Towards an understanding of dynamic demand on ground support

by Y. Potvin* and J. Wesseloo*

Synopsis

The proper understanding of the functioning of ground support under dynamic loading and the current approaches to designing of dynamic support are plagued by a great deal of uncertainty and lack of knowledge. This applies equally to the understanding of the support capacity as well as the demand placed onto the support due to dynamic loading. This lack of understanding currently leads to a case of design indeterminacy. This paper does not aim to solve this problem of design indeterminacy but to explore some of the issues that need consideration to obtain a better understanding of the dynamic demand on ground support systems.

Keywords
ground support, dynamic loading.

Background

The hazard associated with mine seismicity generally increases with stress, which in turn increases with depth. As the mining industry is becoming more mature, orebodies are mined at increasing depth. In South Africa, the deepest mines have already reached beyond 4 km, in Canada 3 km, and in Australia, where the horizontal stress gradient is relatively high, underground mining has progressed beyond 2 km. Deeper mining means that, in general, there will be more frequent occurrence of large seismic events and elevated seismic hazard.

The damage caused by a seismic event is commonly called a rockburst. There is a great deal of uncertainty and variability associated with the spatial and temporal occurrence of seismicity. On top of that, the interaction of the ground motion with the excavation and the installed ground support is extremely complex and is currently not well understood. The economic consequences may also be severe, especially when fatalities are involved. In Australia for example, the last two fatalities related to rockburst (Beaconsfield in 2006 and Big Bell in 2000) resulted in the mines being shut-down for periods exceeding well over one year.

The uncertainty combined with the potentially severe consequences leads to a generally high risk associated with rockbursting.

Fortunately, only a relatively small proportion of all seismic events cause damage. The main factors determining the rockburst damage level include the magnitude of the seismic event and its proximity to excavations. The larger and the closer the seismic event is to an excavation, the more likely it is to cause damage. Other factors also influence the severity of damage, including the localized ground conditions, the stress field around the nearby excavations and the capability of the ground support system to sustain dynamic loading (Heal, 2010), the source mechanism of the of the event, and the relative orientation of rock mass deformation and the excavation.

To mitigate rockburst risks, a combination of a strategic approach based on optimizing the mining sequence to minimize stress build-up on seismically active structures and a tactical approach relying on dynamically capable ground support is often implemented (Potvin, 2009). However, the engineering design of dynamically capable ground support systems suffers from major gaps in the current technology.

These gaps exist both in assessing the capacity of the support system and in estimating the dynamic demand. Although a brief discussion on the capacity is presented, this paper focuses on the demand estimate. The objective is to explore some of the issues that need considerations to obtain a better understanding of the dynamic demand on ground support systems.

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Support capacity

Ground support design must rely on the assessment of both the demand and capacity of the support system. Significant research work has been conducted over the last two decades towards a better understanding of the dynamic capacity of ground support (Yi and Kaiser, 1994; Kaiser *et al.*, 1996; Ortlepp and Stacey, 1997, 1998; Ortlepp *et al.*, 1999; Stacey and Ortlepp, 1999; Ortlepp and Swart, 2002; Gaudreau *et al.*, 2004; Plouffe *et al.*, 2008; Player *et al.*, 2004, 2008a, 2008b, 2009; Villaescusa *et al.*, 2005; Tannant *et al.*, 1993, 1994; Hagan *et al.*, 2001; Hildyard and Milev, 2001; Reddy and Spottiswoode, 2001; Espley *et al.*, 2002; Archibald *et al.*, 2003; Heal *et al.*, 2005; Andrieux *et al.*, 2005; Heal and Potvin, 2007; Potvin and Wesseloo, 2010).

In the work cited, the dynamic load applied to reproduce rockburst loading involved either some form of drop tests or blasting tests. Stacey (2012) rightly argued that neither of these is *'truly representative of rockburst loading, in a similitude sense*'. These tests by no means account for the complexity of the loading transmitted from the failing rock to the support elements, which likely involves a combination of several mechanisms including tensile, shear, bending, and torsion. It should be noted, however, that this was in many cases not the intention of the tests. These tests aimed to subject ground support elements to sudden impulse loading to enable a quantification of performance under dynamic loading. These tests should, therefore, be seen as index tests and the absolute values of, for example, energy absorption should be used with caution.

Furthermore, the drop testing programmes to date have generally looked only at ground support elements individually, with no means to account for the load transfer or cumulative energy absorption.

Stacey (2012) makes the point that although the results from testing individual elements do not provide useful data for deterministic support design purposes, they provide essential information for understanding the behaviour and comparing the capacity of support elements under tensile dynamic load.

Recent research work on improving our understanding of the dynamic demand on ground support systems is almost non-existent. Since the capacity of ground support systems and the demand from seismically induced dynamic loading cannot be quantified reliably, Stacey (2012) concludes that '*a clear case of design indeterminacy results*'.

This design indeterminacy can be solved only with the development of methods to quantify the true system capacity. In this regard the combined use of instrumented laboratory tests and detailed numerical modelling will prove valuable.

