness decreases with an increase in the in-situ rock stresses. It has been often observed that the thicknesses of disc cores of the same rock type at the same depth vary in a large range, indicating various stress magnitudes.

On the subject of strength measurements, the real issue with standard rock tests concerns the *selection* of diamond drill core taken from a mine exploration program since certain sample requirements exist that serve to bias the results to favour only the *strongest* values that will be obtained, *irrespective* of the number of samples tested. Various attempts have been made to estimate the number of standard strength tests required against confidence intervals [8]. This can be irrelevant when the requirements of the test method itself contribute a sample bias. Perhaps the best way to obtain rock "strength", which after all is never a true *intrinsic* property but test-dependent, is through Point Load Tests (PLTs), Fig. 6, owing to its ease of use and relatively unbiased testing procedure.

Based upon the above remarks concerning *in situ* stress and strength and the commonly observed spread and uncertainty of their respective values, it seems clear that the *location* in a mine where the necessary mechanisms noted above that trigger and drive rockbursts will not likely be obvious, at least in the initial stages of a mine design when data is limited.

3.2. Many data types

In fact, the type and volume of data available for decision-making on a daily basis to the practicing rock engineer in a mine today is enormous, if not intimidating. Most of these data are in 3D spatial as well as temporal coordinates and their sources are as varied as numerical model predictions of stress and deformation, diamond drill core inferred structures and geology mapping, together with discrete deformation measurements, LiDAR scans and global microseismic system event locations and magnitudes. And this does not include the daily observations of unusual events made by underground crews or MWD data from jumbos and bolting equipment, etc. Currently much of this data *flow* is uncoordinated and obtained serendipitously in the sense that it is not fully driven with well-defined, real-time objectives or strategies in mind. In part this is due to the empirically based mean value decision-making ground support and mine design tools that are most commonly used and the characteristic that they all share to smooth-out the details in their

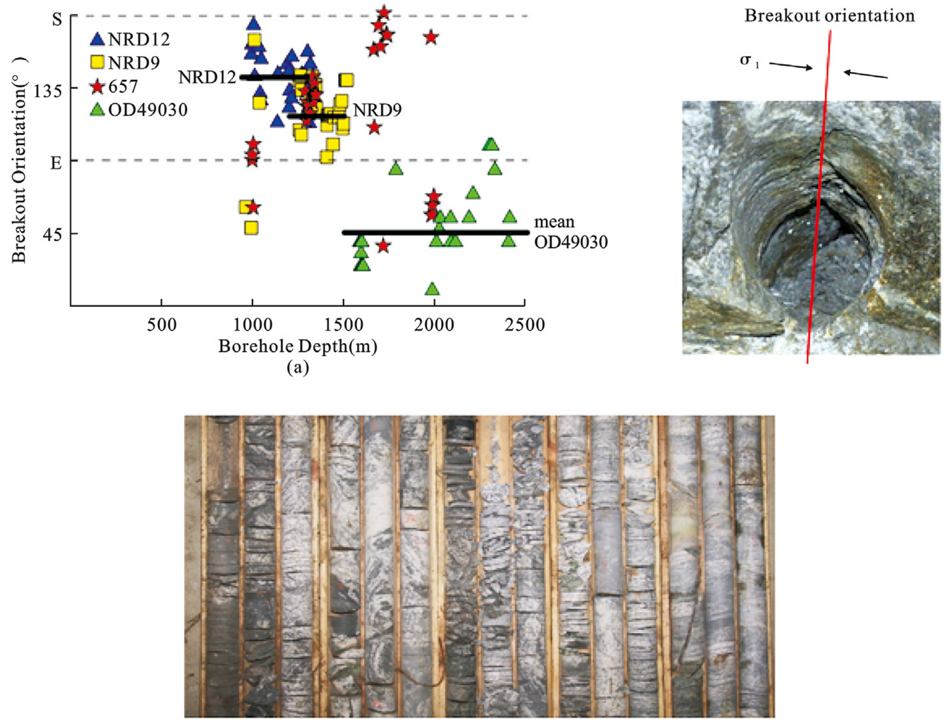


Fig. 5. Clear evidence of a highly variable stress/overstress condition taken at a deep mine and (a) based upon borehole breakout orientations and (b) from core disking.

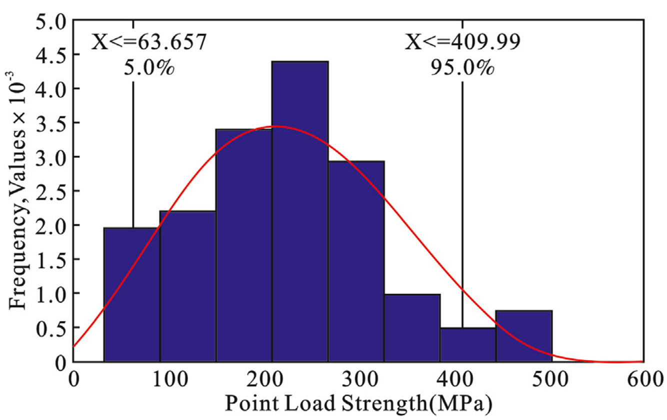


Fig. 6. Example distribution of the point load strength (i.e. the UCS calculated from the Point Load Test Index) over 1000 measurements taken sequentially along a diamond drill core.

various input data sets. However, equally important, it is the realization that the data volume has become overwhelming and therefore impossible to process in a timely manner. All this suggests that we need to find more realistic design tools that use data *distributions* as input, as well as intelligent, machine-driven learning methods of processing the vast amounts of available data to produce new knowledge on a timely basis.

3.3. BIG data and AI

The ground control decision-making task is most challenged when faced with bursting rock conditions, where an assessment of all available data – in the case of an underground mine, seismic, geological/structural, survey, visual, stress, deformation measurements, etc. - must be made in real time and in a timely manner (day-to-day as a minimum). Currently, most of the data is only available in different formats on different digital platforms and may not even be digitized at all. There are also questions about the quality of data in general. Because a mine is continuously changing in the sense that production continuously creates new or enlarges existing voids, in many cases the data is dynamic and must be continually updated to be relevant. Furthermore, even if it is possible to visualize some of these data in 3D space and time, extracting new knowledge from a visual assessment will not of itself necessarily lead to better decision-making.

With the above in mind, and in order to move the existing and real-time ground support decision-making process in a direction that can anticipate improvement benefits, hard evidence of a developing failure condition of support function and of the ground itself needs to be developed and corroborated using all relevant data. This naturally suggests a role for AI (artificial intelligence) and its various decision-making classification, regression and clustering algorithms. The questions that remain are how and what will be the extent and value of the new knowledge for ground support decision-making. Given the growing attention to this subject we should expect some answers in the next few years.

4. On the management of uncertainty

From an engineering perspective we live in a world co-habited, on the one hand with certainty or deterministic solutions, but on the other with uncertainty or probabilistic solutions. Certainty means that the engineer arrives at a unique solution to the problem in question and that this solution is readily acceptable to all, including management. This is commonly the case in civil, near surface works. However, uncertainty results in their being a distribution of solutions, and somebody has to make a decision as to which one is acceptable in any given application. In the case of mining this will be mine management, not the engineering support staff, though the onus will be on the latter to provide management with all the options and their respective probabilities of success/failure. In the case of civil works the decision will rest with the public authority responsible for its construction. Either way there needs to be a very clear understanding at the outset of the respective responsibilities of engineering and management groups when faced with project design uncertainty.

4.1. Monitoring uncertainty

Having established an agreement concerning the various engineering and management roles and assuming that the project becomes operational, a decision needs to be made on the choice of method and extent of monitoring mine design uncertainty. This requires a clear knowledge of which variables are critical to the design's success, but also on which scale – in terms of required measurement resolution and global extent. Here, as noted above, limited campaigns of stress or deformation measurements is not at all appropriate given that the design uncertainty is in fact global, particularly in deep mining applications.

