THIN SPRAY-ON LINERS: ASSESSMENT OF SUPPORT PERFORMANCE UNDER DYNAMIC LOADING CONDITIONS

By

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ABSTRACT

The effectiveness of ground support systems under dynamic loading conditions is of prime interest to the mining industry for the successful and safe operation of deep mining operations. All ground support systems must have static capacity to provide stability against gravity induced failure mechanisms. However, to ensure excavation stability and safety in seismically active areas and in blasting environments, the support system must also offer dynamic capacity.

A method of measuring and quantifying the dynamic capacity of ground support systems has yet to be established. There have been some attempts at measuring dynamic capacity in a laboratory setting, however very little information exists on large-scale in-situ testing programs. Proposed within this thesis is a large-scale dynamic load test methodology and analysis technique.

The focus of this large-scale dynamic load test program was to collect support performance data specific to thin spray-on liners (TSLs), as it pertains to dynamic loading. To fully examine support performance, multiple data types were collected throughout the testing program. The integration of the various data types and formats required a unique analysis process, from which the dynamic limit of five ground support systems was defined. The definition of the dynamic limit allows for an improved underground support design in areas of anticipated seismicity.

With the availability of dynamic load capacity data, a refinement of the support design for TSL systems in underground excavations was possible. The CRRP (Canadian Rockburst Research Program) have proposed a support design guideline for dynamic loading conditions. Additionally, Espley and Kaiser have developed a unique support design guideline specific to TSLs. The author expanded the existing TSL support design proposed by Espley and Kaiser by including a dynamic loading component. Through integration of the design principles proposed in the CRRP with the existing TSL design guideline, an improved TSL support design process was generated.
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DEDICATION

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1 INTRODUCTION

It is suggested that thin spray-on liners (TSLs) may offer a number of advantages to underground operations. Firstly, TSLs offer a unique support advantage over traditional support tools under certain conditions. Characteristics such as high tensile strength and improved elasticity (flexibility) allow TSLs a unique functionality not available with current support tools. Secondly, the operational aspects of a TSL offer an advantage to mining operations, with the primary benefits derived from its ability to achieve 100 percent strength in just minutes and its minimization of material handling requirements. Such a benefit would allow mine operators to improve cycle times significantly through a decrease in the support installation process time. Finally, the latest research indicates that TSLs may possess superior support performance under dynamic loading conditions as compared to traditional support tools. It is the dynamic support benefit that will be investigated throughout this thesis.

1.1 Background

1.1.1 Ground Support Theory

In simplified terms, the main purposes of a ground support system are to ensure worker safety, prevent damage to equipment and enhance ore zone access. However the science surrounding ground support engineering is not nearly as simple. The principal objective of a ground support system is to help the ground support itself (Hoek and Brown, 1980; McCreaht, 1999). Hence the goal is to enhance the existing self-supporting capabilities of a rockmass.

In almost all mining situations, there is some horizontal stress in the rockmass, which prevents the rock from immediately falling when an opening is created. This compressive effect within the rockmass allows the rock the potential to be self-supporting by creating a beam (in compression) along the top of an opening. However, based on the conditions of the rockmass zone and the opening, such as a low horizontal stress or a very wide opening, the rock (or beam) may sag into
the opening and a zone of tension may develop (See Figure 1-1). It is in this zone of tension that the rockmass has the potential to fail (or fall out). Preventing that from happening however are the horizontal stresses, which arch up and over the opening, creating a zone of increased compression and hence a more stable opening. The rock blocks keyed together (through compression) up above are referred to as a "Voussoir arch". The lower tension zone below the keyed blocks are suspended from the stable Voussoir arch, preventing rockmass failure in the back (Diedrichs & Kaiser, 1999; McCreath, 1999; Parker, 2001).

![Diagram of rockmass mechanics](image)

Figure 1-1: a) Jointed rock beams; b) Voussoir arch analogue

(Diederichs and Kaiser, 1999)

This self-supporting mechanism is greatly affected by rockmass conditions and localized effects. Firstly, the depth of the tension zone will be a significant factor in determining whether the rockmass and excavation can maintain stability or not.
That shape of the zone (and hence the depth) is largely controlled by the ratio of
the horizontal stresses to the vertical stresses. Within moderate limits, a high
vertical stress forms a high elliptical arch (possibly to a point above the disturbed /
undisturbed rock contact) and a high horizontal stress forms a low elliptical arch.
If the state of stress is such that the tension zone exceeds the depth of the broken
rock, the compression zone (Voussoir arch) offers no assistance in retaining the
rock blocks, since the compression zone is located up above the rockmass
"beam". Under these conditions, the disturbed rockmass (or rock blocks which
make up the beam) immediately above the excavation has no natural retaining
forces and if left unsupported will unravel.

In blocky ground, the formation of wedges provides an opportunity for rock arch
instability, where the potential for gravity induced wedge failure may compromise
the integrity of the rock arch. As mentioned above, the rock arch is made up of
multiple pieces of rock or blocks locked together by (a) compressive forces just
above the excavation and (b) shear strength between the blocks. The coupling
between the rock blocks and the shape of the arch allows for the loading force to
be shed onto the abutments or walls of the excavation. However if one block or
piece in the arch is removed, as in the case of gravity induced wedge failure,
there is a loss of continuity within the arch, creating a barrier to the flow of
stresses towards the abutments. The stresses need a coupled material to travel
through in order to reach the abutments and provide the stabilizing mechanism of
the arch. When a piece of the arch fails, the ability to impose compressive
strength on the remaining blocks is no longer available and the shear strength of
the blocks immediately adjacent to the lost piece is reduced to zero. Under such
conditions, the rockmass is unable to remain self-supporting and ground support
is required to maintain excavation stability.

The principal concept behind ground support design is to enhance the self-
supporting capabilities of the rockmass. Through the installation of ground
support, the stabilizing effects of the rock arch are enhanced. One natural
stabilizing mechanism of the rock arch is the compressive strength of the rock
blocks; coupling the blocks together and allowing for stress shedding to the abutments. By installing tendon support, internal reinforcement of the rock arch is achieved in much the same was as the presence of reinforcing steel acts in reinforced concrete. By knitting the rockmass together and by limiting the separation of individual blocks, the reinforcing elements limit the dilation in the rockmass immediately surrounding the tunnel (Hoek and Brown, 1980). A second stabilizing mechanism within the rockmass is the shear strength between the rock blocks. By preventing any dilation between the rock blocks, the shear strength can continue to contribute to the overall rockmass stability. Through tying (or binding) the rock blocks together and fastening them to the intact rock strata, a ground support system works with the rock to maintain excavation stability.

It is generally agreed that it is very difficult for support to be able to hold up large volumes of dead-weight loading once the rockmass has loosened (Hoek and Brown, 1980). All rockmasses have some self-supporting properties, however based on the characteristics of the rockmass and the excavation; there are limitations to those self-supporting mechanisms, limitations which are often a function of time. Under some conditions, the rockmass may begin to unravel almost immediately upon excavation; in other cases unraveling may only occur some time later and yet again in other situations unraveling of an excavation may only occur in response to the creation of a new excavation in the nearby vicinity. To prevent the occurrence of unraveling, consideration must be given to the timeliness of the support installation. In fact, for most underground situations in low to moderate stress levels, ground support should be installed as quickly as possible within newly blasted headings. Fast support installation limits the amount of rockmass loosening and aids in preventing large volumes of dead-weight load from developing.

Two general rules in selecting a ground support system are: (1) help the rockmass support itself and; (2) if rockmass begins to fail, allow deformation to occur in a controlled manner (McCreath, 1999). The remainder of this section provides a list of the main elements generally considered when selecting a ground
support system and describes the mechanism through which each element contributes to supporting the rock.

**Mechanically Anchored Bolts**

- Mechanically anchored rockbolts connect weak, thin strata together to create thicker, stronger strata.
- By tensioning the bolt, the rock-support interaction is immediate or active; bolt head is engaging a force upwards on the rockmass blocks.
- Rockbolts provide a supporting connection between loose surface rock and deeper intact rock.
- A fairly flexible, soft system allows for some deformation within the rockmass.

**Grouted Rebar**

- Similar to mechanically anchored bolts, but anchoring occurs over the full length of the bolt, coupling the bolt to all rock zones along the length of the bolt. A continuously anchored/tensioned bolt protects against long-term loss of tension.
- Bolt can still be tensioned to provide “active support”.
- A stiff element with very little yielding capacity – minimal ability to deform with the rock. Any movement that occurs (in excess of the rebar support capacity) will like break the grouted bond between the bolt and the rock or break the bolt itself.

**Cablebolts**

- Used in the same sense as rockbolts and rebar but exceed the capacity of the
aforementioned supports.

- Radial displacement or dilation of the interface between the grout and the cablebolt (when tensioned) creates confining stress on the rock as the strands in the bolt bulk. This in turn increases frictional resistance of the bolt to pulling out.

**Friction Bolts**

- Installable in broken ground
- Allows yielding of ground in axial direction
- There is a high anchorage capacity (over the full length of the bolt), but minimal tensioning capabilities.

**Mesh and Screen**

- Mesh and screen are used to retain smaller pieces of loose rock that have the potential of falling from in between the bolts. By preventing smaller pieces of rock from falling, potential unraveling is also prevented.
- Mesh has no confining strength initially; it allows the rockmass to deform – acts as a “passive support” initially.

**Shotcrete**

- Acts like a “super-mesh”. Shotcrete has stronger load capacities but lacks the deformability of mesh.
- Nature of application creates “key-block” support – stops onset of loosening.
- Shear strength and adhesion distributes loads.

Based on rockmass conditions, a ground support system will usually consist of a combination of the above described elements. By identifying the support requirements and selecting elements with matching support capacities, a much stronger rock arch can be created, allowing for a safer underground opening.
1.1.2 Thin Spray-On Liners

A variety of passive area support techniques have been used in the mining industry to act as the retaining component in a ground support system. The retaining component of a ground support system is considered to be an important element; preventing the dislodgement of key blocks and hence increasing the self supporting capabilities of the rockmass. Traditional tools such as screen and shotcrete have been used almost interchangeably in the past, yet the development of support resistance for each system is widely different.

Thin spray-on liners are a new innovative support tool that act like a passive support, in the same sense that shotcrete and screen are passive support tools, but have support capabilities that fall somewhere in between those of shotcrete and screen (Archibald et al. 1999). With the welded wire mesh or screen system, support resistance is only offered after significant displacement of broken rock has occurred. The support resistance is achieved only when tensioning of all local strands within the mesh has been mobilized, typically through large rockmass deformation. At small deformations, the load is distributed onto adjacent strands; and not until enough deformation has occurred to allow for load distribution onto all strands will the support offer any resistance to the rockmass. In the case of shotcrete, resistance is offered against small rock movements through its compressive and shear strength properties. However, if rock movement does occur (if load exceeds resistance capacity) fracturing and failure of the shotcrete will typically lead to full system degradation. Mesh- or fiber-reinforced shotcrete can resist some rock displacement, up to tens of millimeters, before reaching full system failure, but in contrast to screen such displacement is minimal.