Challenges in understanding the dynamic demand on ground support systems

There are many challenges in trying to assess the dynamic demand on ground support systems encompassing every aspect of the problem from source to effect. These include the complexities of the radiation, refraction, and reflection of the seismic waves, their interaction with excavations, and the mechanism by which they load the support.

Seismic events produce dynamic stress waves which, based on the principles of physics, attenuate as they radiate from the source through the rock mass. Given that rock masses are complex and imperfect composite media, and most mines have complicated geometries, the wave propagation/attenuation effects from any seismic event can be extremely complex and difficult to model or effectively account for in design.

In particular, the effect of rock mass anisotropy on the seismic wave radiation pattern can be significant, as reported by Hildyard (2007). Depending on the stress field, the stiffness of the rock layers, and the presence of infill material between layers, the attenuation across lamination can be much more pronounced than along lamination. This is indeed difficult to take into account when estimating the attenuation of the stress wave as a function of the distance from the source, as the waves often travel through different rock mass domains with different degrees of lamination.

Radiation pattern

For the sake of simplicity, the radiation pattern from a seismic event is often ignored in rock engineering evaluation, and both the P-wave and S-wave intensity is assumed to be constant in all directions from the source. This is a simplification of a much more complex behaviour described by Aki and Richards (1980).

Figure 1 shows the radiation pattern of the S-wave and P-wave displacement in a plane of constant azimuth generated by a double-couple point source as presented by Aki and Richards (1980). The line graphs in Figure 1 shows the amplitude of the S- and P-wave for different orientations with respect to the direction of slip. The thin black arrows show the direction of the first motion of these waves.

The full three-dimensional amplitude distribution is shown in Figure 2. The radiation pattern, as shown by Aki and Richards (1980), is shown on the three-dimensional locus as a thin black line. The original figures by Aki and Richards (1980) are also shown orientated to fit that of the three-dimensional patterns. These figures illustrate the amplitude distribution but do not do justice to the complexity associated with the true motion.

Figure 3 shows the radiation pattern illustrated as vectors (Shearer, 1999). The direction of the first motion is shown by the orientation of the small arrows, while their length is proportional to the wave amplitude.

Figure 4 illustrates the importance of a due consideration of the radiation pattern in post-mortem analysis and design. These charts show a theoretical S-wave peak particle velocity

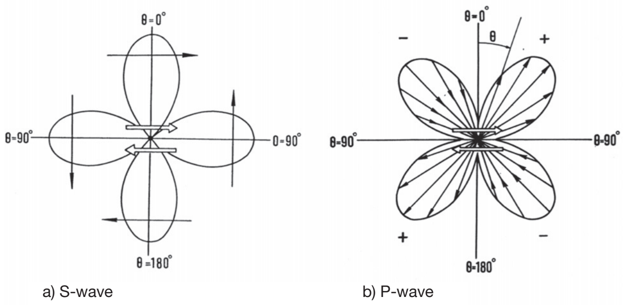

Figure 1—Illustration of the radiation pattern in two dimensions (original line drawing after Aki and Richards, 1980)

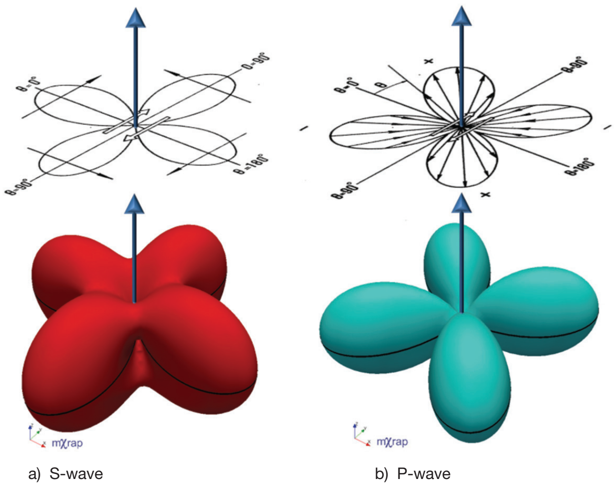

Figure 2—S and P wave amplitude distribution in three dimensions

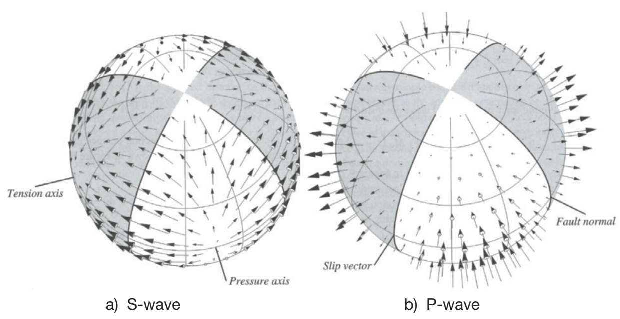

Figure 3—The far-field radiation pattern (after Shearer, 1999)

(ppv), calculated on different points in a mine following a seismic event. Both the colour and size of the points along the mine excavations are scaled according to the theoretical ppv. The difference between Figure 4 (a) and (b) is a result only of a different slip orientation. Figure 4 (c) assumes a spherical radiation pattern. Note that these figures illustrate only the influence of the radiation pattern. The effect of geological features, lithology, and mining voids on the radiating waves is not taken into account and is discussed in the following section.