Ideally, a deep mine monitoring campaign begins once the initial access development is completed, and mining begins. In fact, this is commonly the case with microseismic monitoring, the system expanding its global reach as mining proceeds. Unfortunately, these systems do not in fact monitor rock deformation occurring at the surface of development access excavations or, more importantly, the volumetric convergence between footwall and hangingwall of an orebody as the ore is removed. The key ultimately should be to obtain a global measurement of incremental convergence that integrates the many small-scale details of wall rock response to the act of removing the orebody incrementally. Such measurements would provide the best possible data for validating (or otherwise) and updating the initial calibration of large-scale numerical mine models commonly used today for mine design purposes. While there is no easy way of measuring volumetric convergence, that some effort is justified and should be made needs to be recognized. One interesting possibility, available to mines installing paste backfill, might be to install pressure monitoring cells in completed stopes that would integrate the effects of wall deformation as the mining of adjacent stopes proceeds [9]. These would be used to augment point measurements of convergence taken prior to these same stopes being mined.

In the case of access development this should be possible using repeated LiDAR scans relative to a zero datum. Such data should be able to differentiate surface ground support effect in terms of deformations associated directly with bolts (primarily, since liners are of secondary interest) as well as the (indirectly supported) rock between the bolts. Due to the presumed reinforcement effect, as mining proceeds and bolt ring forces increase with rock deformation, such global deformation monitoring should indicate the stage at which a stress arch forms in the back (or not) and the rock abutments begin to carry the superincumbent loads. With further mining it may even be possible to observe the return of back deformations in a situation where the rock abutments begin to fail. Finally, with the onset of rockbursts and dynamic loading events, ongoing surface deformation scans could detect where the sought-after reinforcement effect in the back of excavations has been compromised, meaning the ground support systems are now carrying "dead" loads: suggesting the need for rehabilitation of the entire ground support system.

4.2. Yielding support

4.2.1. Dynamic loading characteristics

As noted above, ground support can be installed at the surface of an excavation to reduce the kinetic energy term W_k. However, a necessary requirement for this to happen is that the support elements' load/stress and deformation/strain characteristics need to match in some way the rockburst energy demands during the rock failure process. Crucially, this also demands that the tensile properties of the supporting elements should be both yielding (as opposed to stiff) and match to such a degree that enables them to act in unison. A survey of data from various sources of conventional and yielding bolts [10] and surface liner elements [11] are compiled in Fig. 7. This data demonstrates, at any given strain, an

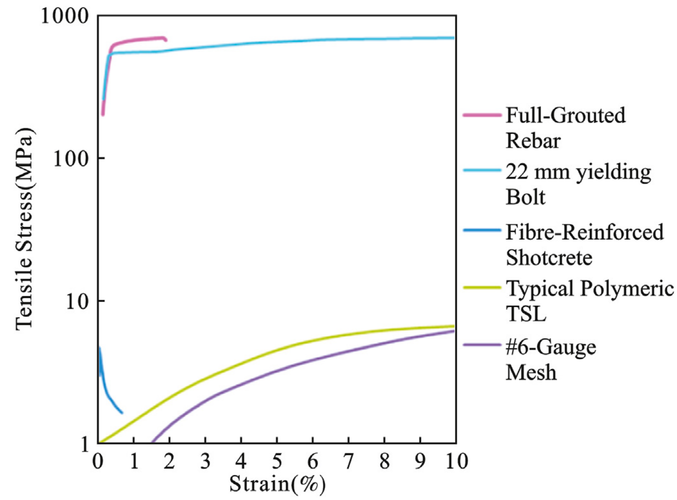


Fig. 7. Tensile test data taken from representative stiff and yielding bolt and liner ground support elements. It can be seen that, at any given strain, the tensile stress mismatch between bolts and liners is several orders of magnitude.

evident mismatch of several orders of magnitude in stress level comparing paired stiff rebar bolts and shotcrete or yielding bolt with a thin spray-on liner (TSL) or mesh.

This obviously means that, compared to yielding bolts, the capacity of yielding liners to reduce W_k is relatively low, which may be demonstrated using a simple energy balance calculation, attributable to Wagner [12] and Roberts & Brummer [13], which assumes a deforming rock bolt, liner, fractured and dilating rock geometry in reaction to a rockburst, the latter characterized by a peak particle velocity of v, shown in Fig. 8. Fig. 8a shows the dynamic failure at the wall of a square tunnel that is supported by a pattern of yielding rock bolts and surface liner. This support system absorbs the kinetic energy of the failed rock ejected at a peak particle velocity v after deforming by an amount c. As the severity of a rockburst increases, represented by v, so must the energy absorbing capacity of the ground support system, which is related to the properties shown in Fig. 7. However, due to the relative mismatch noted above, the relationship shown in Fig. 8b between v and c demonstrates a minimal contribution attributable to yielding surface liners, in this case mesh or TSL. Note that this calculation assumes the liner-bolt load-deformation behavior is uncoupled and that the energy absorbed by the support elements is entirely seismic. While yielding bolts show good capacity to absorb a requisite amount of kinetic energy, all liners have their limitations due to their relatively soft load-deformation characteristics. Interestingly, if a distributed air pressure of 200 kPa is to be applied, the bagging deformation at equilibrium is significantly reduced, by the order of 40%. In practice, support effect due to an elevated air pressure has never been demonstrated in a deep mine, whereas the predicted mismatch between bolt and liner support function has, on many occasions, as shown in Fig. 9.

4.2.2. State of the art of dynamic ground support (DGS)

After a long time of research and applications in the past decades [e.g. 14–19], it has been recognized and acknowledged worldwide that all support elements in a DGS system must be yieldable in order to sustain the impact load of a bursting event. The yielding support elements in the current DGS systems include yielding bolts, meshes (both welded and woven), mesh straps, fiber-reinforced shotcrete and thin spray-on liners (TSLs). Long cables are added in intersections to strengthen the hanging capacity of the support system. More details about yielding rock bolts and TSLs will be introduced in the subsequent sections.

Three types of DGS systems are currently used in deep metal mines. The first type consists of simply yielding bolts (either weak or strong) and wire meshes, Fig. 10. It is accounted as the first generation of the so-

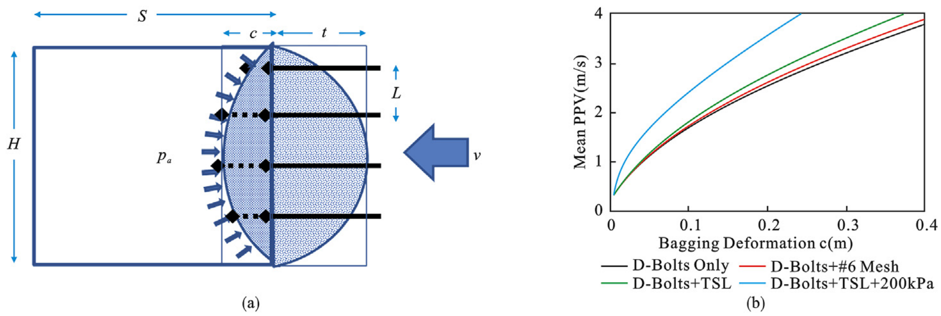


Fig. 8. (a) A sketch depicting dynamic spalling failure (assuming a 2D semi-circular geometry to a depth t) at the wall of a square tunnel that is supported by a pattern of yielding rock bolts and surface liner at an air pressure p_a and (b) derived relationships between the mean peak particle velocity (PPV) v of a seismic event and the bagging deformation c for different yielding supports, after Wagner [12] and Roberts & Brummer [13].