As discussed above, the objective of ground support is to mobilize and conserve the inherent strength of the rockmass local to excavations so that it can remain self-supporting (Archibald et al., 1999). Like traditional forms of passive support, TSLs act to provide resistance to rock deformations when subjected to tension. By resisting rock deformation, the individual fractured pieces of rock about an
excavation can be held in place and interlocked, hence allowing a rockmass to retain its frictional strength reinforcement capabilities.

Effective TSL support is created by a two-component liquid chemical system consisting of a polyisocyanate and a resin mixture which combine upon application to form a continuous membrane that is firmly adhered to the rock. Support is provided by a TSL in the form of improved frictional resistance to shear movement of blocks through reinforcement of existing fracture surfaces. The liner is sprayed at high velocity onto the excavation surface, during which small open joints or cracks are infiltrated to effectively glue the surface of the rockmass together with a tough membrane coating. In addition to resisting rockmass movement, thin spray-on liners retain broken rock through mechanical strength properties, such as adhesive strength and tensile strength. By examining the potential failure mechanisms of TSLs (as defined by Espley, 1999), the support concept behind the liners will be more clearly understood.

Espley identified four potential mechanisms of failure relating to thin spray-on liners:

1. Adhesive Failure

Adhesive Failure: Loss of adhesion can occur either by debonding of the liner and the substrate material or by delamination of the substrate itself, whereby weak and friable rock becomes detached from the surrounding rockmass. Espley states that liner adhesion is critical for distributing and transferring applied loads across the rockmass surface and within the liner. Furthermore, adhesion prevents the loosening of the rock blocks, hence aiding in the self-supporting capacity of the rockmass as described above. A typical adhesive strength of a high quality TSL is greater than 0.5 MPa.
2. Tensile Failure

*Tensile Failure*: Tensile failure occurs when adhesion is lost along a joint surface and large rock deformations are permitted. As the liner is displaced and stretched, the tensile loads in the liner material are increased. At high tensile loads, the TSL will generally rupture along a plane perpendicular to the liner surface. A typical tensile strength of a high quality TSL ranges from 5 MPa to 15 MPa.

3. Direct Shear Failure

*Direct Shear Failure*: Direct shear failure could occur when the adhesive strength is greater than the shear strength of the material. This failure mode is associated with low rock displacements, no debonding of the liner, and a shear failure plane that is perpendicular to the excavation surface. In practice (as noted by past researchers), direct shear failure has not been an observed failure mode of TSLs.

4. Diagonal Tensile Failure.

*Diagonal Tensile Failure*: Diagonal tensile failure could occur with small rockmass displacements, where the tensile strength is exceeded on a diagonal rupture plane (i.e. at 45°). Similar to the direct shear failure, this mode of failure has not been observed with any of the TSL testing, since tensile rupture is most often associated with some adhesion loss and with some rock displacement.

Overall, the adhesive strength determines the ultimate support capacity and failure mode of the spray-on liner, as follows: (1) – if adhesion were maintained, the liner could fail by either of: (i) direct shearing or (ii) diagonal tensile failure, (2) – if the adhesive bond is lost, the liner will eventually fail under direct tension loading. From the accumulated experience, only the latter mode (2) seems to be of practical relevance (Espley, 1999).
1.1.3 Ground Support and Dynamic Loading Conditions

For all underground environments, ground support systems must have static capacity to provide stability against gravity-induced failures. The above sections have focused on the support tools and mechanisms dedicated to maintaining stability under static loading conditions. However, in many cases additional dynamic capacity is needed too – for application in burst-prone areas and in blasting environments.

1.1.3.1 Dynamic Loading Conditions

Dynamic loading, in the context of this thesis, refers to a sudden violent change in load. When such loading conditions occur, and the stability of an excavation is compromised, the event is often referred to as a rockburst. Rockburst is a general term that is defined by the Canadian Rockburst Research Program 1990-1995 (CRRP) as: damage to an excavation that occurs in a sudden or violent manner and is associated with a seismic event. The term rockburst is often used to describe many different types of dynamic loading conditions such as pillar burst, crush burst, strainburst, fault slip event and even a seismic event itself. Included under the rockburst category may even be blast-induced damage to an excavation.

The source or the cause for each type of dynamic loading condition listed above is vastly different, however the damage mechanisms inflicted onto the rockmass from any dynamic load is similar. To further understand the concept, it is important to differentiate between source mechanism and damage mechanism. The source mechanism is the means by which a seismic event is generated or, in other terms, it is the release mechanism of stored energy within the rockmass. An example of a source mechanism is a fault slip event. The damage mechanism is the mode by which an excavation or rockmass loses its integrity and/or stability. It is the damage mechanisms that are of prime interest when examining ground support for dynamic loading conditions, as opposed to the actual event triggering the damage. Rather than attempting to control the source mechanism itself,
support techniques are employed to control the damage imparted onto the excavation. Hence the remainder of this thesis focuses on the effects of ground support in mitigating dynamic load induced-damage to an excavation.

According to the CRRP, in focusing on the damage mechanism, there is another factor that is important to consider: severity. By assessing both the damage mechanism and the severity, the demand placed upon the ground support system can be defined. Contributing factors such as existing stress levels, the quality of the rockmass, the excavation shape and the seismic source characteristics do affect the damage mechanism and the severity. By fully studying these two broad aspects, the nature of the demand can be more clearly understood, facilitating an engineered approach to ground support design under dynamic loading conditions.

As mentioned above, the rockmass damage mechanisms are similar for any dynamic loading condition. Based on the Canadian industry experience, there are only three distinct mechanisms that are involved in most of the damage caused by rockbursts in Canadian mines. As listed in the CRRP, in order of priority or frequency of occurrence, these mechanisms are:

- Sudden volume expansion or bulking of the rock due to fracturing of the rockmass around an excavation;
- Rockfalls (or falls of ground), which have been induced by seismic shaking;
- Ejection of rock due to energy transfer from a remote seismic source.

Each of the damage mechanisms listed above however may vary in terms of the level of damage (or severity) to an excavation and support system. Various damage levels that were observed to occur in association with rockburst phenomena have been defined (Kaiser et al., 1996).
Minor
- Commonly described as rock spitting, spalling or shallow slabbing.
- Damage would involve only a shallow skin of fractured or loose rock, generally less than 0.25 m in thickness.

Moderate
- Rock is heavily fractured and may have displaced violently.
- Holding elements will have failed but volume of broken rock is limited such that drifts are still accessible.
- Fractured or loosened rock is 0.25 – 0.75 m in thickness.

Major
- Drift becomes impassable due to substantial amount of displaced rock.

The goal of a ground support system under dynamic loading conditions is to control damage severity associated with these three damage mechanisms. In doing so, a support system must be able to: (1) survive the displacement and (2) remain functional. All three mechanisms (bulking, rockfalls and ejection) can lead to displacement or expansion of the support system, which for stiff and brittle elements (like shotcrete) may lead to support failure. For this reason, it is important that systems selected for dynamic loading conditions be capable of expansion and/or displacement. By employing a system that is capable of displacement, the support capacity is not lost through energy transfer within the rockmass and excavation. The allowance of movement within a support system demonstrates the element's capability to absorb and transfer energy. With this characteristic, a support system is capable of maintaining support capacity (remaining functional) following the occurrence of a dynamic event. Therefore, by surviving the displacement and remaining functional, a ground support system can control the damage mechanisms imparted under dynamic loading.

1.1.3.2 Support System Characteristics for Dynamic Loading

In accounting for both static and dynamic loading conditions, there are three primary functions of a ground support system (McCreath and Kaiser, 1992).
Firstly, it has to **reinforce** the rockmass (or strengthen it) in order to capitalize on the self-supporting capabilities of the rockmass and prevent the failure of the rock excavation (Hoek and Brown, 1980). When this is unsuccessful, it then has to **hold** the failed rock and try to control the amount of displacement that occurs. The holding function ensures that the support element maintains contact, through anchoring, with the stable intact rock. Finally, a support system must be capable of **retaining** the failed rock and absorbing the energy with which this material is being forcibly driven. Figure 1-2 is a simplified view of the three primary support functions.

![Diagram](image)

**Figure 1-2: Three Primary Functions of Support Elements (Kaiser et al., 1996)**

To design a support system capable of demonstrating all three of these functions it is necessary to incorporate multiple types of support elements. As mentioned earlier, ground support systems are typically constructed by combining individual support elements into an integrated system. Since each type of support element has a unique set of support characteristics, it is highly unlikely that a single element type would be capable of behaving in such a fashion as to perform all three support functions (i.e. reinforcing, retaining and holding).

Support elements have three key characteristics; elasticity, strength and behaviour. The elasticity of a support element can be described as either stiff or soft, its strength can be characterized as high or low (i.e. strong or weak), and its behaviour can be either ductile or brittle. The three load-deformation characteristics of support elements are plotted in Figure 1-3.
The necessary material characteristics (elasticity, strength and behaviour) required to perform any one of the support functions are typically ineffective and in essence detrimental to achieving any one of the other support roles. For example, strong and stiff support such as fully grouted rebar provides an excellent reinforcing capability but these characteristics become a disadvantage when the support needs to accommodate the violent ejection of rock. Table 1-1 (from the CRRP) shows the support functions matched against the key characteristics of the support elements.

**Table 1-1: Functions and key characteristics of support elements (CRRP, 1995)**

<table>
<thead>
<tr>
<th></th>
<th>Reinforce</th>
<th>Hold</th>
<th>Retain</th>
</tr>
</thead>
<tbody>
<tr>
<td>Stiff</td>
<td>grouted rebar</td>
<td>grouted rebar</td>
<td>shotcrete arch or ring</td>
</tr>
<tr>
<td>Soft</td>
<td>-</td>
<td>long mechanical bolts</td>
<td>chain link mesh</td>
</tr>
<tr>
<td>Strong</td>
<td>cablebolts</td>
<td>cablebolts</td>
<td>Mesh-reinforced shotcrete</td>
</tr>
<tr>
<td>Weak</td>
<td>thin rebar</td>
<td>Split set bolts</td>
<td>#9 gauge mesh</td>
</tr>
<tr>
<td>Brittle</td>
<td>grouted rebar</td>
<td>grouted rebar</td>
<td>plain shotcrete</td>
</tr>
<tr>
<td>Ductile</td>
<td>conebolts</td>
<td>yielding Swellex bolts</td>
<td>chain link mesh; lacing</td>
</tr>
</tbody>
</table>

It is clearly accepted that during a seismic event a number of processes occur nearly simultaneously. Initially, the rockmass immediately adjacent to the surface
of the excavation is fractured by the strain wave generated from the source of the seismic event. At this stage, the support characteristics of those elements dedicated to reinforcing and holding, such as dowels and conebolts, are engaged. It is through the reinforcing elements that the rockmass is strengthened and the self-supporting mechanisms are enhanced. Reinforcement of the rock raises the trigger limit of the excavation to rockburst damage.