Reflection and refraction of the stress wave

The presence of mine openings, major discontinuities and lithological contacts, and different lithological units will create both reflection and refraction as the stress waves radiate from the source. Daehnke (1997) used a very simple photo-elastic physical model to demonstrate the complexity of stress waves reflection and refraction when interacting with one long and narrow excavation, akin to a longwall front (Figure 5a). He also demonstrated that the presence of discontinuity planes creates further reflection and refraction of the waves (Figure 5b).

If one looks at the complexity of the geometry of a mature mine layout combined with its geological setting, which often includes multiple fault systems and lithological contacts, the difficulties related to simulating the reflection/refraction patterns of real seismic waves in a mining environment become self-evident.

One possible effect of this complex reflection/refraction phenomenon is the potential superposition of stress waves to create locally a very high dynamic loading condition. This could contribute (among other factors) to a very high localized load, resulting in the occurrence of isolated rockburst damage, often observed in underground mines (Figure 6) following a far-field seismic event.

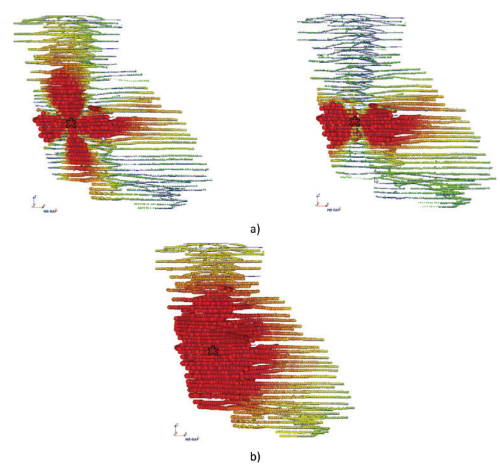

Figure 4—Illustration of the influence of the radiation pattern on the wave intensity in three dimensions. (a) Theoretical ppv resulting from a large seismic event. The difference in the two illustrations is a result of the assumed slip direction only. (b) Theoretical ppv resulting from a large seismic event when radiation patterns are ignored

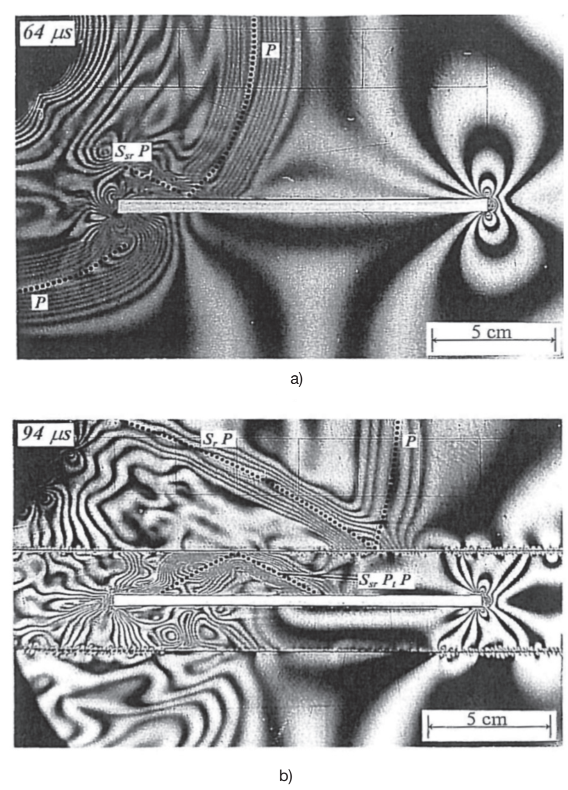

Figure 5—Photoelastic model showing the reflection and refraction of stress wave when (a) interacting with a long narrow opening, and (b) when interacting with a similar opening bounded by two discontinuity planes above and under the opening (after Daehnke, 1997)

Figure 6—Photograph of a localized incidence of seismically induced damage to ground support. Note that the surrounding support to the damage area is showing no signs of loading

The scale-distance relationship and the dynamic demand expressed as ppv

In many cases, rockburst damage occurs at several locations and at different distances from the theoretical hypocentre of a large event. Due to the radiation and attenuation, the intensity of the seismic wave reduces exponentially with distance from the source. To capture this effect into a single parameter for design purposes, traditionally the ppv is used in mining, while peak particle acceleration (ppa) is also used in other branches of earthquake engineering (St John and Zahrah, 1987).

Some have argued that ppa should be used, as this can be related to a force through the simple principle of Newtonian physics by multiplying the acceleration with a mass of ejection. This argument, however, ignores the complex wave-rock mass-excavation interaction. Kaiser and Maloney (1997) point out that only low-frequency accelerations, with wavelengths sufficiently long to accelerate the entire volume of rock in one direction, are relevant for damage prediction and support design.