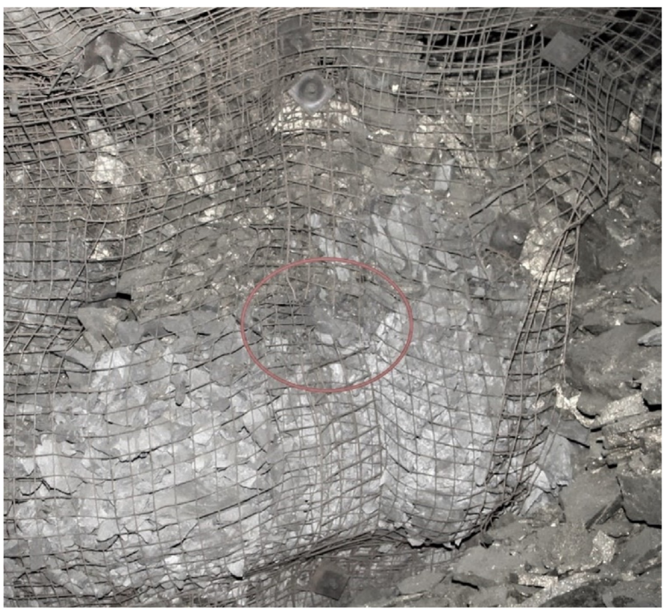


Fig. 9. Illustration showing the mismatch in a yielding bolt and liner system capacity to resist rock failure deformation during a moderate strain burst in a deep mine.

called one-pass DGS systems. Hereby one-pass means that all support elements in the system are subsequently installed in the same installation scheme. Type 1 DGS is nothing different from the conventions static ground support except for the use of yielding rock bolts. It was recognized in practice that this type of GDS is too weak to sustain dynamic loads mainly owing to the low load capacity of the mesh and the weak bolt-mesh link. At presently many deep mines, among others, in Canada, have abandoned it, but it is still used in a good number of mines in Australia. The second type of DGS has been developed to improve the bolt-mesh link and to enhance the bagging capacity of the mesh, by laying mesh straps on the mesh and between the yielding bolts, Fig. 11. In other words, Type 2 DGS consists of yielding bolts, wire mesh and wire straps. The second type of DGS is being used at present in several deep metal mines in Canada. The practice has shown that it performs very well in seismic events up to 3 in magnitude. Fig. 12 is a photograph of the wall of a mine stope that was subjected to seriously fracturing after two subsequent seismic events of 2.9 and 2.2 Mn, respectively, within two days. The tope was supported with Type 2 DGS method. The fractured rock was well contained by the mesh, mesh straps and the strong yielding bolts in the system.

The third type of DGS consists of yielding bolts in walls, long cables in crown, a thick shotcrete layer (up to 100 mm), and mesh on the top of the shotcrete, as illustrated in Fig. 13. This support system has been used in areas of high burst risk in the Kiruna mine in the past ten years.

4.2.3. Energy distributions on bolts and mesh – field test results

The answer to the following is useful in the design of a DGS system: how is the energy dissipated by the DGS system distributed among the support elements in the system? No one could give an accurate answer to the question because the energy distribution would depend on too many uncertainties in the field, for examples, in the rockmass quality, the loading condition, the impact location and the impact magnitude. The rockmass quality may be one of all factors that affect the energy distribution. The results of full-scale drop tests by Roth [21] may provide us some hints about the dependence of the energy distribution on the rockmass quality. In the tests, the samples comprised a thick square concrete slab, rock boulders and gravel on the slab, a sheet of woven

Fig. 10. Examples of Type 1 GDS system that were used in two Canadian metal mines.

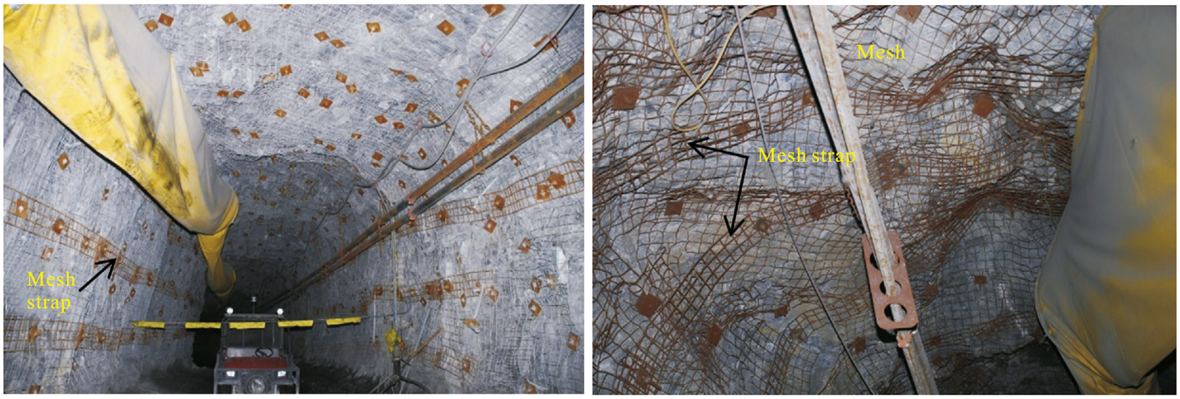


Fig. 11. Examples of Type 2 GDS system in two Canadian metal mines.

Fig. 12. The wall of a mine stope reinforced with Type 2 support method GDS system immediately after the second seismic event of two subsequent events 2.9 and 2.2 Mn, respectively. (Courtesy of D. Counter).

mesh underneath the slab, and four D-Bolts attached to the mesh in four corners of the concrete slab, as shown in Fig. 14a. The four D-Bolts were spaced in 1.2 m × 1.2 m and were grouted in steel tubes suspended on the frame of the test setup. The support system to the test sample is similar to Type 3 described in the previous section, that is, it consists of yielding bolts (D-Bolts), a concrete slab and Geobrugg's chain-link mesh. An impact platform was placed on the rock blocks and gravel. The drop mass hit the platform that transferred the impact load to the test sample. A drop mass of 6280 kg fell from a height of 3.25 m, corresponding to an impact energy of 200 kJ. Two types of samples were tested with identical testing specifications and procedures. The "blocky" sample was constructed with a thick concrete slab and large rock blocks to simulate a relatively competent rock mass, while the "fractured" sample had a thin concrete slab and rock fragments to simulate a broken rock mass. Upon impact, the D-Bolts visibly displaced (Fig. 14b), the concrete slabs fractured to pieces and the mesh deformed (Fig. 14c). The slab was more fractured and the mesh were more deformed in the test of the "fractured" sample than in the "blocky" sample. In the "fractured" sample, the bolts dissipated 10 kJ (~30% of the total energy 36 kJ, dissipated by the support system), and the mesh dissipated 26 kJ (~70%). In the "blocky" sample, the bolts dissipated 32 kJ energy (~75% of the total 40 kJ), and the mesh dissipated 8 kJ (~25%). The rest of the impact energy was dissipated by fragmenting and deforming the rock blocks as well as lost out of the testing system by vibration etc. The test results indicate that yielding bolts may dissipate more energy than mesh in a competent rock mass, while mesh may play a more important role than yielding bolts in a fractured rock mass.

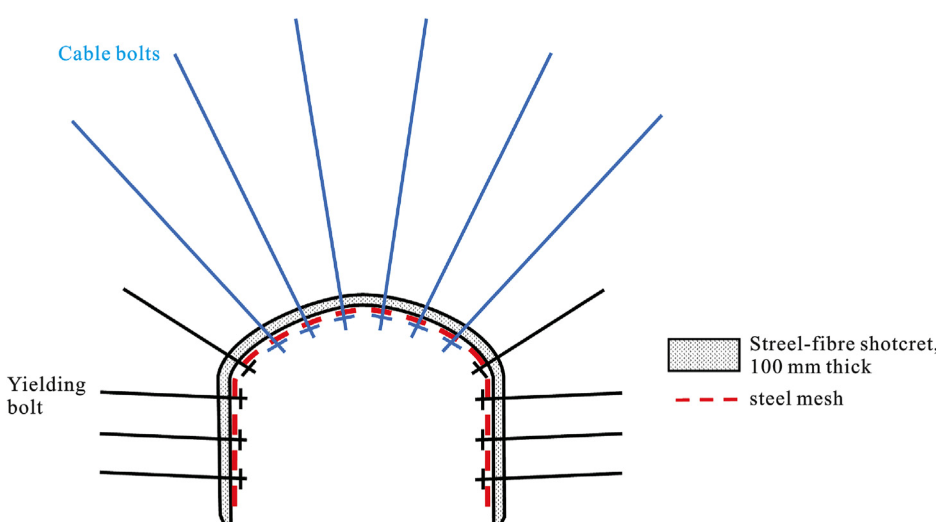


Fig. 13. Sketch of the Type 3 GDS system used in the Kiruna iron ore mine in Sweden. Modified from Malmgren et al. [20].