Once (and if) the rockmass has been fractured, the next process to consider is the displacement of the fractured rock. The detachment of any broken rock can be passive, in which case the energy of the event is just sufficient to fracture and displace the rock. Or, the displacement can be violent whereby the available seismic energy causes rapid ejection of the fractured material around the walls of the excavation. In either instance, surface applied support mechanisms offer effectively no resistance to the fracturing of the rockmass, but do affect whether fractured material is retained and held at the excavation surface or not. As such, the retaining capabilities of support materials become an important consideration when designing for anticipated dynamic load.

Support elements dedicated to retention, such as screen or welded wire mesh, have traditionally only been expected to perform a safety function, preventing injury or damage by containing loose or detached rock. Any potential benefit from its role as providing some confinement to the walls has been regarded as secondary. However, under high stress conditions mesh can stop the progressive failure processes (due to displacement of fractured rock) that lead to unraveling. In this role, full areal coverage of the retaining element becomes increasingly important as rockburst severity increases.

In focusing on the retaining element, the CRRP states that although the retaining elements offer very little strength or load bearing capacity, they demonstrate an increased ability to accommodate large displacements and can absorb energy much more effectively than the reinforcing or holding elements. In the previous section, the objective of a ground support system under dynamic loading
conditions was described as the ability to control the three damage mechanisms (bulking, rockfalls and ejection of rock). It is through the support system's ability to accommodate large displacements and its capacity to absorb increased energy, both of which are characteristics unique to retaining elements, that the damage created in the event of a rockburst is controlled.

1.1.3.3 **Dynamic Loading and Thin Spray-On Liners**

It is proposed that thin spray-on liners may offer a support advantage as a retaining element under dynamic loading conditions. As discussed in Section 1.1.3.1, a support system for dynamic loading conditions must be capable of (1) surviving the displacement and (2) remaining functional. It is through the holding and retaining functions, as discussed in the previous section, that a support system demonstrates the ability to survive displacement and remain functional.

In terms of these two key requirements, TSLs demonstrate ideal support characteristics. Indeed, the highly elastic nature of a TSL generates long-term residual strength capacity, allowing the thin liner system to "survive the displacement". However, the high tensile strength and energy absorption capabilities of a TSL contribute to the continued holding and retaining functions of the system once the dynamic event has occurred. Furthermore, the support characteristics that have been demonstrated through laboratory testing and some underground trials indicate that TSLs will play an important role as a support tool for seismically active ground. Section 1.1.4 will provide a review of the precursory research that has led to the focus of this thesis on thin spray-on liner support elements under dynamic loading conditions.

1.1.4 **TSLs under Dynamic Loading Conditions**

The application of TSLs as a support tool for seismically active conditions is the prime focus of this thesis. As mentioned in the previous section, the high tensile strength and energy absorption capabilities of TSLs offer a potential support advantage under dynamic loading conditions, as compared to traditional support tools. In the last 15 years, researchers have begun to recognize the effects of
dynamic loading on ground support and have initiated test trials to measure support performance under such conditions. This section will offer a brief review of the research conducted into TSLs and dynamic loading.

1.1.4.1 Laboratory Testing of TSLs under Dynamic Loading

There are no standardized testing procedures for TSLs from which researchers can collect applicable data and high quality results. A number of acceptable testing procedures have been adopted by the mining industry to quantify tensile strength, adhesive strength and static load capacity for design purposes (Tannant et al., 1999). However there have been very few attempts to measure and test the dynamic load capacity of TSLs.

Full evaluation of liner performance under dynamic loading conditions is required before liner use in underground environments can be fully endorsed. To date, there is only limited experience with spray-on liners in bursting conditions (Mineguard™ survived a small rockburst, < 1 Mn at Lower Coleman Mine). One area of potential application is the stabilization of post pillars. A tough flexible liner should be able to contain and reduce the damage and violence resulting from potential pillar bursts (Espley et al., 1999).

Archibald et al. (2000) completed a series of laboratory tests that established the significant reinforcement potential of TSLs in controlling pre- and post-yield rock failure. The tests attempted to reproduce (under laboratory conditions) the situation described by Espley et al. (1999), where post pillar confinement in a highly stressed environment is the potential application. In a wide range of side-by-side failure tests, using highly homogeneous rock and brittle concrete materials, fully unconfined (uncoated) and passively confined (TSL coated) samples were subjected to substantial axial strain, at high rates, sufficient to propagate violent failure of the core specimens; similar to what might be experienced in a rockburst. For fully uncoated specimens, catastrophic failure was induced through violent formation of axial and shear fractures. TSL coated samples, while demonstrating similar shear failure effects, exhibited significantly
less severe damage response. In fact, where passive TSL coatings were applied, controlled and significant post-yield progression was systematically achieved in all tests (Archibald et al., 2000).

Archibald at al. (2000) reports from his testing that in every instance where TSL coating applications were made onto rock specimens, total loss of residual strength was prevented from occurring. In comparison, for all core specimens, tested unconfined and without placement of passive spray-on coatings, no residual strength capacity was manifested. In terms of each characteristic rock mechanic parameter measured, significant positive benefit, at least at a laboratory scale, was demonstrated to occur in samples subject to passive confinement by varying layer thickness of TSLs. Furthermore, the tenacious adhesion properties of these liner materials, in combination with their considerable tensile strength and capacities for elongation allowed for the stable, frictional yielding of even brittle rock specimens.

Similar to the laboratory research conducted by Archibald et al. (2000) using the core samples, the Mines Research Department at Inco Limited also conducted laboratory scale dynamic load testing (Moreau, 2001). Rather than applying an axial load onto the cores to induce a dynamic loading response, blasting techniques were applied to each core sample. A length of detonating cord running through the centre axis of the core, when blasted, simulated the dynamic load. Measurements for each specimen included the resultant axial strain, which was used to determine radial deformation.

Results from the testing program were comparative in nature and indicated that thin spray-on liners (when used alone) performed better as compared to plain shotcrete and uncoated cores when considering support capacity in response to dynamic loading. The metric used to compare samples in the Inco testing program was radial deformation, therefore the samples that demonstrated increased radial deformation either before the sample failed or without sample failure were considered to have better performance. By comparing the resultant
radial deformation of test specimens, inference to the elongation properties of the various support types (including an unsupported sample) were derived. As stated earlier, samples coated with TSLs demonstrated increased radial deformation as compared to the shotcrete specimens and the unconfined specimens, which reinforce the findings of Archibald et al. (2000), thereby indicating the potential support advantage of thin spray-on liners under dynamic loading conditions.

Presentation of the TSL laboratory data to the mining industry, with the goal of implementation, often meets operator objections. Often the question is on the validity of characterization tests on TSL materials, focusing on differences in testing scale between laboratory and underground trials. It is known that laboratory trials, due to limitations in testing equipment capacity or to regulatory standards, often work only with small scale samples, having dimensions in the order of centimeters to tens of centimeters. Field test applications are often at the scale of several meters. This discrepancy has been noted and attempts at large scale field trials have been conducted; of which this thesis is an example. The following section details large scale field testing where the support performance of TSLs under dynamic loading conditions was investigated.

1.1.4.2 Field Trials: TSLs under Dynamic Loading Conditions

Through support of the Workplace Safety and Insurance Board of Ontario, a research project to characterize the support capabilities of conventional and innovative spray-on linings (TSLs) for mitigating dynamic failure effects created by simulated rockbursts was undertaken (Archibald et al., 2003). Blasting trials were conducted in which surface rock reinforcement was provided by two general types of materials, these being:

Conventional support materials

- Rockbolts only
- Rockbolts and welded steel mesh
- Rockbolts and shotcrete
• Rockbolts, steel mesh and shotcrete
• Rockbolts and steel fibre-reinforced shotcrete (SFRS)

Innovative spray-on (TSL) materials

• TSL materials, applied directly onto rock surface in stand-alone fashion
• TSL layers, in combination with rockbolts
• TSL layers, in combination with rockbolts and welded steel mesh

The research was conducted as a large scale field trial, using a wide expanse of visible outcrop surface that was divided into individual trial sites, measuring 4-m by 5-m. Rockburst simulation was imparted onto each trial site separately, using a blasthole located in the middle of each site and loaded with a set quantity of explosive. During the explosive detonation, ground motion measurements were recorded, in the form of peak particle velocity. Post blast observations were made of the local surface rock fracture conditions and the physical condition of the supporting materials (cover agents). This assessment included evaluation of the degree of cover agent fragmentation/intactness (if visible), of the incidence of material tear-through, of the ability of cover agents to maintain tight contact with rock lying within the fracture zone, and of the capability of cover agents to restrict rock fragment travel or expulsion away from the outcrop surface following explosives detonation (simulated rockbursting).

Observations from the test trials indicated that conventional support techniques were not suitable for mitigating rockburst damage. In the site trials where rockbolts alone and rockbolts-and-mesh were tested, induced damage included significant rock fracturing, large scale ejection of rock fragments (in excess of 10 cm diameter), and failure of the support media adjacent to the central blasthole. It was mentioned however that the inclusion of welded wire mesh, as opposed to rockbolts alone, provided improvements in terms of rock retention in response to dynamic loading.
All forms of spray-on surface area support; including shotcrete, SFRS (steel fibre reinforced shotcrete) and TSL, were observed to better mitigate rock damage due to rockbursting than conventional media through reduction in: the extent of zones of fracture observed to result locally about blasthole sites; quantities of loose, ejected rock fragments that were caused to displace away from sites of explosive detonation (i.e. – simulated rockbursting); the number of propagating fractures that were generated through the various support media, indicating that continuous, tight adhesion was maintained between the liner materials and the underlying rock; and the lengths of generated fractures that were created through simulated rockbursting, indicating that the severity of rock damage could also be inhibited by the presence of adhering rock liner materials. Overall, the test results demonstrated that TSL products were equivalent to or better than conventional support methods for providing safe, capable and sustainable support in the event of dynamic rock failure.

In 2001, Inco continued evaluation of TSL products with a series of field trials at its in-house underground research facility (Moreau et al., 2003). The objective of the test program was to assess the support performance of various surface support tools (i.e. shotcrete and TSLs) under simulated dynamic loading conditions. Table 1-2 lists the several different liner types that were tested.