St John and Zahrah (1987) argue that peak ground acceleration is not necessarily a good measure of damage potential because it is often repetitive shaking with strong energy content that leads to permanent deformation and damage. As a result, 'effective peak acceleration' has been used to refer to an acceleration which is less than the peak value but is more representative of the damage potential (Newmark and Hall, 1982).

McGarr (1983) showed that damage experienced correlated better with ppv than with peak ppa, and Kaiser *et al.* (1996) mention that:

'*...the ground motion velocity represented by the peak particle velocity ppv is accepted as the most representative parameter to define the dynamic design load*'.

A design methodology often used in mining assesses the energy balance (energy demand *versus* capacity of energy absorption), in which the demand can be determined from the kinetic energy of a mass moving at a given velocity as a result of a seismic event. An underlying assumption here is that the movement can be characterized by ppv and takes its origin in the stress (or strain) waves generated by a seismic event.

Having said this, it is fair to question whether ppv is the ideal parameter to assess the demand on ground support and whether the way it is generally applied and assessed is adequate.

Generalized ground motion relationships have been developed to capture the effect of attenuation in the ground motion with distance from the source. The most frequently used relationship is that presented by Kaiser *et al.* (1996) and Kaiser and Maloney (1997), which is often represented in chart form as shown in Figure 7.

The attractiveness of Figure 7 is its simplicity and the facility with which one can estimate the far-field and near-field ppv based on magnitude and location of events, which are readily available from seismic monitoring systems. As no other simple alternative exists, this graph has gained relative popularity with mine practitioners and researchers in recent years.

This generic form of the equation used was based on previous work by McGarr *et al.* (1981):

$$Log_{10}(R \cdot v_{\max}) = A \cdot M + Log_{10}(10) \qquad [1]$$

where:

V_{max} = the peak particle velocity
R = distance to the source (m)
M = the magnitude of the seismic event
A and C = are mine-specific scaling parameters.

The attenuation effect on ppv is in this case simplified by using two site-specific empirical constants that require calibration. Kaiser *et al.* (1996) used 95% confidence limit regression analysis on seismic data populations from Brunswick Mining, El Teniente mine and Creighton mine, together with data from McGarr (1984), to recommend using values of A = 0.5 and C = 0.25 if seismic data from the specific mine is not available for calibration. Kaiser *et al.* (1996) also specified that this should lead to conservative (high) estimates of ppv, since 95% of the data is 'below the regression line'.

Using the values for A and C proposed by Kaiser *et al.* (1996), Equation [1] can be re-written as (Heal, 2010):

$$v_{\max} = \frac{C \cdot 10^{a(M_R+1.5)}}{R} = \frac{0.25 \cdot 10^{0.5(M_R+1.5)}}{R} \approx \frac{1.4 \cdot 10^{M_R+1.5}}{R} \qquad [2]$$

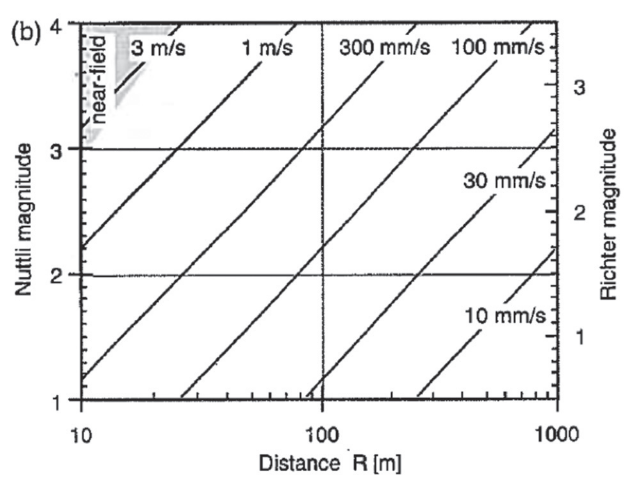

Figure 7—Peak particle velocity for recommended design conditions (90 to 95% confidence and normal stress drop; a* = 0.5 and c*= 0.25 m^2/s, (reproduced from Kaiser *et al.*, 1996)

Note that Kaiser's calibration was made based on the Nuttli magnitude scale (M_n), and for Local Richter scale (M_R): the following equivalence is suggested: $M_n = (M_R + 1.5)$.