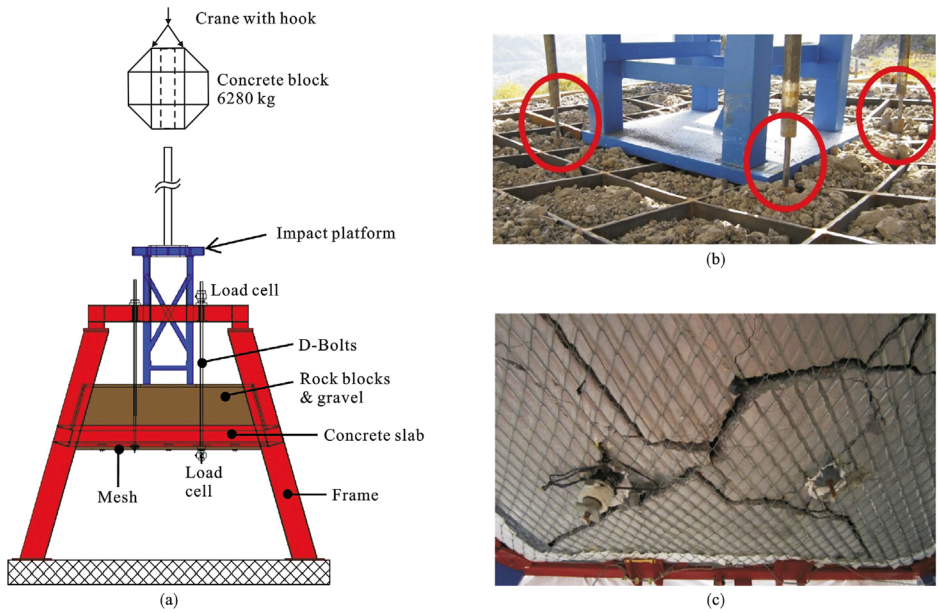


Fig. 14. Full-scale impact tests of "DGS systems": (a) the setup of the impact tests, (b) displacements (circled) of the D-Bolts after tests, and (c) the fracturing of the concrete slab and the deformation of the mesh in the "blocky" sample [21].

4.3. Yielding bolts

The yielding rock bolts that are used in deep mines at present can be classified in two groups, stretch and slide bolts, according to their yielding mechanisms. In addition, cable bolt is also counted as a type of yielding bolt when its deformation capacity is enhanced.

4.3.1. Stretch yielding bolts

Stretch yielding bolts dissipate the deformation energy through plastic elongation of the bolt steel. The D-Bolt is a type of stretch yielding bolt, that is made of a smooth steel bar with several anchors arranged along its length in order to enhance the installation reliability. Fig. 15a shows the configuration of a D-Bolt with two long sections. The number of the long sections can be adjusted according to requirements. The bolt is fully grouted in the borehole. The anchors are firmly fixed in the grout, and the smooth bar sections between anchors elongate upon rock dilation. The bolt dissipates a quite large amount of energy by fully employing the strength and deformation capacity of the bolt's steel [22]. The distribution of the axial load along the bolt length is illustrated in Fig. 15b. The load within a section between two adjacent anchors is induced by the total rock dilation in the stretch the section traverses. Each section works independently so that loss of one section will not affect the reinforcement effect of the rest sections. The D-Bolt has been accepted as the internal rock reinforcement element in one-pass DGS systems in several Canadian and USA deep mines.

Another type of stretch yielding bolt is also made of smooth steel bar but possesses only two anchors, that is, one at the distal end and the other one at the proximity end including the anchor close to the thread section, the plate and the nut. PAR1 [23] is an example of this type of bolt. The two-point anchored yielding bolt is also fully grouted in the borehole. The rock dilation in the stretch of the bolt length loads the bolt shank and the entire bolt long section is identically loaded and gets into yielding at

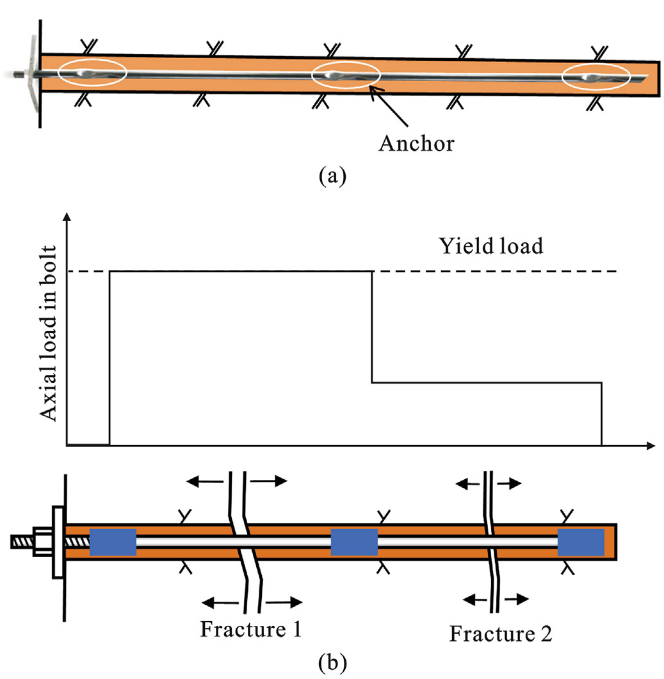


Fig. 15. (a) The configuration of a two-section D-Bolt and (b) the distribution of the axial load along the bolt length.

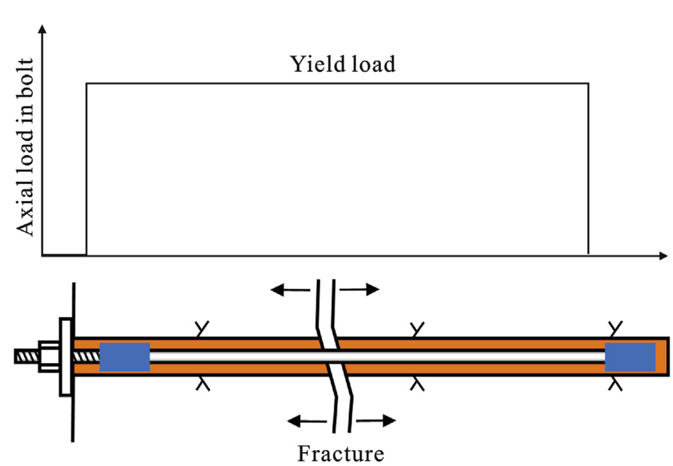


Fig. 16. Distribution of the axial load along the bolt length of a two-point anchored stretch yielding bolt.

the same time, Fig. 16. The bolt could dissipate a good amount of energy as long as the anchors are firmly fixed in the grout. It is a concern whether two-point anchored yielding bolts could be used in one-pass DGS systems because of its relatively low stiffness (owing to the long smooth section length) and the low anchoring reliability (owing to the one long section).