**Table 1-2: 2001 Inco Field Trials - Summary of liner types (Moreau et al., 2003)**

<table>
<thead>
<tr>
<th>Wall #</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>2</td>
<td>Composite liner with 50mm of shotcrete and 4mm of Rockguard</td>
</tr>
<tr>
<td>3</td>
<td>Composite liner with 50mm of shotcrete and 2mm of Rockguard</td>
</tr>
<tr>
<td>5</td>
<td>Shotcrete thickness 50mm with steel fibres</td>
</tr>
<tr>
<td>6</td>
<td>Shotcrete thickness 100mm with mesh reinforcement</td>
</tr>
<tr>
<td>8</td>
<td>Composite liner with 50mm shotcrete and 3mm Masterseal</td>
</tr>
<tr>
<td>9</td>
<td>Shotcrete with thickness of 50mm</td>
</tr>
<tr>
<td>10</td>
<td>Standard bolting and mesh (#6 gauge)</td>
</tr>
</tbody>
</table>
The trial site consisted of drift walls, driven as small cross cuts off a drift, for the installation of the various support systems. The walls were located in a low stress environment, lying at a shallow depth of 27 metres below surface. The support is installed onto the walls of the small bay. The dynamic load is imparted onto the support system with a detonated blast charge (using D90 Pentolite primers alone) in holes parallel to the supported wall. In most cases the test walls were subjected to multiple blasts. With subsequent blasts, additional primers were used until failure of the support system occurred. The general perspective for one of the cut-out bays is shown Figure 1-5.

![Walls to be supported](image)

**Figure 1-4: Photo - Cut-Out Bay in the Inco Test Site (Moreau et al., 2003)**

The dynamic performance of the bonded support systems was assessed through observation of four response variables: (1) level of damage; (2) intensity of the vibration – measured through peak particle velocity; (3) severity of the damage – captured by measuring the volume of rock ejected and; (4) seismic violence – measured through rock ejection velocity.

For assessment purposes, the four types of observations were combined together into a single diagram to facilitate correlation between the various response variables. This ultimately allowed for an estimate of the critical dynamic loading levels corresponding to failure of each support system.
The analysis of the data focused on identification of critical vibration levels for the various support systems. In general, a critical vibration level of 1400mm/s was found to cause some loss of support functionality in all support systems, however the data exhibited rock fracturing and possible rock ejection (depending of the rockmass quality) at vibrations levels ranging from 500mm/s to 3000mm/s.

The analysis also examined the following relationships: vibration level - ejection velocity and vibration level – rockburst severity. The relationship between vibration level and ejection velocity followed the expected trend of increasing ejection velocity with increasing vibration level, regardless of the support system used. In contrast, the type of wall support installed appeared to have an effect on vibration level and corresponding rockburst severity. Shotcrete-based support systems withstood approximately 50% greater vibration levels (~1500mm/s) before failure, however they failed in a more violent manner and displaced larger quantities of broken material as compared to the thin spray-on liners or bolts and mesh.

A summary of the peak vibrations levels, volume ejected and the level of damage for the various support systems is presented in Table 1-3.

<table>
<thead>
<tr>
<th>Wall Number</th>
<th>Support System Description</th>
<th>Peak Vibration Level</th>
<th>Volume Ejected m$^3$</th>
<th>Volume Retained m$^3$</th>
<th>Depth of volume ejected mm</th>
<th>Description of damage</th>
</tr>
</thead>
<tbody>
<tr>
<td>2</td>
<td>Shotcrete and Rockguard</td>
<td>2840mm/s</td>
<td>0.418</td>
<td>0.122</td>
<td>400mm</td>
<td>Moderate</td>
</tr>
<tr>
<td>3</td>
<td>Shotcrete and Rockguard</td>
<td>1625mm/s</td>
<td>1.838</td>
<td>0.020</td>
<td>1100mm</td>
<td>Major</td>
</tr>
<tr>
<td>5</td>
<td>Shotcrete with steel fibres</td>
<td>2690mm/s</td>
<td>5.005</td>
<td>0.003</td>
<td>1100mm</td>
<td>Major</td>
</tr>
<tr>
<td>6</td>
<td>Shotcrete with mesh</td>
<td>1980mm/s</td>
<td>0.699</td>
<td>1.105</td>
<td>600mm</td>
<td>Moderate</td>
</tr>
<tr>
<td>8</td>
<td>Shotcrete with Masterseal</td>
<td>2540mm/s</td>
<td>4.919</td>
<td>0.124</td>
<td>1400mm</td>
<td>Major</td>
</tr>
<tr>
<td>9</td>
<td>Shotcrete</td>
<td>2290mm/s</td>
<td>1.428</td>
<td>0.140</td>
<td>900mm</td>
<td>Major</td>
</tr>
<tr>
<td>10</td>
<td>Bolts &amp; Mesh</td>
<td>2850mm/s</td>
<td>0.627</td>
<td>0.271</td>
<td>400mm</td>
<td>Moderate</td>
</tr>
</tbody>
</table>
Outcomes of the 2001 Inco Dynamic Loading Field Trials were positive. As a result of the research, critical vibration levels corresponding to the onset of damage were identified (thin spray-on liners: 800-2000mm/s; composite liners: 1200-2500mm/s). The discovery of this information has lead to key advancements in the design of support for burst-prone areas. One particular advancement is the development of a design graph (Figure 1-5) identifying the stable zone and unstable zone for shotcrete support of any given magnitude and distance. As this approach is further developed, accuracy can be increased, as well, new graphs with new data can be created readily.

![Figure 1-5: 2001 Inco Field Trials - Nuttli magnitude as a function of distance for shotcrete support (Moreau et al., 2003)](image)

Figure 1-5 is a preliminary result from the initial field study and accounts for stand-alone shotcrete supported walls only. As such, further identified R&D work includes: refinement of the test methodology; incorporation of support systems as a whole (rather than individual support components) and validation of the design graph prior to using the results for support design. This thesis is in response to those identified recommendations.

### 1.1.5 Operational Considerations

When selecting and designing a ground support system, first and foremost, it is important to examine the technical factors such as anticipated load. However, it is equally important to include the operational considerations into the selection process. For a support element to be selected as a suitable support tool it must
not only meet the technical requirements but must also be economically feasible. Additionally, any support tool selected must also meet stringent health, safety and environmental regulations. The focus of this thesis is the technical capabilities of TSLs as they pertain to dynamic loading yet a brief discussion on the operational considerations is merited.

1.1.5.1 Economic Considerations

In considering support economics, it is important to examine the capital cost investments necessary to implement the support system, the direct operating costs (including materials and labour), and those costs required to perform the activity (such as materials handling and maintenance) (Archibald et al., 2000).

Focusing on capital costs, thin spray on-liner technology is most closely related to shotcrete in terms of initial setup and equipment requirements. It has been estimated that the cost of manual TSL spraying equipment is C$50,000 per unit (Archibald et al., 2000); a cost which is similar to that of a manual shotcreting unit. Prototype remote-controlled liner applications have been estimated at C$250,000 (Zeitz and Manfred, 2000), approximating the estimated cost for remote-controlled shotcreting equipment (C$170,000 + infrastructure). In comparison, a MacLean Bolter™, which allows for semi-automated bolt and screen installation, has a capital cost of C$500,000 based on 1998 quotes. Manual installation of bolts and screen requires several pieces of equipment from scissor-lift trucks to pneumatically driven drills, with cost estimates of C$205,000 and C$3,700 respectively. Currently, it is expected that the ground support process will become more mechanized and remote-controlled equipment will be required. Capital costs for mechanization with both the shotcrete and TSL system are expected to be similar while both will continue to be cheaper alternatives as compared to mechanized bolting and screening units.

With direct costs, it is the cost of the actual materials consumed in supporting the rock and the labour required for installing the support system that must be considered. In comparing the direct material costs between conventional systems
and TSLs, conventional systems are relatively low cost (see Table 1-4). However, the associated labour costs for the conventional systems are relatively high due to the reduced rate of application.

Table 1-4: Summary of Direct Material and Labour Costs (Archibald, 2001)

<table>
<thead>
<tr>
<th>Consideration</th>
<th>Criteria</th>
<th>Non-Reinforced Shotcrete</th>
<th>Reinforced Shotcrete</th>
<th>Bolts-and-Screen</th>
<th>TSLs</th>
</tr>
</thead>
<tbody>
<tr>
<td>Material Cost ($Cdn)</td>
<td>$/m²</td>
<td>$14.00-$32.00</td>
<td>$24.00-$51.00</td>
<td>$7.00-$13.00</td>
<td>$21.00-$42.00</td>
</tr>
<tr>
<td>Rate of Application</td>
<td>m²/min</td>
<td>0.14-0.33*</td>
<td>0.12-0.20*</td>
<td>0.11-0.15</td>
<td>1.77-2.32**</td>
</tr>
</tbody>
</table>

* - typical manual rates of application
** - intermittent manual application rate, with approximately 50% down time.

Archibald et al. (2000) examined the effects of labour on direct cost comparisons between the various support systems (bolts and screen, shotcrete and TSLs). They found that when examining support methods based on direct material costs only, bolts and screen support is most competitive. However, the inclusion of labour shifts the total direct costs for each system upwards. The most significant increase in total direct cost is found with the bolts and screen system, due to the number of workers required to install such a system and the slower installation rate. Overall, the inclusion of labour costs shifted support costs upward on the order of $20/m², $10/m², and $1/m² for bolts and screen, shotcrete and TSLs respectively.

The issue of moving support materials from surface to the work headings underground is recognized as a critical element to overall costs and productivity. For most mines, cage time is at a premium; with each trip from surface to underground costing mining companies' significant resources in terms of time and energy. For this reason, the material transportation cost, as it relates to the volume of material required to support the heading, must be factored into the overall economics. Currently, dry mix shotcrete, which typically requires a
considerable number of 50-kg bags, is a burden to the cage system and therefore has a higher material handling cost. In contrast, a substantial number of bolts and screen can be transported in a single run and therefore will have a lower material transportation cost. Likewise, because TSL systems are all thin applications and do not experience significant rebound, only a small volume of material is required to support a comparative size heading and hence TSLs also benefit from a lower material transportation cost.

As an example, Archibald at al. (2000) calculated the materials handling volume for a 2,300 m$^2$ heading. It was estimated that 750 bags of dry shotcrete (or 90 cage runs) would be required to support the area. Using TSLs, the same area could be supported with a 4 mm thick coating by using thirty four 200-litre drums (requiring 4 cage runs). Espley et al. (1999) equated one drum set of TSL (installed in a 4 mm thick layer) to 33 bags of dry shotcrete (installed in a 5 cm thick layer) in terms of relative bulk material requirements. In considering rockbolts, it was stated that such an area would require over 100 wire mesh sheets and approximately 1,040 rockbolts (assuming a bolt spacing of 1.5 m) to provide ground support and therefore would be within the same order of magnitude as TSLs, with respect to cage resources.