These equations do not in any way account for differences in the ground motion in the near field. According to Aki and Richards (1980), the near field physically extends to once or twice the source radius. Although a few models have been proposed to quantify the near-field ppv, they rely on several assumptions. Durheim *et al.* (2005) further developed the original work of McGarr (1991) and present the ground motion relationship in the following form:

$$v_{\max} = \begin{cases} \dfrac{V_s \cdot \Delta\sigma}{G} & \text{for } R \le r_0 \\ \dfrac{V_s \cdot \Delta\sigma}{G}\dfrac{r_0}{R} & \text{for } R > r_0 \end{cases} \qquad [3]$$

where:

$V_{\max}$ = the peak particle velocity
V_s = shear wave velocity
$\Delta\sigma$ = static stress drop
G = shear stiffness of rock mass
ro = the near-field radius
R = hypocentral distance

Equation [3] shows a saturation of the ground motion in the near field. This near-field saturation has also been suggested by earthquake studies (Campbell, 1981). Wesseloo (2010) suggested a modification to the original far-field relationship suggested by (Kaiser *et al.*, 1996) to incorporate this saturation in the near field, and this is represented in Equation [4] and in Figure 8.

$$ppv = \frac{C \cdot 10^{\frac{1}{2}(m_L + 1.5)}}{R + R_0} \qquad [4]$$

where:

C = 0.2–0.3 is recommended for design purposes
R = the distance
R_o = the source radius (R_o) estimated as (Kaiser *et al.*, 1996)

$$R_0 = \alpha \cdot 10^{\frac{1}{3}(m_L + 1.5)} \qquad [5]$$

α = 0.53–1.14.

The near-field ppv is, however, complex, non-uniform, and not well understood, and investigation into this phenomenon is difficult due to the lack of reliable near-field data. As a result, these approaches to account for the near field may be inadequate oversimplifications.

The same criticism may be applicable to the use of the general ground motion relationship, and more reliable results may be achieved by deriving a site-specific relationship based on data obtained from each site.

As mentioned before, ppv is used as a design parameter to capture the severity of the influence of the seismic event at a distance. Unsurprisingly, the application of this type of oversimplified methodology often produces mixed results. Sometimes there appears to be a reasonable correlation between damage and estimated ppv, but this is often not the case. Ignoring the effect of ground support, when back-analysing rockburst damage case studies using this simple graph, it is common to have low ppv values creating extensive damage to the rock mass and high ppv values resulting in little or no damage. Morissette *et al.* (2012) plotted 133 cases of rockburst damage from the Creighton mine using a similar magnitude–distance–ppv graph (Figure 9). In this case, the generic ppv scaling attenuation law applied was the one proposed by Hedley (1992), which is similar to Equation [2] with different variable values, calibrated for Elliott Lake uranium mines. Morissette *et al.* concluded that based on this data, the relationship between the estimated ppv at the location of damage and the amount of damage appears to be random. They also made the important observation that:

'*...the amount of displaced material from a rockburst depends on more variables than the magnitude and the distance*'.

This highlights the fact that the complex nature of the problem cannot be reduced to a single design parameter. In this case, a design ppv, and any design methodology that is based on a theoretical ppv value without in any way accounting for the other factors influencing the excavation stability and support performance, will simply be inadequate.

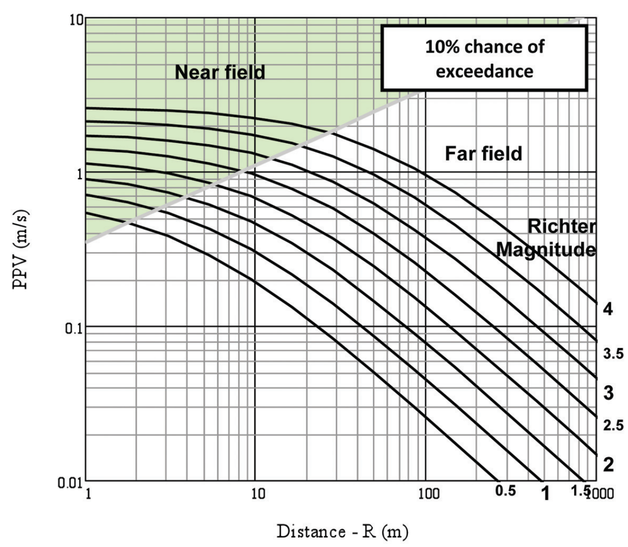

Figure 8—Graphical representation of the relationship between the magnitude–distance–ppv (after Wesseloo, 2010)

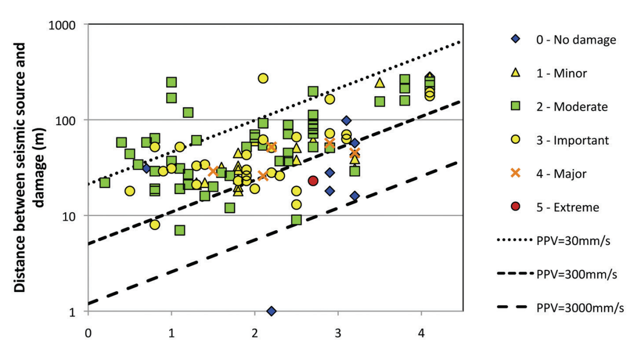

Figure 9—Magnitude-distance relation for rockbursts encountered at Creighton. Rockbursts are colour-coded based on rockburst damage index (reproduced from Morissette *et al.*, 2012)

Some of the effects that are likely contributing to the problem that need to be accounted for are: the effect of the radiation pattern, the complexities in reflection and refraction, local site conditions, and the effect of rock mass brittleness and the possibility of dynamic triggering of local brittle failure.