4.3.2. Slide yielding bolts

A slide yielding bolt dissipates the deformation energy through sliding (ploughing or frictional slip) of the anchor, often, at the distal end of the bolt. The so-called cone bolt is an example of this category, Fig. 17a. The cone bolt is a smooth steel bar with conical flaring at the distal end. It is also required that the bolt must be fully grouted in the borehole. Rock dilation results in a load on the bolt plate that transfers the pull load to the cone at the distal end via the smooth bolt shank. The cone plows in the grout when the load is equal to the crushing strength of the grout. The axial load in the bolt is identical along the entire bolt length, Fig. 17. The anchoring reliability of slide bolts is also a concern because of the two-point anchoring mechanism. Slid yielding bolts cannot be accepted as the internal reinforcement elements in one-pass DGS systems.

4.3.3. Cable bolts

Fully grouted plain cable bolting (Fig. 18a) is sometimes used to reinforce the crown of large-scale underground openings, such as drift intersections, where blocks risk to be shaken down in seismic events. The technical reason why it is used as a dynamic support element is that laboratory pull tests have shown that cable bolts can accommodate a very long displacement after yielding without loss load-bearing capacity [e.g. 10,25]. Many think that plain cable bolt is a type yielding DGS element. However, the yielding performance of plain cable bolt is not always mobilized in the reality but conditionally. The cable bolt yields through slip of the strand in the grout. All cable bolts tested in the laboratory are shorter than 1 m on each side of the opening fracture. Based on the pull tests by Stjern [26], it is estimated that the critical bond length of a twin-strand plain cable bolt is approximately 2 m [10,27]. That implies that the plain cable bolt would not yield as demonstrated in the laboratory tests if the bond length was longer than 2 m. In the reality, all cable bolts are longer than 6 m. not all cable bolts, particularly those longer than 10 m, could slide in the grout under dynamic loading. That has been proven by broken cables observed in the field.

One reliable option to enhance the deformation of cable bolts is to mobilize the elongation capacity of the cable wires. The ultimate strain of the wires is only 3–5%. Therefore, a long section of the cable strand must be de-bonded in order to meaningfully improve the deformability of the cable bolt. A practical solution to construct a yielding de-bonded cable bolt is to wrap a smooth sleeve on the middle segment of the cable and leave an anchor length of 3–5 m in the distal end and a short anchor length in the proximate end, as sketched in Fig. 18b [29].

4.4. Thin spray-on liners (TSLs)

For over 30 years the Thin Spray-On Liner has variously been the subject of research and development, featuring a variety of laboratory tests and numerous underground mine trials, as a candidate yielding surface support component. In mining, at least, its functional and operational promise has yet to be realized. While the explanation for this is certainly technical, it is also clear that proving and communicating the operational requirements, as well as the economic case for why a mine should change from mesh and/or shotcrete liners to a TSL is not a trivial matter [30,31].

Technically, many attempts have been made to formulate and reformulate the properties of TSLs and of the polymeric materials which they comprise, generally in the interests of increased toughness and deformability as compared to fiber-reinforced shotcrete liners [11,32]. Aside from chemistry, the operational challenges of TSLs include quality control while spraying a thin (<5 mm thickness) liner onto a rough,

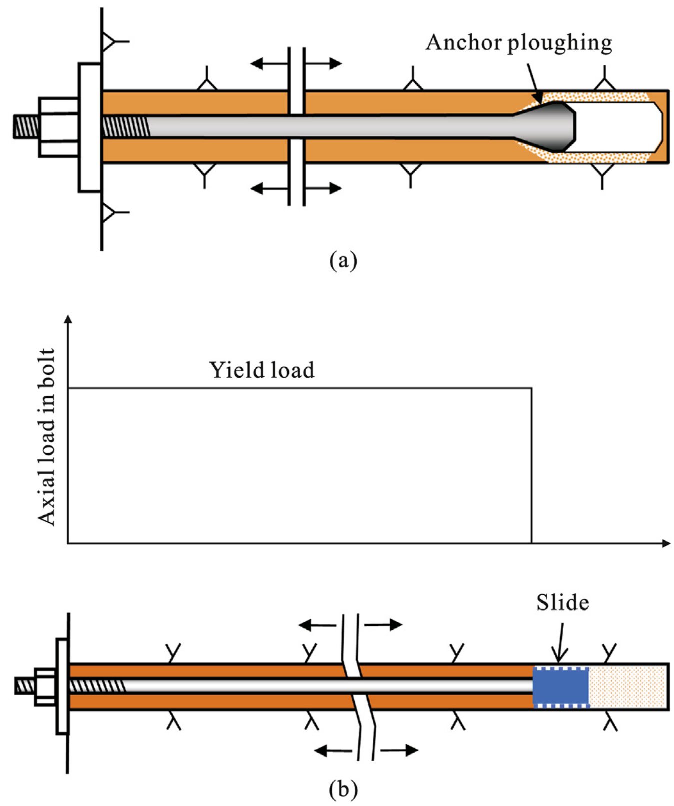


Fig. 17. (a) Sketch illustrating the configuration of a cone bolt and (b) the distribution of the axial load along the bolt length.

blasted rock surface, together with health and safety concerns, particularly in the effective management of by-product gasses in the form of methyl diphenyl isocyanate (MDI). Furthermore, the prospect of achieving face cycle time benefits in handling significantly reduced volumes of material compared to shotcrete has been hard to realize due to the extended cure time requirements needed to mitigate fly-rock damage when blasting the advancing round.

While the above challenges should be noted, over the years much progress has been made in understanding the functional specifications of TSLs with respect to ground support demands, particularly under failing conditions, including rockburst dynamics. For example, early formulations of TSLs were thought to require good tensile and adhesive strength properties - in equal measure. This would be true provided the liner is simply supporting *passive* loose between bolts and there is no expectation of an active rock failure condition over the life of the mine. In such an application a TSL's adhesive strength, of say 1.5 MPa or better after 1–2 h, should serve to “glue” and hold any loose in place. That is more in the interests of safety than of providing a ground support function. However, in the case of an active failing ground condition shown in Fig. 19a, such a high adhesive strength will prevent the TSL's toughness capacity from being mobilized, resulting in tensile and/or tearing failure of the liner due to high stress concentrations. This condition can be overcome by providing a means for the TSL to de-bond in response to an active and dilating rock failure state, as demonstrated in a laboratory test shown in Fig. 19b. Various tests have shown that for this to happen the TSL's limiting adhesive strength should be $\leq$ 0.25 MPa, depending upon its tensile strength [11]. This being the case and providing the toughness properties of the TSL are tolerant of the high strain rates that accompany strain-type rockbursts. Such a liner in theory at least could act as a truly yielding support component with good retention of loose between bolts. However, in the light of the discussion above concerning the mismatch in load bearing capacity between bolt and TSL, for the retention function to work in practice, the connection between bolt and TSL must be

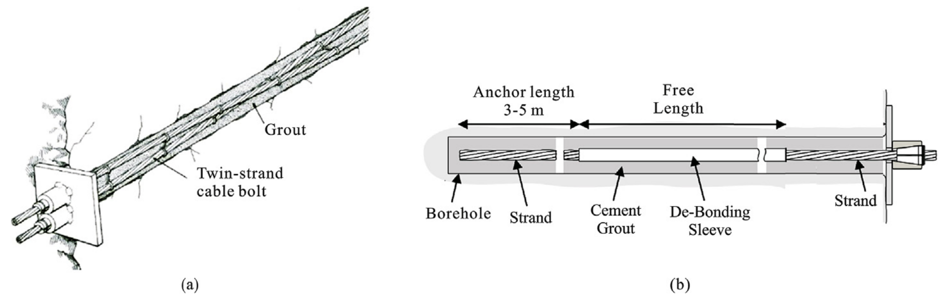


Fig. 18. (a) Plain cable bolt [24] and (b) partially de-bonded yielding cable bolt, modified from Thompson [28].