Other cost elements to consider include rehabilitation costs. It has been found that TSLs perform better that welded-wire mesh in terms of resistance to routine blast damage and were deemed to have performed similar to mesh in terms of scoop abrasion resistance (Tannant, 1999). Additionally, bolts and screen are susceptible to corrosion. As such, rehabilitation costs for TSLs would be expected to be lower than for bolts and screen support. However, TSL materials require specific storage and handling procedures and, while spray wastes are typically low, there is a greater risk of increased material spoilage and spillage. While dry shotcrete mix can also be lost due to spoilage or spillage, its lower material unit cost makes this a less critical factor (Archibald et al, 2000).
Archibald et al. (2000) also raise the issue of equipment operating and maintenance costs. Operating costs for a typical manual shotcrete system and for a typical semi-automated bolting system are on the order of $5 to $10 and $40 to $50 per operating hour, respectively. Since TSLs tend to be contractor operated, there is no information available for operating and maintenance costs with this support system. However, current TSL spray equipment is not necessarily designed for underground use and is therefore less rugged. (Espley, 1999) Therefore TSL application equipment maintenance costs could be significant until more mining-oriented equipment becomes widely available.

Although the above mentioned costs are important figures to consider when assessing the economics, the rate of advance of the development heading and how the ground support effects the cycle time is an often overlooked economic factor that should be included in any comparison. Archibald et al. (2000) demonstrate that the rate of return for an operation using TSLs is improved along with a drop in the total mining cost by approximately 14%, as compared to the rate of return for the same operation using bolts and screen. This comparison assumes that spray-on support offers the same or improved stability as that provided by bolts and screen. The biggest benefit offered to the mining operation is the speed with which the development advances when spray-on liners are used for support and how much faster the ore zones can be accessed for revenue generation. Certainly, beyond just the direct, indirect and capital cost factors, the time value of money must also be considered in any economic analysis of ground support for the development headings in a mine (Archibald et al. 2000).

1.1.5.2 Productivity

Aside from the face value costs as discussed above, probably one of the more significant operational aspects to consider is time. The influence of time in the development cycle can have a significant economic impact on the overall mining operation; and in the case of ground support, a traditionally long duration activity, the impact is negative. An internal study conducted at Inco Limited's Sudbury Operations demonstrated that ground support installation accounts for the second
highest consumption of time in the development cycle, lagging behind the highest time consumption activity, which is remucking.

The duration of development access to the ore greatly affects the profitability (and even feasibility) of an orebody. In many cases, especially with deeper orebodies, the access development duration is critical. It is in the ground support process that operations have the opportunity to gain some time and hence increase the profitability of an orebody. Traditional support methods, such as bolts and screen, although highly effective support tools, are cumbersome and slow with regards to installation and the process is not readily automated. Shotcrete offers significantly more productivity in terms of application, but is considerably more expensive and requires substantially more material handling (another high time consumption activity). Spray-on liners tend to have high material costs associated with them, but they result in significantly increased productivity and reduced material handling efforts as compared to the other support systems. According to Archibald et al. (2000), spray-on liners, used in conjunction with teleoperation (no manned access at the face) allow for the greatest improvement in development advance rates since a reduced amount of ground support is installed (i.e. with equipment that is more tolerant to mine ground instabilities) with a fast cycle time.

A simulation study conducted by Inco (Espley, 1999), using AutoMOD\textsuperscript{TM}, demonstrates the improved profitability potential of an orebody by employing TSLs to increase access development. The study showed that the mining of a 1.5 million tonne orebody could be accomplished approximately three months faster using TSLs versus conventional bolt and screen support in development and production access heading. The rate of development when using a polymer liner system was improved by 20.2%, while production began 27.8% more rapidly and was completed 12.8% sooner than for conventional bolt and screen support (see Table 1-5).
Table 1-5: Simulation comparison between bolt and screen and TSLs. (Espley, 1999)

<table>
<thead>
<tr>
<th>Aspect</th>
<th>Operating Time (Days)</th>
<th>Difference (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Bolts and Screen</td>
<td>TSLs</td>
</tr>
<tr>
<td>Start of Development</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>End of Development</td>
<td>895</td>
<td>714</td>
</tr>
<tr>
<td>Start of Production</td>
<td>209</td>
<td>151</td>
</tr>
<tr>
<td>End of Production</td>
<td>1132</td>
<td>987</td>
</tr>
<tr>
<td>Total Production Time</td>
<td>923</td>
<td>836</td>
</tr>
</tbody>
</table>

This time savings translated into economic benefits in the form of reduced overhead and direct-mining costs, as well as faster generation of revenues.

1.1.5.3 Environment, Health and Safety

Despite the noticeable gains that may be attributed to the use of thin spray-on liners, the implementation of such support systems within the underground mining industry has not been rapid. The reasons for such are two-fold: there exists a perception that unacceptable occupational health and safety risks develop when such materials are used, and operator resistance has been expressed towards the adoption of thin, flexible liners in place of more robust, historically-proven support methods such as bolts and screen and shotcrete (Archibald et al, 1999).

The recent distribution of laboratory and field support performance data, from a variety of sources, has helped to validate support predictions originally made by TSL manufacturers. In many cases, partial substitution or complete replacement of components of traditional area support elements, such as the screen in bolt and mesh systems, or thin shotcrete layers (while leaving bolts in place), have been shown to provide equivalent or better rock support performance. Only through continued use and application of these materials will eventual industry-wide acceptance of their beneficial support capacities be demonstrated.
Archibald and DeGagne (2000) provided a comprehensive description of the occupational health and safety considerations with respect to thin spray-on liners, from which the majority of the following information is sited.

All current forms of the liquid/liquid polymers used to create TSLs (Mineguard™, Rockguard™, RockWeb™) are typically white or off-white in colour, and designed to be highly reflective. This attribute, in addition to gas diffusion restriction and reduction of airflow frictional resistance potential, offer significant benefits for safety improvement in underground mine environments. With no change in current lighting practice, installation of highly reflective coatings could provide better worker illumination in many worksites. TSL capabilities to both restrict harmful gas inflows (such as methane and radon from rock strata) and optimize flow capacities of ventilation networks will provide additional worker health benefit while reducing mine power costs.

Of additional benefit is the flame retardancy capability built into the physical properties of current TSL support systems. TSL products designated for mine use have been developed to be flame-retardant and non-flammable. According to testing conducted by Archibald, all TSL products tested (as of the year 2000) demonstrate Flame Spread ratings which are less than 75, with some products shown to be significantly below this value. The rating of 75 is a regulated maximum limit which is required by the Canadian building code for construction and use of materials in restricted entry sites, such as prisons or hospitals, where high levels of fire safety and control are required. It is anticipated that similar, if not better, regulatory compliance would be necessary for underground workplaces.

All of the liquid/liquid polymer mixtures require mixing of two component liquids for creation of final, cured solid membranes. All liquid/liquid TSL materials utilize an "A" component consisting of an isocyanate formulation in which monomeric methylene bisphenyl isocyanate (MDI) and an oligomeric/prepolymerized derivative are mixed. The "B" component however is slightly different depending
on the TSL product. For Mineguard™ (a polyurethane) the “B” component consists primarily of various hydroxyl polyols (polyester/polyether hydroxyl resins); whereas the “B” component of Rockguard™ contains primarily amine-terminated polyols (polyetheramines); and the “B” component of RockgWeb™ is a hybrid blend of each (polyurethane and polyetheramines). The spray combination of each of these polymers results in the formation of a rapidly-curing, inert, solid coating. During spraying, reacting solid particles of cured materials can be formed, and smaller particles may become disseminated via air movement throughout downstream air volumes. Such discrete particles are known, however, to be less brittle and coarser than other common airborne dusts, chemical composites and cementitious materials (as released during shotcrete application), which are found in ventilating air streams. At exothermic reaction temperature conditions of 200°C, some of the unreacted MDI may also volatilize (evaporate) and be released into discharge air volumes. The polyol (“B”) components, when inhaled may be absorbed into the lungs of workers and cause minor irritation; on skin contact, no absorption occurs, though some irritation may result.

The main issue with respect to worker exposure to TSLs derives from exposure to the “A” component, which is a designated substance in Ontario. It is known that some 5-20% of exposed workers can become sensitized, and may manifest debilitating or even life-threatening asthmatic symptoms following repeated exposures either by inhalation or possibly skin contact. It is therefore only exposure to the “A” or MDI component of spray-on liners, and largely that fraction that may only exist during, and for short time intervals after, spray release into the workplace that must be rigorously controlled to minimize worker health problems. Once full reaction between the “A” and “B” components occurs, or when unreacted MDI particles come into contact with moisture in the air during downstream travel away from the site of spraying, no additional human health exposure hazard will exist. Complete chemical reaction between components occurs within seconds of spraying, such that no free isocyanate vapour offgassing can develop thereafter.
It is therefore important that adequate health and safety precautions be taken to prevent worker exposure at the spray source and through downstream exposure by unreacted MDI and/or polymer dust. The Code for Respirator Equipment ("Regulation Respecting Isocyanates" – Reg. 842, RRO, 1990) under the Occupational Health and Safety Act stipulates the use of supplied air respirators for the protection of spray applicators. The regulators exposure limit (Ontario) for MDI is 0.005 ppm on an 8-hour time-weighted average (TWA) basis, and 0.02 ppm "in any period of time". Due to limited numbers of in-situ installation tests performed, only limited evidence has been gathered to document the adequacy of current ventilation strategies for limiting worker MDI exposure.

Espley (1999) reported that MDI concentrations decrease rapidly downstream from spray sites where Rockguard™ and Mineguard™ installation trials have been established. In building on this data, the author conducted an environmental monitoring program to assess the quantity of ventilation required to sufficiently dilute the level of airborne isocyanates that are created when spraying RockWeb™. Outcomes of the laboratory test program suggested that ventilation levels of 15,000 to 16,000 cfm would be sufficient to dilute the isocyanate levels to below the regulated TLV (Threshold Limit Value). A copy of the report detailing the Environmental Monitoring Program for RockWeb™ conducted by the author is included in Appendix A: Environmental Monitoring Program.