The site effect

From time to time, it has been observed that rock ejections following some rockbursts far exceeded the theoretical distance to which they should have travelled. Notwithstanding our poor understanding of factors such as anisotropy, reflection, refraction, and attenuation (as discussed above), which can all have a strong influence on the seismic wave propagation and therefore the damage and ejection observed, it has been proposed that a site amplification effect can be responsible for the discrepancy between theoretical and observed ejection distances. The site effect is often described in anecdotal narratives from rockburst damage observations. Milev *et al*. (1999) estimate that the ppv can be amplified by 4 to 10 times the expected value due to the site effect. Based on earlier work completed by Durrheim *et al*. (1998) and Hagan *et al*. (1999), Durrheim (2012) also suggested that the ground motion at the surface of excavations in South African mines can be amplified by a factor of 4- to 10-fold. Durrheim also proposed, as a possible explanation for the site effect, that the amplitude of the stress waves can be expected to double at the surface of an excavation (Figure 10). This is not surprising, given that the last layer of rock through which the wave travels before reaching the excavation is unconfined in at least one direction. It is reasonable to expect that the shape of the excavation, i.e a concave or convex curvature, will also influence the amount of amplification.

Durrheim (2012) explained that the fractured zone typically present around excavations at depth creates a contrast in velocity which contributes to

'*...trap seismic energy as the low velocity surface layer enhances the formation of surface waves such as Raleigh and Love waves*.'

The above proposition, combined with the possibility of wave superposition, emphasise the complexity of the stress wave interaction with excavations. Current design methodologies make an implicit assumption of a simple energy transfer akin to what happens in a 'Newton cradle' shown in Figure 11, while the reality is much more complex with repeated compression, shear, torsion, bending, and pure tension loading in the walls of the excavation.

Milev *et al.* (2002) have made a large number of ppv measurements at the surface of excavations at the TauTona mine, Kloof mine, and Mponeng mine using a custom design surface-mounted instrument called the Peak Velocity Detector (PVD). 'Theoretical' ppv values calculated from the seismic monitoring systems projected at excavation locations were then compared to ppv values measured at the surface of excavations from the PVD instrument. If one assumes that the ratio between these two values depends on a site amplification factor, measurements indicated that this amplification varied between approximately 1 and 25 times the theoretical ppv for each of the three mine sites, suggesting that the site effect may vary significantly, even within a mine. Webber (2000) correctly concludes that the site effect is highly variable from mine to mine and at this stage is poorly understood.

The quoted site factors of between 1 and 25 are likely to be the cumulative effect of different factors which include the radiation pattern, the complex interaction of the body waves with the geology and excavation, and the effect of surface waves. In order to improve the current approached in support design, these effects need to be disentangled and quantified. The combined use of laboratory testing and numerical modelling may provide a valuable avenue to achieve this.

The effect of rock brittleness

Beyond the difficulties in understanding and modelling the stress waves attenuation and propagation phenomena and accounting for the amplification of ppv due to the site effect, it is proposed that rock brittleness around excavations can also play a major role in the rockburst damage outcome.

Let us consider a certain volume of brittle rock located close to an excavation and submitted to a high stress regime. If loading of the volume of rock approaches its peak strength value, then it is conceivable that even a relatively small and/or attenuated stress wave from a far-field event may be sufficient to bring the rock mass beyond peak strength, and trigger a self-sustained violent failure near the excavation. This concept is similar to the idea of dynamic remote triggering of earthquake aftershocks (Kijko and Funk, 1996; Naoi, 2011, Butt, *et al*. 1998), which Kgarume (2010) suggests may be applicable to mining.

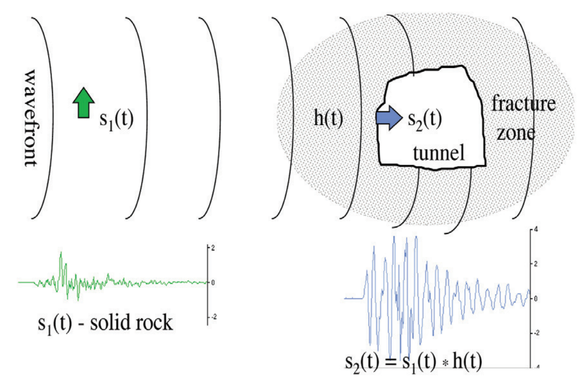

Figure 10—Diagram reproduced from Durrheim *et al.* (1996) showing the increase in the stress wave amplitude from seismograms recorded at 10 m from an underground drive (left) and at the surface of the drive (right)

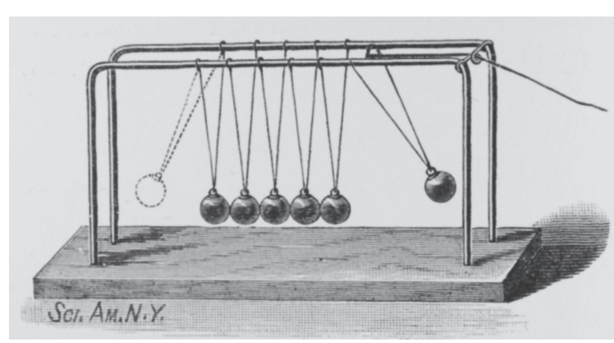

Figure 11—Newton cradle

This may provide an additional explanation to the amplification of the ppv due to the site effect, where extensive damage is observed as a result of low 'calculated' ppv from either a small event or a large but distant event (like the yellow and red points with ppv lower than 300 mm/s in Figure 2).