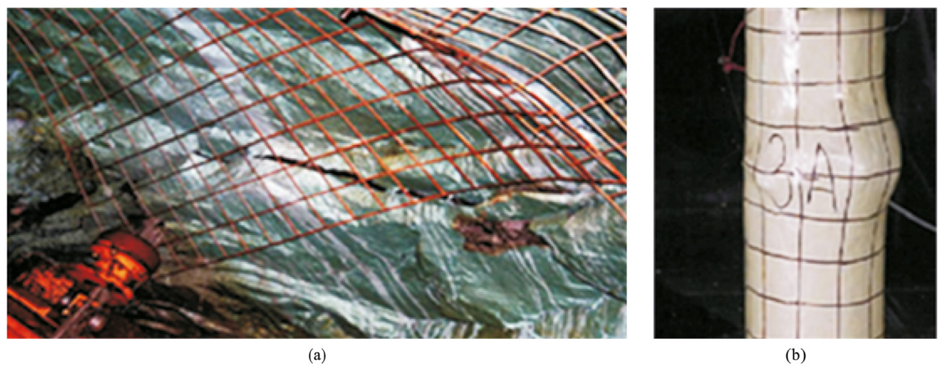


Fig. 19. (a) Example of a TSL in tensile/tearing failure mode due to excessive deformation of the underlying rock without adhesion failure; (b) example of compressive rock failure accompanied by adhesion failure, resulting in full retention of the failing rock.

sufficiently tenacious to survive the expected large differential movements between these two support elements.

4.5. Yielding pillars

It was noted above that while standard yielding ground support systems are capable of absorbing up to 10 kJ/m^2 and this makes them suitable given moderate strain bursting conditions, large fault-slip convergence-driven events require much greater energy absorbing solutions, on the order of MJ/m^2 that can only be found, realistically, on the scale of yielding pillars. Interestingly, a well-designed yielding pillar acts as a reinforcing ground support element and on a much larger load-bearing scale than rock bolts or cable bolts. This can be appreciated by reference to borehole deformation field data observed along the axis of a yielding ore pillar while mining an overhand cut-and-fill stope, Fig. 20. As mining advanced vertically over 9 cuts, the first 2 anchors from the collar show an upward deformation, indicating an increasing horizontal compressive stress condition in the stope floor Anchors located well inside the pillar and outside of the influence of the floor condition all show a downward or contracting strain state, indicating the pillar is unloading or yielding in a post-failure condition and a reinforced arch has been established with the overburden load carried by the abutments, Fig. 21.

In fact, much is known from Canadian hard rock as well as South African and US coal mining experience, about yielding pillar design and its application. Perhaps the greatest uncertainty lies in the necessary inequalities between local mine unloading stiffness, K_{LMS} , and the slope of the declining curve of a post-peak pillar, which was referred to as the pillar's post-peak stiffness by Salamon [35], K_P , that ensures stability at failure and load carrying capacity in post-failure, given by: $K_{LMS} - K_P < 0$.

The reason for this uncertainty lies in the difficulty of *a priori* obtaining an *in situ* local, full-scale mine stiffness determination, but instead depending upon a numerical model to obtain an estimate [36]. Nevertheless, given that a working yielding pillar design is obtainable, the rockburst mitigation benefits are easily appreciated with reference to Fig. 1 above and the discussion that followed concerning available

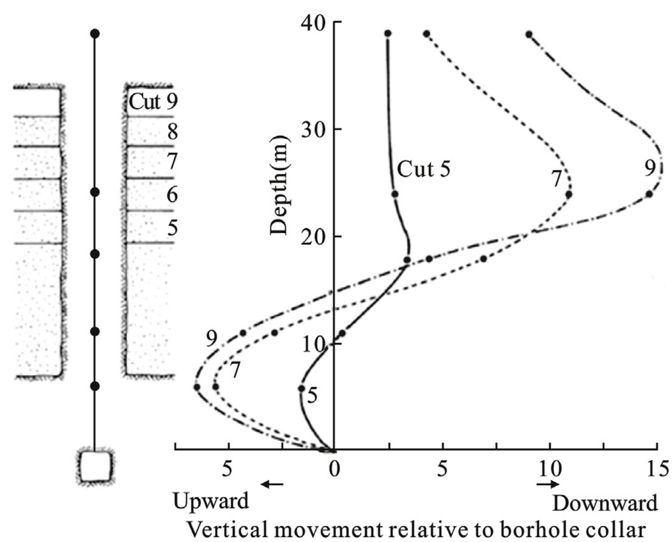


Fig. 20. Vertical deformation in a yielding pillar borehole relative to the collar and as a function of the advancing stope cuts [33].

seismic energy: a yielding ore pillar (or pillars), being a much larger scale support element than bolts, will a) reduce the volumetric convergence rate between footwall and hangingwall as a function of mining rate, and b) absorb energy as it compresses, otherwise available as seismic (i.e. rockburst) energy. Such pillars have also been shown to deliver a similar result, albeit on a smaller scale, when constructed of reinforced shotcrete and installed on the fractured footwall of a stope as the face advances in narrow vein mining [37]. In this case the yielding capability arises from the pillar's soft footing and not the shotcrete pillar itself, while the load-carrying and reinforcing action to the abutments illustrated in Fig. 21 is still obtained.

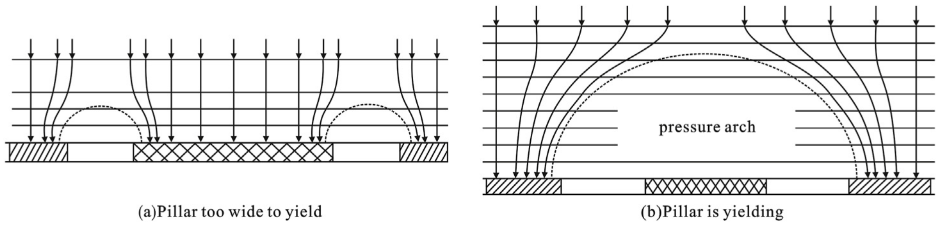


Fig. 21. Pillar width/height ratio is a key factor that determines whether it yields or not [34].

Fig. 22. Premature failure of plates and thread of the rock bolts.

(a) (b)

Fig. 23. The weak link between the plate and mesh: (a) Damage of the mesh by the plate edge, and (b) detachment of the mesh after deformation [38].

4.6. Weaknesses in the current dynamic ground support systems

All support elements (bolts, mesh, strap and shotcrete) and attachment accessories (plate, washer, nut, etc.) in a DGS system must have strong links in order that the system optimistically performs. The weaknesses in the current DGS systems are in the attachment accessories. The failure of a support system is often owing to the failure of the bolt plate, washer, nut and thread (Fig. 22), detachment of the mesh and shotcrete liner from the plate (Figs. 23 and 25), and opening of the overlap of the mesh sheets (Fig. 26). Too large plate hole, too small plate and too weak thread would lead to the failure of the plate and thread shown in Fig. 22. Use of washers, larger and dome-shaped plates and rolled threads could avoid such failures.

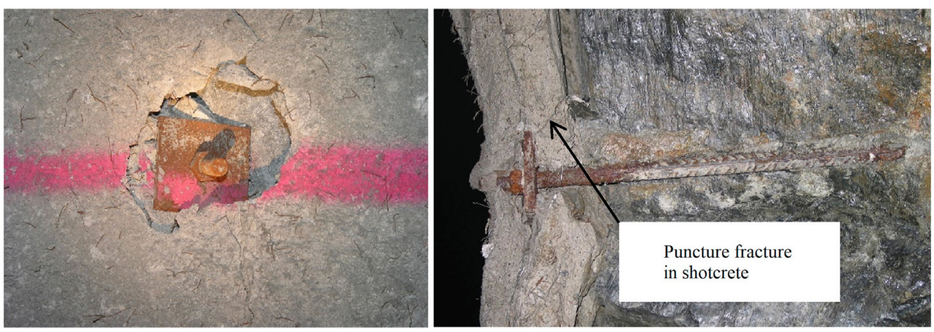
Stiff strong plates must be used in order that the bagging load of the mesh and straps can be transferred to the rock bolts. Sharp and stiff plate edges could damage the mesh wires in the stage of installation (Fig. 23a), and the mesh wires are cut off afterward at the edges when rock surface deforms under the ground pressure (Fig. 23b). One solution to reduce the stress concentration at the plate edge is use of dome-shaped plate (Fig. 24). The plate edges would be bent upward when such a plate is pressed in the apex of the plate dome by the nut/washer (Fig. 24b). Placing a strap pad under the plate even further improvement the loading condition (Fig. 24a).