An extension of the environmental monitoring program continued into the dynamic loading trials (the principal component of this thesis) to confirm the validity of the results during in-situ application. Of the five ground support systems tested, three systems included TSLs. For the purpose of environmental monitoring, each test heading was equipped with four impingers to collect air quality samples during TSL spraying; two impingers were located in the immediate spray area while the remaining two impingers were located downstream (see Appendix B: Schematics – Air Quality Monitoring for schematics). In addition to collecting air quality samples, the quantity of air flow within each heading was measured. Results of the in-situ monitoring program are listed in Table 1-6.
<table>
<thead>
<tr>
<th>Wall ID</th>
<th>Supplied Air Quantity</th>
<th>Level of Airborne Isocyanates*</th>
<th>Spray Site</th>
<th>Downstream</th>
</tr>
</thead>
<tbody>
<tr>
<td>C-3</td>
<td>28,330 cfm</td>
<td>0.03 ppb</td>
<td>&lt;0.02 ppb</td>
<td></td>
</tr>
<tr>
<td>D-4</td>
<td>37,200 cfm</td>
<td>&lt;0.02 ppb</td>
<td>&lt;0.02 ppb</td>
<td></td>
</tr>
<tr>
<td>E-5</td>
<td>12,600 cfm</td>
<td>7.00 ppb</td>
<td>0.65 ppb</td>
<td></td>
</tr>
</tbody>
</table>

* Measurements taken at the face, immediately next to the spraying. Value is a calculated concentration based on if the spraying occurred for just 1 hour during an 8 hour shift.

Comparing the measured isocyanate levels with the TLV regulatory limit, all measured levels are below the regulatory limit of 0.02 ppm in any period of time. When considering the 8-hr TWA regulatory limit (0.005 ppm), the isocyanate level for Wall E-5 is slightly above the limit, with a reading of 0.007 ppm. The most probable cause for the elevated isocyanate level at Wall E-5 is the reduced quantity of air at this site as compared to the other sites; Wall E-5 measured air quantities of 12,600 cfm, while Walls C-3 and D-4 were at 37,200 cfm and 28,330 cfm respectively. These findings are significant, such that they demonstrate the requirement for increased airflow during TSL spraying to ensure worker protection against elevated levels of airborne isocyanates.
1.2 Thesis Objective

The application of a TSL as a support tool for seismically active ground is the focus area of the Dynamic Loading of Support study. Prior to using a liner in a dynamic mining environment, an understanding of the functional dynamic limit (or the maximum critical energy that can be imparted onto a thin liner without compromising its support role) is required. In achieving this understanding, the Dynamic Loading of Support study has been developed with two objectives: (1) to assess the support performance of individual support elements (focusing on thin liner technology) under dynamic loading conditions and (2) identify the functional dynamic limit of the represented support systems.

In determining the functional dynamic limit, there are in reality two distinct limits. When considering support performance, two hypothetical support response categories are possible - damaged support and failed support (not including no response or intact support). As an underground operator, it is advantageous to have available the conditions under which each support response category may occur. For this reason, two limit values are presented for each support system: the dynamic limit for damage and the dynamic limit for failure. With the availability of both dynamic limit values, an underground designer has the option of choosing the support response they are trying to mitigate.

Ideally, mine operators will be able to apply the research contained within this thesis to improve support design for dynamically active headings. Methodologies exist which allow mine planners to predict dynamic energy values imparted onto underground openings, both from nearby blasting activities and from naturally occurring seismic sources. By identifying the dynamic limit of the support system and matching that value with the predicted dynamic energy, the optimum support system can be chosen. This application of research will allow for a more informed support design approach in regards to dynamic loading, resulting in safer and more productive mine headings. A case study is included, demonstrating the application of the results of this research.
Overall, outcomes of the study will allow for a comparison between the functional dynamic limits of the available support tools with the anticipated seismicity levels of an underground excavation, thereby assisting engineers in selecting a suitable support system. More importantly, the results will help determine the suitability of thin spray-on liners for deep mine excavations.
2 TEST PROGRAM

The study was developed as a large-scale in-situ test program at Inco's Research Mine, hereinafter referred to as the 175 Orebody. Located in Sudbury, Ontario, the 175 Orebody is a small low-grade nickel mine that originally closed in the 1970's due to low nickel prices. However, in the mid-1990's the Mines Research Department re-opened the mine for testing of Mining Automation Equipment, as part of an Automation Program at INCO. Located relatively close to surface (~50-m), with ramp access and no production currently on-going, the 175 Orebody was an obvious choice for the Dynamic Loading of Support site location.

2.1 Site Geology and Rockmass Characteristics of the 175 Orebody

The 175 Orebody is an extension below a pre-existing surface pit from about 200 feet to 400 feet below the surface, residing at an elevation of 10,000 to 10,860 feet. At the 200-foot level, the orebody has a strike length of 500 feet but tends to decrease in size as depth increases. Averaging 25 feet in thickness, the orebody dips 60 to 70 degrees to the west.

Geologically, the 175 Orebody emerges at the norite-amphibolite (greenstone) contact. Occurring as a sulphide enriched area of norite, the orebody consists predominately of two ore types: disseminated sulphide, and stringer sulphide. The orebody averages about 25% sulphides. Constraints of available sites within the Research Mine resulted in test walls located sporadically throughout the facility; hence not all test walls are composed of the same rock type. Three of the test walls are located in the footwall (amphibolite) while two test walls were located within the massive sulphide orebody itself.
A geotechnical assessment at the 175 OB was completed as part of a previous research project (Stochmal, 1999). A summary of the joint set data for the 175 OB is given in Table 2-1.

Table 2-1: Summary of joint sets for 175 OB

<table>
<thead>
<tr>
<th>Joint Set</th>
<th>Dip</th>
<th>Dip Direction</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>7°</td>
<td>268°</td>
</tr>
<tr>
<td>2</td>
<td>86°</td>
<td>154°</td>
</tr>
<tr>
<td>3</td>
<td>82°</td>
<td>297°</td>
</tr>
</tbody>
</table>

Rockmass classification, employing both the Q and RMR systems, is presented in Table 2-2. Mirarco/GRC conducted the rockmass classification as part of an earlier project.

Table 2-2: Rockmass Classification

<table>
<thead>
<tr>
<th>Q Classification</th>
<th>RMR-89 Classification</th>
</tr>
</thead>
<tbody>
<tr>
<td>RQD (%)</td>
<td>90 to 100 (µ=95)</td>
</tr>
<tr>
<td>Jn</td>
<td>12 to 15 (µ=12)</td>
</tr>
<tr>
<td>Jr</td>
<td>1 to 4 (µ=1.5)</td>
</tr>
<tr>
<td>Ja</td>
<td>1 to 3 (µ=2)</td>
</tr>
<tr>
<td>Jw</td>
<td>1</td>
</tr>
<tr>
<td>SRF</td>
<td>2.5</td>
</tr>
<tr>
<td>Range of Q</td>
<td>0.8 – 13.3</td>
</tr>
<tr>
<td>Typical Q</td>
<td>2.4</td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
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<td></td>
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<td></td>
</tr>
</tbody>
</table>
According to the parameters that were determined by Mirarco/GRC, the rockmass specific to these trials is closely to medium jointed (joint spacing of 0.005-m to 0.6-m) and typically has three or more joint sets. Joint surface conditions are considered to be variable with joints ranging from smooth planar or slightly rough, with an aperture of less than 1mm, to very rough discontinuous joints. Joint walls have a range of conditions, from unaltered with surface staining (estimated φ of 25° to 30°) to having a non-softening sandy or silty clay coating (estimated φ of 20° to 25°). The rock is considered to be very strong, with an estimated Unconfined Compressive Strength (UCS) of more than about 200-MPa.

2.2 Site Layout

The test program was spread out over five sites within the 175 Orebody to accommodate the significant amount of wall area and rock mass volume required for the testing. A single test site is characterized as a vertical wall area with approximate dimensions of 3-m in length by 2.5-m in height (see Figure 2-1a). The purpose of the wall area is to provide a sample drift wall onto which the ground support undergoing investigation is installed. Some additional characteristics of each sample wall include: one end of the wall (toe-end) is always confined by a drift face while the other end of the wall (collar-end) is considered a ‘free face’, contributing one of two free faces that meet in a corner. To further explain this concept, a plan view of a single test site, identifying the toe-end and the collar-end of the test wall, is presented in Figure 2-1b.
To achieve the above stated objective, a test wall design was generated. The design plan called for a range of energy levels from a single event to be imparted onto the test wall, allowing the support performance to vary from fully intact to total failure – See Figure 2-1a. For ease of data collection, the design stated that the failed zone be located at the far end of the wall (or the confined end) and that the intact zone be located at the free end (corner) of the wall.

### 2.3 Simulation of a Dynamic Load

Within this thesis, the effects of dynamic loading were examined from a ground support perspective (i.e. the performance of a support system subjected to a dynamic load). From a mine operations point of view, it is desirable to obtain results that are applicable under seismic (rockbursting) conditions typical of deep underground mines. Hence, a critical component to the thesis project is the development of a dynamic loading mechanism that is reflective of the true seismic loading conditions experienced in underground mine operations.

A key activity in this thesis project will be the simulation of a seismic load, under controlled conditions, onto various support systems. Currently, very little data exists regarding the simulation of a seismic load; however the proposed method
to be implemented in this thesis is the use of blasting techniques. To aid in the
design of the simulated seismic load and justify the use of blasting technique for
simulation, further understanding of the rockbursting phenomenon in mine
operations is required. For this reason, the author embarked on a study, entitled
Rockburst Data Collection and Analysis, with the objective of achieving a
justifiable seismic load simulation technique.

To achieve the above objective, there are three main tasks.

- Identify a relationship between the level of dynamic loading subjected onto
  a support system and the level of damage incurred to that support system.
- Determine the range of seismic conditions (i.e. energy, magnitude,
  distance, frequency) that are problematic.
- Review blasting theory and determine (or ensure) that the blast design will
  result in conditions similar to those identified in the above bullet.

To complete the above points, rockburst information was collected from Inco
Limited’s Creighton Mine. Specifically, the following information was gathered:

- Event Magnitude
- Source Location Coordinates
- Damage Level
- Support System Type
- Damage Location Coordinates

From the data collection, a plot of seismic events in terms of magnitude and
distance (Figure 2-2) could be developed. It was hypothesized that a plot of the
event magnitude versus distance would provide an idea of the dynamic loading
parameters that must be achieved through blasting techniques, in order to
ensure that the level of dynamic load imparted during the testing was
representative of the seismic loading experienced at an underground mine.
Further to the actual event magnitude, it is the amount of dynamic load imparted to the ground support system that is of significance. Wave theory dictates that the dynamic energy generated at the source location is dissipated as it travels through a medium (in this case rock). Also seismic waves can become altered as they encounter and travel through changes in medium (fault zones, jointing, different rock types, etc.). Hence the dynamic energy at the source location is not equal to the dynamic energy at the damage location. For this reason, it was important to quantify the dynamic loading at the damage location as it is this value that is the true dynamic energy imparted onto the ground support system.