The work of Tarasov (Tarasov, 2010, 2011; Tarasov and Randolph, 2011; Tarasov and Potvin, 2012) provided evidence that for brittle rock under triaxial compression, a significant portion of the stored elastic strain energy is not consumed during the fracturing process. The unconsumed energy or the 'released energy' can then be transformed into the failure process dynamics, particularly associated with fragmentation, flying fragments, seismicity, heat, etc.

Figure 12 shows generic stress-strain curves of ductile rock, classified as Class 1, and brittle rock, classified as Class 2, based on the classification of Wawersik and Fairhurst (1970). The energy balance at three different stages of deformation is shown using the coloured polygons. The first graph on the left is at peak stress (point B), the middle graph is at an intermediate post-peak stage, and the right graph is at failure (point C). The red triangles represent the elastic energy stored in the rock specimen while the grey area is the energy consumed during post-rupture. The yellow area represents the excess energy and occurs only for brittle rock Class 2.

Tarasov and Potvin (2012) described the energy transformation as follows:

> 'The graphs illustrate the dynamics of transforming the *elastic energy accumulated within the specimen material at peak stress, into post-peak rupture energy. The red areas (elastic energy) are partly replaced in the graphs by the grey areas (rupture energy). The elastic energy represents the source of the post-peak failure process and provides the physical basis for the post-peak failure regime. For Class II, the fracture development occurs entirely due to the elastic energy available from the material. The failure process has a self-sustaining character, with the release of excess energy, corresponding to the yellow area (ABCD)*.'

Clearly, the above laboratory results show that brittle rock (Class 2) under triaxial compression and loaded at near post-peak stress (possibly located at a short distance from the excavation) is clearly capable of releasing a significant amount of energy during the post-peak failure process, even if the trigger is a modest stress wave from a far-field event. Therefore, adding to the possible site amplification effect and the possible superposition of stress waves, the strain elastic energy my also contribute to larger than expect ppv and damage.

Seismic events and rockburst damage mechanisms and the demand on ground support

A number of authors have proposed mechanisms for mine-induced seismic events (Gibowicz; 1990, Hasegawa *et al.*, 1989; Ortlepp, 1997; Hudyma, 2009). In particular Gibowicz (1990) offer a very simple distinction between two types of seismic events:

1 . 'Those directly connected with mining operations, i.e., those associated with the formation of fractures at stope faces
2. Those that are not, i.e. those associated with movement on major geological discontinuities'.

By definition, the damage associated with type 1 mechanism would generally be associated with the event's near-field ppv, and the issues identified earlier relating to propagation and attenuation and, to a lesser extent, anisotropy and even perhaps the site amplification effect, would not have a major influence on the demand on ground support. On the other hand, the stored elastic energy and more specifically the release of excess energy associated with brittle rock will most likely be a dominating factor in estimating the demand on ground support.

One can correlate the Gibowicz (1990) type 1 mechanism with the first three mechanisms (strain-burst, buckling, and face/pillar burst) proposed by Ortlepp (1997) shown in Table I. All three are closely associated with an excavation, are believed to be dominantly implosive motion, and are on the lower end of the magnitude scale. These mechanisms

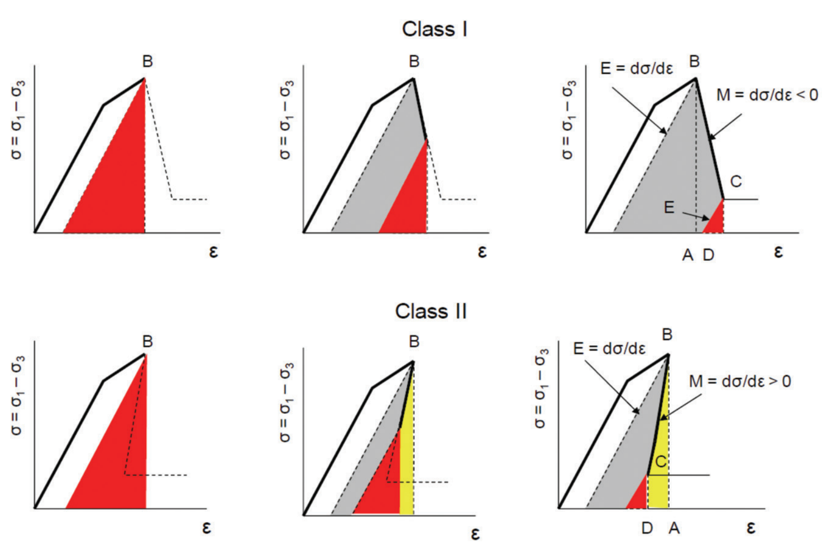

Figure 12—Illustration of the post-peak energy balance for rocks of Class I and Class II behaviour (reproduced from Tarasov and Potvin, 2012)

require a free surface to occur and thus occur at a lower confining stress. The violence of such a burst is expected to increase with lower confining stress.