Puncture failure could occur in deformed shotcrete liners, as shown in Fig. 25. The liners could be easily separated from the rock surface underneath fall after such puncture failures because of the low adhesion between them, particularly in chlorite and talk rich weak rock masses. Fiber-reinforced shotcrete liners can sustain a longer time than plain shotcrete, after puncture failure, but the load-bearing capacity is significantly reduced. A satisfactory solution to the shotcrete fall is to lay a mesh on the top of the shotcrete liner as illustrated in Fig. 24a.

The overlap of metal mesh sheets is the weakest position in the surface retention. It has been observed in many cases that the mesh overlaps are opened by either excessive rock dilation or by dynamic loading (Fig. 26). Use of stronger mesh straps as shown in Fig. 24a can improve the situation, but cannot completely get rid of the problem. Efforts are made at present to use rolled woven meshes to avoid overlaps. It may take a while to solve the problem.

(a) (b)

Fig. 24. Use of dome-shaped plates to avoid mesh wire damage by plate edge-cutting: (a) a dome-shaped plate together with a strap pad underneath after installation, and (b) the deflection of the dome-shaped plate after subjected to heavy rock deformation.



(a) (b)

Fig. 25. Puncture failure of the shotcrete behind the bolt plate.

5. Discussion

Any onset of a rockbursting condition in the context of a civil or mining excavation confronts the worker, engineer and management alike with safety and production-related decisions. While more remote and robotic methods of excavation are serving to help mitigate worker exposure at the advancing face of an excavation, in mining at least conditions are dynamic on a temporal and spatial scale that relates to changes with on-going production throughout the mine. In fact, Salamon [39] showed that seismicity or at least the ratio W_k/W could be expected to increase with the *rate* of excavation of any given geometry, Fig. 27. This theoretical prediction suggests that as mines go deeper their

Fig. 26. Weak surface retention in the overlap of the mesh sheets [38].

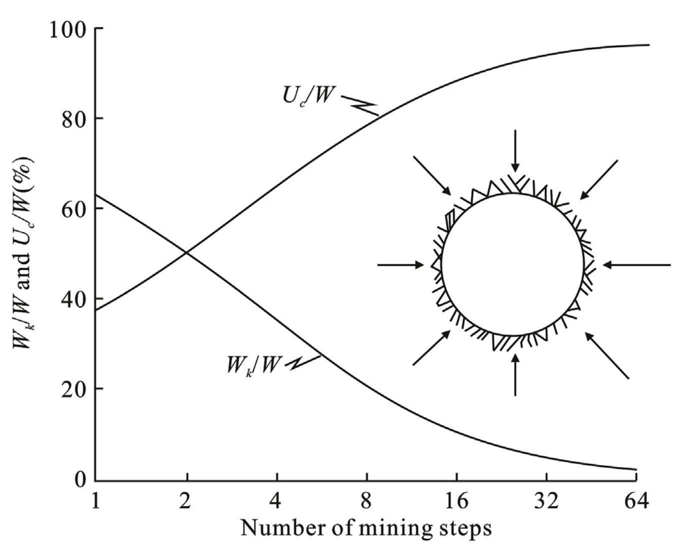


Fig. 27. A simple example showing the seismic energy dependency on the number (i.e. size) of mining steps while excavating a circular tunnel or shaft subjected to hydrostatic stress, after Salamon [39].

production rates will need to be reduced in the interests of mitigating an otherwise increasing risk of rockbursts, at least in magnitude if not in the frequency of events. Many mines in Canada and South Africa have had to face this reality, even as production costs increase for other reasons but all related to deep mining.

Additional and complementary actions, many noted above, relate to the general use of yielding ground support systems to absorb energy. Specifically, from bolts and liners on the small scale, to ore pillars or related strategic changes in mine design, on the larger scale, the latter in some way also contributing to a general reduction in production rate.

On the subject of effective ground support under dynamic loading conditions, a neglected area of understanding is that concerning the reinforced arch concept attributed to patterned rock bolts: well known for static loading conditions [e.g. 40] but, to the authors' knowledge, this is not the case with seismic loading. In "fair" to "very good" hard rock-masses a reinforced arch ensures that ground support systems never actually carry the full overburden load, but something approaching 50% of the dead load and, in mining at least, bolt capacity and bolting patterns are designed on this basis [29,41]. However, the validity of this concept under rockbursting conditions, given near-surface stress fracturing and/or stress wave reflection at the surface, is still an open question. Here it is suggested that some compromise of the reinforcement effect might be expected and this of itself could overload the support system(s).

6. Conclusions

For any ground support system to function effectively under the dynamic loading conditions associated with rock bursts, a clear understanding of the energetic demands of the seismic source, as well as the combined rock-support capacity to manage these demands, is imperative. Central to this understanding of risk is a recognition of the scale of the source mechanisms, knowing that ground support elements in the form of yielding bolts and liners have limiting capacities. In this paper these have been duly noted, together with suggested areas where improvements in capability as well as understanding could be made, as follows:

- The current DGS systems consist of yielding rock bolts with exceptional toughness capacity, surface support liners (mesh and/or shotcrete) and mesh straps. However, incompatibilities between the stiffnesses of bolts and liners, for which there is no easy solution, continues to challenge a yielding bolt's effectiveness;
- Polymeric thin spray-on liners have shown promise as having desirable yielding characteristics but have proven to be limited at this time by operational and quality control issues;
- The demands upon a rock-support system's capacity may be mitigated by reducing the incremental step/round size while advancing an excavation;
- Maintaining a rock-support system's reinforced arch effect subject to seismic loading may prove to be a capacity limitation, though this has yet to be conclusively demonstrated;
- Improvements to real-time decision-making concerning the location/timing for ground support system rehabilitation in a mining operation can be expected given the capacity of AI to process large, multi-faceted, 4-dimensional data sets;
- Where large fault-slip seismic events can be shown to be related to volumetric convergence in the case of deep mining, yielding ore pillars are theoretically appropriate, though their design must ensure against pillar bursts.

Declaration of competing interest

The authors declare that they have no known competing financial interests or personal relationships that could have appeared to influence the work reported in this paper.

Acknowledgement

The authors acknowledge the financial support by the Department of Geoscience and Petroleum in NTNU Norwegian University of Science and Technology in the preparation of this article.