In the Canadian Rockburst Research Program, scaling laws for seismic events, like those commonly used to estimate the level of vibration from blasting activities, are presented. From the scaling laws, it is the peak particle velocity (PPV) that is calculated, rather than dynamic energy itself. In calculating dynamic energy, the energy at any time \( t \) is a product of density \( \rho \) and the square of the velocity at time \( t \), \( v(t) \):

\[
E(t) = \frac{1}{2} \rho v^2(t)
\]

Equation 1

while the peak particle velocity is given by
\[
PPV = \max[|v(t)|]
\]

(Note the absolute value sign.) Given Equation 2, the peak particle velocity only represents the energy at one instant of time, the time when the peak occurred during the velocity time history. What may be more representative from the point of view of effect on the rockmass or ground support system is the cumulative energy over the duration of the velocity record \( T \).

\[
E = \frac{1}{2} \rho \int_0^T [v(t)]^2 \, dt
\]

Equation 3

However, it is justifiable to use PPV as an approximation of the energy level.

As mentioned above, scaling laws allow for the calculation of peak particle velocity (PPV) at the location of the ground support system, given the event magnitude (\( M_n \)), source location and damage location. The scaling law used for the Rockburst Data Collection and Analysis is as follows:

\[
PPV = 1000 \times \frac{10^{0.2}}{R}
\]

Equation 4

where PPV is peak particle velocity (mm/s), \( M_n \) is Nuttli magnitude, and \( R \) is the distance (m).

Using a sample set of calculated PPV values (which would represent PPV data at the location of support damage), another interpretation of the rockburst data is formed – see Figure 2-3.
This plot is a histogram of sample data demonstrating the proposed frequency of events in relation to their PPV levels (also referred to as dynamic load levels). The sample data was plotted as two separate series, where one series represents events that have resulted in damage to the ground support and the second series is a plot of events that have not caused damage to the ground support. It was anticipated that a histogram of PPV levels associated with support damage (as presented in Figure 2-3) would allow for a clear determination of the ideal dynamic load level that would be an output of the seismic simulation. Note that the above figures are a plot of random numbers and do not represent the situation at any current underground operation.

Although the ideas and process behind the Rockburst Data Collection and Analysis study seemed to be logical tools for determining the requisite parameters in simulating a seismic load, the results proved otherwise. A disparity was identified when comparing the calculated dynamic load from the histograms (inferred through PPV) with the measured dynamic load from the initial series of Dynamic Loading Trials (see 1.1.4.2 Field Trials: TSLs under Dynamic Loading Conditions). The comparison showed that the calculated dynamic load associated with support damage were all lower (< 625 mm/s) as compared to the measured dynamic load associated with support damage (800 – 2500 mm/s). There are a number of reasons that could be attributed to the
disparity. One consideration is that all of the calculated dynamic load values are
initiated from a far-field dynamic source while all of the measured dynamic load
values are initiated from a near-field dynamic source. A second contributing
factor is the scaling law used to calculate the dynamic load from the Rockburst
Data Collection study. The scaling law is not a perfect solution; it has site-
specific constants, which are not easily determined and require multiple re-
iterations to converge onto the correct values, hence the accuracy of the
calculated dynamic load is questionable. Following every reasonable attempt,
application of the Rockburst Data Collection and Analysis study to the simulation
design were minimal.

For the purpose of designing the simulation, the author chose to rely heavily on
the measured results of the Dynamic Loading Trials I (Section 1.1.4.2). From
these trials, critical vibration levels corresponding to the onset of damage were
identified to be 800-2000 mm/s for thin spray-on liners alone and 1200-2500
mm/s for composite liners. The earlier testing found that at these dynamic load
levels, damage was initiated to the support system. The goal of the simulated
dynamic loading is to initiate damage to the support systems being tested,
therefore it was determined that 2500 mm/s would be a suitable PPV level for the
simulation.

A modeling tool developed by J. Heilig was provided to assist in the design of the
simulated seismic load. The tool uses local rockmass behaviour characteristics
in response to dynamic loading and predicts PPV levels around a set charge.
Figure 2-4 is a picture of the model interface with set parameters.
The purpose of the model is to determine the appropriate blasting parameters for simulating the dynamic load. A forecasted dynamic loading level (which is set at a desired intensity) is represented as a vibration contour level, demonstrated with a dashed line surrounding the blasthole. As the chosen level of dynamic loading to be imparted unto the test walls is 2500 mm/s, the contour level is shown for 2500 mm/s. Flexibility in the model allows the contour level to be set to any desired intensity of dynamic loading, thereby showing a change in the spatial distribution of various loading intensities respectively. For example, a contour level set to 2000 mm/s would be shown as a larger elliptical shape around the blasthole as compared to that shown in Figure 2-4 and given the identical blast parameters. The reason for this change in size is that energy levels drop with increasing distance away from the blasthole. Employing this information with the desired level of dynamic load equal to 2500 mm/s and the requisite test wall

Figure 2-4: Design of Dynamic Load - Model Interface
design (discussed in the Section 2.2 Site Layout), a simulated dynamic load design was achieved.

Within the model, blasting parameter such as primer diameter, angle of the blasthole, burden spacing, hole length and collar length were altered until the forecasted dynamic load contour (2500 mm/s) was imparted onto a half section of the wall. Looking at the contour in the model, the contour crosses at the mid-section of the wall, hence it can be stated that the wall is subjected to 2500 mm/s at the mid-point. Since the 2500 mm/s contour is located just beyond the test wall (into the excavation for the back half of the wall) the assumption can be made that the back section of the wall will be subjected to dynamic loading levels in excess of 2500 mm/s; as this section of wall is closer to the blast source than the 2500 mm/s contour line. Conversely, the 2500 mm/s contour line for the front section of the wall does not reach the test wall area and remains within the rockmass. For this section, it can be assumed that the dynamic loading levels imparted onto the test wall will be less than the 2500 mm/s; as the test wall is located beyond the 2500 mm/s contour line.

This visual observation of the dynamic load contour with respect to the test wall, such that the contour envelopes the back half of test wall but not the front half, indicates that the associated blast parameters used to achieve the model output are the optimum simulation design parameters. The desirable outcome was for only half of the wall to be subjected to the damage-inducing dynamic load, such that an undamaged section of wall would be available following dynamic loading for comparison purposes. Ideally, the wall was to be subjected to a range of loading intensities, where one end of the wall would be imparted with minimal dynamic load intensity while the opposite end of the wall would be imparted with the maximum dynamic load intensity. Within the two extremes, a range of intensities and support system performances would ensue along the length of the wall for analysis purposes, as discussed in Section 2.2 Site Layout.
2.3.1 Execution of Blast Design

The simulated dynamic load was imparted onto each test wall separately by detonating a set quantity of explosive, as per the design, loaded into a single blasthole behind the test wall. As discussed earlier, the dynamic load was engineered to ensure that test wall failure occurred at the far end of the wall while the support remained intact and undamaged at the free end of the wall, resulting in a transition zone somewhere in the middle. To achieve the desired range of support response, the blast was designed such that a sufficiently high load intensity (enough to incur failure of the supported wall) was imparted at the far end of the wall while simultaneously imparting a low dynamic load intensity at the free end (corner) of the wall.

It is known that the burden spacing effects blast energy level; hence variation in the load intensity was built into the design by increasing the burden spacing along the length of the test wall. The blasthole orientation was chosen such that the toe at the far end of the test wall had minimal burden (~0.2-m) with the intent to completely fail the support and the wall in that area. The corner end of the test wall, at which the blasthole collar was located, had an increased burden distance of approximately 1.2-m to ensure that the collar end of the test wall remained intact. Figure 2-1b illustrates a plan view of the test site showing the blasthole orientation with respect to the test wall (where the dotted line denotes the damage area).

In a realistic underground situation, dynamic loading of ground support often occurs from one of two sources: a seismic event caused by stress redistribution or a reaction event in response to mine blasting. For the purposes of this test program, the goal is to simulate true underground dynamic loading conditions (seismic events, reaction events) using blasting techniques. By employing conventional blasting techniques, damage occurs as a result of two separate mechanisms: (1) damage or crack initiation by the emitted strain wave followed by (2) propagation of the damage or crack and displacement of the rock by the explosive generated gases. In an attempt to simulate 'true' dynamic loading
conditions underground, the effects of blast gases must be minimized from the test program, as neither seismic events nor the form of reaction events demonstrated underground result in damage attributable to blast gas generation. Damage caused by seismic events and or reaction events are the result of the energy contained solely within the strain wave.

The field trials have endeavored to minimize the effect of the explosive gases and better reflect a true underground dynamic event by using cast D90 Pentolite primers. The advantages of cast primers include [Heilig, 2002]:

- A high density and a high velocity of detonation that directs energy towards high shock, more closely approximating the type of energy released in a seismic event;
- A lower quantity of gas generated than produced by other bulk or packaged explosives, restricting the amount of breakage attributable to gas effects;
- A consistent and repeatable detonation, leading to better control over the source energy.
- All blastholes were unstemmed to allow the immediate release of the explosion gases through the collar of the blasthole.

In addition to the variation in blasthole burden spacing, the concentration of explosive energy along the blasthole length also varied. Initially, all primers were placed at the toe of the hole with no spacing in-between, resulting in an explosive column length of 1-m and an unstemmed collar length of ~2.5-m (as per the simulation design). This created a particularly high energy level at the toe end of the test wall while imparting an extremely light charge at the collar end. With this original configuration, localized failure at the toe end of the wall was in excess of the design requirements, while energy levels at the collar end of the wall were too light to impart any significant vibrations. To improve on the blast design, 7-cm spacings in-between the primers were incorporated into the subsequent test
trials. With the improved blast design, the required test wall configuration was successfully achieved.

2.4 Test Program Candidates: Seismic Support Systems

Due to space limitations at the 175 Ore Body, the test program was limited to five test sites and hence only five different ground support systems were included in the field trials. A decision to test total support systems as opposed to individual support elements was taken, based on results from the 2001 Dynamic Loading trials at Inco. Results from the 2001 field trials found lower than desired dynamic capacity values for stand alone liners, hence the possibility of using thin liners as the principle support element in seismic ground was ruled out. Also, the decision to test total systems was based on the fact that a support system consisting of bolts and liners is considered to be more applicable to current support used in seismically-active mining operations.