Also by definition, the damage associated with the Gibowicz type 2 mechanism would generally be associated with, or at least triggered by, the far-field ppv. This makes the assessment of the demand on ground support more complicated as it is affected by all the factors described in the previous sections of this paper, including propagation/attenuation, site effect, and brittlenes. The Type 2 mechanism can be correlated with the last two mechanisms (shear rupture and fault-slip) of Ortlepp's Table I. Although they could be associated with excavations, they are often located remote from them in areas where confinement is significant, the motion is predominantly shear, and they account for the higher end of mine-induced seismic event magnitudes. They are also often responsible for major damage either by inducing sufficiently strong ground motion to overcome the capacity of the support or by triggering a Type 1 event at the boundary of the excavation.

Regardless of the seismic event mechanism, or the origin and nature of the ground motion, or whether the damage is caused by near-field or far-field ppv, the damage according to Kaiser *et al.* (1996) can be expressed as three distinct processes: rock fracturing, displacement, and ejection. Rock fracturing occurs when the peak strength of a volume of rock is exceeded due to either the incoming stress wave (Gibowicz Type 2 seismic event mechanism) or by sudden or gradual stress change due to a change in geometry after blasting (Gibowicz Type 1 seismic event mechanism). In both cases, the energy input in the system is from a transient stress wave of the seismic event itself or from the seismic event combined with the elastic strain energy (for brittle rock). These two potential sources of energy will be dissipated as rock fracturing, and the unconsumed energy will be released in rock mass displacement (or bulking) and ejection. The last two (bulking and ejection) will likely be transferred to the ground support system.

Figure 13 illustrates how the support system is loaded. The rock fracturing and bulking will contribute to stretching of the surface support reinforcement (shotcrete or mesh) to a displacement D. The displacement of the reinforcement d will be the result of the surface movement transferring tensile load to the bolt together with the internal bulking of the rock mass, which also produces an axial load on the bolts. The support system will be stable if the portion of the energy from the seismic event combined with the stored strain elastic energy (in the case of brittle rock) unused during the fracturing process can be dissipated by deformation of the ground support system. The ground support system capacity to dissipate this energy can be very low if one component of the system is weak (often referred to as the weakest link) (Simser, 2007).

Conclusion

In this paper we examined some of the complexities in dynamic support design and the inadequacies of current approaches. The current state of knowledge does not allow determination, with any confidence, of the capacity nor the demand of ground support systems under dynamic loading. Therefore, this is an indeterminate problem. Peak ground motion (or ppv) has been favoured to date to characterize the dynamic demand, but the ppv as a design parameter is insufficient to adequately capture the complexity of the problem.

If the seismic event is further away from an excavation, the radiation pattern and the complex interaction between the seismic waves, geology, and mining excavations must be

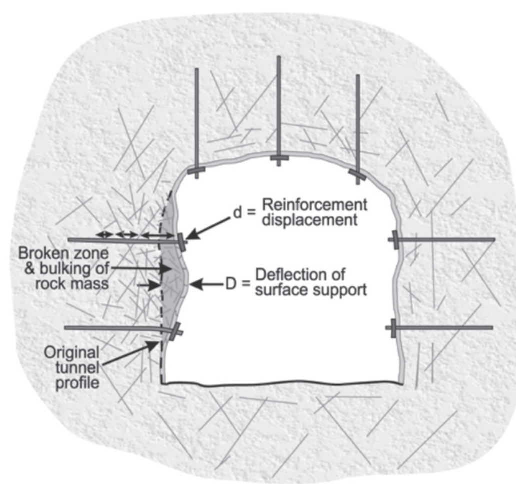

Figure 13—Illustration of the support system deformation as a result of rock fracturing and bulking (after Potvin *et al.*, 2010)

Table I

Classification of seismic event mechanisms modified after Ortlepp (1997)

Confining stress	Seismic event	Postulated source mechanism	First motion from seismic records	Richter magnitude ML
Increasing confining stress	Strain-burst	Superficial spalling with violent ejection of fragments	Usually undetected, could be implosive	-0.2 to 0
	Buckling	Outward explosion of large slabs pre-existing parallel to surface of opening	Implosive	0 to 1.5
	Face crush / pillar burst	Violent explosion of rock from stope face or pillar sides	Mostly implosive, complex	1.0 to 2.5
	Shear rupture	Violent propagation of shear fracture through intact rock mass	Double-couple shear	2.0 to 3.52
	Fault-slip	Violent renewed movement on existing fault or dyke contact	Double-couple shear	2.5 to 5.0

accounted for. The local site effect and the phenomenon of triggering of local strain-burst or buckling type events in stressed brittle rock also needs to be further investigated.

The contribution from each of these mechanisms must be quantified to determine the dynamic demand on ground support

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