References

[1] D.G.F. Hedley, Rockburst Handbook for Ontario Hardrock Mines, CANMET, Energy Mines and Resources Canada, Special Report SP92-1E, 1992.
[2] C. Graham, G. Swan, T. Guse, B. Simser, J.A. Bradley, N.W. McDonald, O. Matikainen, Thin spray-on liner: a project update ion current work in sudbury, Canada, in: *Proc. 23rd World Mining Congress*, 2013. Montreal, Canada, August 11-15.
[3] J. Spencer, A. Sliouniaev, Notes for Falconbridge on Visit to Norilsk, April 19-24. Internal Memo, Falconbridge Sudbury Operations, 1998 (available from G. Swan).
[4] M.G.D. Salamon, Energy considerations in rock mechanics: fundamental results, J. South Afr. Inst. Min. Metall. 84 (1984) 233–246.
[5] C.C. Li, T.B. Zhao, Y.B. Zhang, W.K. Wan, A study on the energy sources and the role of the surrounding rock mass in strain burst, Int. J. Rock Mech. Min. Sci. 154 (2022), https://doi.org/10.1016/j.ijrmms.2022.105114.
[6] B. Amadei, O. Stephansson, Rock Stress and its Measurement, Chapman & Hall, London, 1997.
[7] ISRM, in: R. Ulusay (Ed.), The ISRM Suggested Methods for Rock Characterization, Testing and Monitoring: 2007-2014, Springer Int. Publishing, Cham, 2014.
[8] M.H. Fillion, J. Hadjigeorgiou, Quantifying the impact of additional laboratory tests on the quality of a geomechanical model, Rock Mech. Rock Eng. 50 (2017) 1097–1121.
[9] D. Counter, Paste Fill Research at Kidd Mine, 5th Xstrata Ground Control Workshop, Elliot Lake, Ontario, 2008.
[10] C.C. Li, G. Stjern, A. Myrvang, A Review on the performance of conventional and energy-absorbing rockbolts, J. Rock Mech. Geotech. Eng. 6 (2014) 315–327.
[11] G. Swan, J. Hadjigeorgiou, A critical review of thin spray-on liners (TSLs), in: Report submitted to the Mining Initiative on Ground Support & Equipment (MIGS) II, WP 9 program, HeadMining AB, 2010, p. 62, dated 11th December.
[12] H. Wagner, Support requirements for rockburst conditions, in: 1st. Int. Symp. Rockbursts and Seismicity in Mines, Afr. Inst. Min. Met., Johannesburg, S, 1984.
[13] M.K.S. Roberts, R.K. Brummer, Support requirements in rockburst conditions, J. South Afr. Inst. Min. Metall. 88 (1988) 97–104.
[14] W.D. Ortlepp, An empirical determination of the effectiveness of rock bolt support under impulse loading, in: T.L. Brekke, F.A. Jorstad (Eds.), Proceedings of the International Symposium on Large Permanent Underground Openings, Australian Center for Geomechanics, Oslo, Australia, 1969, pp. 197–205.
[15] W.D. Ortlepp, The design of support for the containment of rockburst damage in tunnels e an engineering approach, in: P.K. Kaiser, D.R. McCreath (Eds.), Rock Support in Mining and Underground Construction, A.A. Balkema, Rotterdam,Netherlands, 1992, pp. 593–609.
[16] CAMIRO. Canadian, Rockburst Research Program 1990-1995: A Comprehensive Summary of Five Years of Collaborative Research on Rockbursting in Hardrock Mines, Canada CAMIRO Mining Division, 1996.
[17] B. Simser, P. Andrieux, F. Mercier-Langevin, T. Parrott, P. Turcotte, Field behaviour and failure modes of modified conebolts at the Craig, LaRonde and Brunswick mines in Canada, in: Challenges in Deep and High Stress Mining, Australia Center for Geomechancis, Perth, Australia, 2006.
[18] C.C. Li, A new energy-absorbing bolt for rock support in high stress rock masses, Int. J. Rock Mech. Min. Sci. 47 (2010) 396–404.
[19] T.R. Stacey, A philosophical view on the testing of rock support for rockburst conditions, J. South Afr. Inst. Min. Metall. 112 (2012) 703–710.
[20] L. Malmgren, E. Swedberg, H. Krekula, B. Woldemedhin, Ground Support at LKAB's Underground Mines Subjected to Dynamic Loads, 2014. Presented at Ground SupportSubjected to Dynamic Loading Workshop, Sudbury, Ontario, Canada.
[21] A. Roth, Full scale tests of mesh in combination with rockbolts subjected to dynamic loading, in: Presentation at ACG Ground Support Subjected to Dynamic Loading Workshop, 2014. Sudbury, Canada.
[22] C.C. Li, C. Doucet, Performance of D-bolts under dynamic loading conditions, Rock Mech. Rock Eng. 45 (2012) 193–204.
[23] G. Knox, A. Berghorst, Dynamic testing: determining the residual dynamic capacity of an axially strained tendon. Ground Support, in: J. Hadjigeorgiou, M. Hudyma (Eds.), Australian Centre for Geomechanics, 2019, pp. 231–242. Perth.
[24] B. Stillborg, Professional Users Handbook for Rock Bolting, second ed., Trans. Tech. Publications, 1994.
[25] A.J. Hyett, W.F. Bawden, R.D. Reichert, The effect of rock mass confinement on the bond strength of fully grouted cable bolts, Int. J. Rock Mech. Min. Sci. Geomech. Abstr. 29 (1992) 503–524.
[26] G. Stjern, Practical performance of rock bolts, Doctoral thesis 1995 52 (1995). Universitetet i Trondheim, Norway.
[27] C.C. Li, Principles of rockbolting design, Int. J. Rock Mech. Geotech. Eng. 9 (2017) 396–414.
[28] A. Thompson, Performance of cable bolt anchors – an update, in: Symposium, 2004, p. 8p.
[29] C.C. Li, Rockbolting: Principles and Applications, Butterworth-Heinemann, Elsevier, 2017, ISBN 978-0-12-804401-8, p. 286p.

[30] G. Swan, G. Doyle, S.A. Mikalachki, B. Martin, R.K. Brummer, Technical and business case arguments supporting the development of a TSL mining system, in: *Proc. 3rd Int. Seminar Surface Support Liners*, 2003. Quebec City, Canada.

[31] C. Graham, P. Andrieux, W. Blake, D.G.F. Hedley, E. Nordlund, D. Phipps, B. Simser, G. Swan, Rockburst case histories 1985, 1990, 2001 & 2013, in: Deep Mining Research Consortium, CAMIRO Mining Division, 2013, ISBN 978-0-9936394-0-1. Dec.

[32] Y. Potvin, T.R. Stacey, J. Hadjigeorgiou, H. Yilmaz, Thin spray on liners (TSLs) – a quick reference guide, in: Y. Potvin, T.R. Stacey, J. Hadjigeorgiou (Eds.), Surface Support in Mining, Australian Centre for Geomechanics, 2004, pp. 3–43.

[33] K.H. Singh, D.G.F. Hedley, Review of fill mining technology in Canada, in: Proc. Conf. Application Rock Mechanics to Cut and Fill Mining, Stephansson & Jones, 1981, pp. 11–24. Inst. Min. & Met.

[34] H. Yavuz, Yielding pillar concept and its design, 17th Int. Min. Congr. Exhib. Turk. (2001) 397–404.

[35] M.G.D. Salamon, Stability, instability and design of pillar workings, Int. J. Rock Mech. Min. Sci. Geomech. Abstr. 7 (1970) 613–631.

[36] A. Mortazavi, R.K. Brummer, The behavior of sill pillars and highly-stressed remnants, in: Report to CAMIRO Mining Division, Itasca Consulting Canada Inc., Sudbury, 2000, p. 76.

[37] H.S. MacIsaac, Swan G. Case Study, Strathcona deep copper mine, in: William A. Hustrulid, Richard L. Bullock (Eds.), Underground Mining Methods: Engineering Fundamentals and International Case Studies, Soc. Min. Met. & Exploration, Inc., 2001. Chpt. 41.

[38] B. Simser, The weakest link- ground support observations at some Canadian shield hard rock mines, Deep Min. (2007) 14p. Perth, Australia.

[39] M.G.D. Salamon, Rock mechanics of underground excavations, Denver, Colorado,, *Proc. 3rd Congr. Int. Soc. Rock Mech.* 1 (1974) 951–1099. Part B.

[40] T.A. Lang, J.A. Bischoff, Stabilization of Rock Excavations Using Rock Reinforcement, in: *23rd* U.S. Rock Mechanics Symp, 1982, pp. 935–944.

[41] N. Krauland, Rockbolting and economy, in: O. Stephansson, A.A. Balkema (Eds.), Rockbolting – Theory and Application in Mining and Underground Construction, 1983, pp. 499–507. Rotterdam.