As one of the goals for the Dynamic Loading of Support project is to assess thin spray-on liners, they were included as elements in three of the five support systems chosen. The remaining systems and elements where chosen based on the support combinations that are most likely to be effective in an underground seismic area (i.e. using bolts and liners that offer resilience in high-deformation and high-energy ground conditions). A survey of the ground support tools currently available on the market found that conebolts, #4 gauge welded wire mesh and steel fibre reinforced shotcrete (wet) contained properties in-line with those necessary for supporting seismically active ground (i.e. ability to survive displacement and remain functional [CRRP, 1995]). Therefore, the support system selections were determined using combinations of the above mentioned support elements in conjunction with thin spray-on liners. Additionally, the current standard of ground support used at Inco in seismically active areas was included in the test program for comparison purposes (“what we have” vs. “what we could have”). The support systems chosen for the test program are listing in Table 2-3.
Table 2-3: Ground Support Systems Tested (listed in order of installation)

<table>
<thead>
<tr>
<th>Wall ID</th>
<th>Wall Area</th>
<th>Support System</th>
</tr>
</thead>
<tbody>
<tr>
<td>A-1</td>
<td>6.4</td>
<td>Conebolts, #4 Gauge Welded Wire Mesh, 50mm Shotcrete</td>
</tr>
<tr>
<td>A-2</td>
<td>6.7</td>
<td>Conebolts, #4 Gauge Welded Wire Mesh</td>
</tr>
<tr>
<td>C-3</td>
<td>6.7</td>
<td>Thin Spray-On Liner, Mechanical Bolts</td>
</tr>
<tr>
<td>D-4</td>
<td>5.5</td>
<td>50mm Shotcrete, Thin Spray-On Liner</td>
</tr>
<tr>
<td>E-5</td>
<td>6.1</td>
<td>Thin Spray-On Liner, Conebolts</td>
</tr>
</tbody>
</table>
3 DATA COLLECTION

The support performance of the ground support systems in response to dynamic loading was assessed through the observation and measurement of four response variables: (1) level of damage; (2) intensity of vibration – measured through peak particle velocity; (3) severity of the damage – captured by measuring the volume of rock ejected and; (4) seismic violence – measured through rock ejection velocity. An examination of the geological structure associated with each test wall was also included as part of the data collection process.

3.1 Level of Damage

The levels of damage suffered to the support system and to the rock wall were gauged via visual assessment. Detailed sketches and digital photographs, taken both before and after wall loading, assisted in accomplishing this task. Integration of the visual damage assessment with the other measured responses is described in Appendix C: Wedge Analysis Output.

3.2 Dynamic Intensity

Dynamic events were imparted by detonating a set quantity of explosive behind each sample wall. It is understood that set quantities of explosive are know to generate varying levels of vibration at the supported wall, depending upon the distance between the explosive and the excavation, the degree of fracturing of the rockmass, and the distance between each of the explosive cartridges within a blasthole. Additionally, the detonation efficiency of the explosives themselves could also affect the vibration levels. To rule out any inconsistency that may arise from these effects, the level of vibration on the support was directly measured using geophones mounted onto the sample wall. Hence, any assessment of support performance is based upon measured vibration levels rather than linked to the quantity of explosive to which the support was subjected.
Through vibration level measurements, or peak particle velocity measurements, the intensity of the dynamic load was captured. Nine single element velocity sensors (geophones) were attached to the support using resin and connected to a high-speed data acquisition system that was configured to simultaneously record the transient level of vibration. The peak level of vibration at each sensor ensuing from the dynamic load was determined and analyzed to produce a vibration contour map of the wall with the level of vibration identified at any point on its surface.

A typical wall indicating the sensor arrangement is shown in Figure 3-1.

![Figure 3-1: Photograph showing the position of vibration sensors](image)

### 3.3 Seismic Severity

The severity of damage from a dynamic event can be linked to the volume of ejected material. Mirarco/GRC calculated wall profiles using a high precision laser survey instrument called the Mensi Scanner. The Mensi Scanner calculates wall profiles by measuring the distance of hundreds of thousands of points on the wall surface before and after the blast. In addition to capturing ejection volume, the wall scans also provided insight into wall bulking. The identification of wall bulking indicates a loosening or potential degradation of the rockmass behind the wall surface yet confirms that the support system is contributing to the stability of the excavation.
Results from the Mensi scans take the form of 3-D models, which represent the sample wall. Onto the 3-D model, colour contours are built-in to visual depict the rock ejection wall area versus bulking rockmass wall area versus the ‘no change in volume’ wall area. An example of the 3-D wall volume model is depicted in Figure 3-2.

![3-D Wall Volume Model (side profile)](image)

**3.4 Seismic Violence**

The ejection velocity of the rock mass adjacent to an excavation is a function of the event intensity and its proximity to the excavation. Large magnitude events located close to any free face will forcefully eject rock at relatively high velocities. Given such loading conditions, effective containment is almost impossible. Therefore, based on the blast design configuration, it is expected that ejection of at least some of the rock mass behind the support would occur. How violent the ejection and how the different support systems respond in containing the rock mass was assessed using velocity gauges.

The velocity gauges chosen for measuring the rock ejection velocity were rotary string potentiometers. Potentiometers use small direct current generating devices to measure ejection velocity. The generator is attached to a spring-loaded spool with thin steel wire wrapped around it. As the wire is pulled off the spool or retracted onto the spool, it turns the generator creating a voltage. The
voltage created is proportional to the rotational speed of the generator and thus the velocity of the wire can be determined from it.

Movement of the sample wall rockmass governs the pulling or retraction of the wire from the spool. The spool is set-up in a stationary location opposite the test wall. The end of the wire is fixed onto a designated point on the test wall. As stipulated in the test design, a total of five string potentiometers were to be installed along the length of the test wall (at the same height) to capture the variation in ejection velocities associated with the differing response categories (intact, damaged, failed). Figure 3-3 demonstrates a typical setup of the velocity gauges.

![Figure 3-3: Ejection Velocity Gauges – Setup](image)

### 3.5 Structural Geology

For the Dynamic Loading of Support field trials, an important component to the analysis is to determine the effects of structural geology on the test results. In this regard, geotechnical data from the five test walls was collected. The data collection process included mapping for structural geology using scanlines as well as taking digital photographs of the rock wall before support was installed to visually capture obvious joint planes.
The presence of structural geology (joints, faults, etc) creates failure planes through which a rockmass is prone to movement. It is found that support failure has a greater chance of occurring along a joint trace as compared to an intact rockmass when submitted to identical dynamic loading conditions. The reason for increased chance of support failure at a joint contact is mainly due to the increased load demand placed on a support system at that point.

The increased load demand is a result of the energy transfer that occurs at joint planes as a wave travels through the rockmass. According to blast wave propagation theory a dynamic energy wave striking a discontinuity (such as a joint) will transmit, reflect and convert some of its energy through the discontinuity. As a result of these processes, any shear strength that may have existed along the joint plane is typically exceeded resulting in a loss of stability. By inducing a loss of stability within the rockmass and transferring energy in the form of dynamic acceleration to the unstable rockmass, an increase in the demand load onto the support system is realized.

Depending on the orientation of the joints, the force of gravity may also be acting in the same direction as the dynamic force, hence increasing the total ejection force of the rockmass even further. The elevated ejection force paired with the existing weakness planes caused by structural geology within the rockmass result in a further increased load demand placed on the support system.

Using the collected data, a geotechnical assessment of each wall was completed. Employing both DIPS™ and the Geomechanical Design Analysis (GDA) software package, ubiquitous wedge analysis calculations were performed. Additional information determined for each wall includes the total number of joints per cubic meter (Jv) and estimated RQD values. The outcomes of the geotechnical analysis, along with the visual level of damage assessments, were used to identify test wall failure mechanisms – described in Section 4 Test Wall Analysis.
4 TEST WALL ANALYSIS

Description of the test wall analysis is divided into five sections, with each section providing the support response and observations for one particular wall / support system. To aid in the analysis, an understanding of the test wall sites within the mine and with respect to the orebody was necessary. Figure 4-1 illustrates the test site locations within the 175 Orebody.

Figure 4-1: Test Wall Locations

4.1 Wall C-3

Wall C-3 is located in the 1003 proposed drift on the 100 level at the 175 Ore Body. This test wall is situated within the footwall of the orebody in which the rocktype is amphibolite. The support system installed on this sample wall consists of thin spray-on liner and mechanical rockbolts.

4.1.1 RockMass Structure

The rockmass quality of Wall C-3 is characterized as good (RMR=70). The three predominant joint sets affecting the wall are listed in Table 4-1. Joint sets
1&2 are described as non-persistent, irregular and planar. Joint set 3 is non-persistent, irregular and undulating.

Table 4-1: Joint Sets for Wall C-3

<table>
<thead>
<tr>
<th>Joint Set</th>
<th>Dip</th>
<th>Dip Direction</th>
</tr>
</thead>
<tbody>
<tr>
<td>Joint Set 1</td>
<td>78</td>
<td>163</td>
</tr>
<tr>
<td>Joint Set 2</td>
<td>68</td>
<td>077</td>
</tr>
<tr>
<td>Joint Set 3</td>
<td>15</td>
<td>347</td>
</tr>
</tbody>
</table>

Analysis of the joint set data in GDA identified a $J_v$ value of 10 (number of joints per cubic meter) and an RQD of 82 for Wall C-3, indicating that the wall was composed of a good quality rockmass.

Joint sets 1 and 2 are sub-vertical joint planes that intersect approximately 0.3-m behind the wall surface, creating a potential wedge. The yellow lines drawn on Figure 4-2(a) highlight the key joint planes that were identified as a potential failure planes prior to loading. It is apparent from Figure 4-2(b) that failure did occur as a result of wedge instability along the identified joint planes.

(a) Wall before loading                          (b) Wall after loading

Figure 4-2: Structurally Controlled Failure – Wall C-3
Dynamic loading of the wall induced failure of the structurally formed wedge. The wedge volume is approximately 0.3-m$^3$, which calculates into half a tonne of rock. This is the amount of material that was ejected from the wall due to the dynamic loading energy.

### 4.1.2 Dynamic Intensity

An interesting outcome from the loading of Wall C-3 is the ability to relate intensity level with wedge mobilization. This relationship defines the critical energy value under which a wedge loses its stability.

Over the span of Wall C-3, a number of similarly shaped wedges are present. As described in the above section, the wedges are approximately 0.3-m$^3$ with a spacing of nearly 15-cm. Keeping in mind that the energy intensity increases over the span of the wall, it is anticipated that wedge instability would occur in areas of high intensity (toe-end of the wall), as opposed to areas of lower intensity (collar-end of the wall).

The dynamic load imparted onto Wall C-3 caused one wedge near the toe-end of the wall to dislodge (See Figure 4-2(b)). In this zone, maximum intensity levels ranged from 2400-mm/s to 3000-mm/s. In comparison, a similarly shaped wedge, located in an area immediately next to the failed zone, maintained stability however the support was slightly damaged. Maximum intensity levels recorded in this zone ranged from 1200-mm/s to 2400-mm/s. At the collar-end of the wall, a third wedge remained stable with no damage incurred onto the support. The maximum intensity level in this zone ranged from 500-mm/s to 2000-mm/s.

These findings indicate that for a blocky rockmass supported with mechanical bolts and a TSL, wedge instability may occur at energy levels in excess of 2400-mm/s.

In terms of overall support performance, dynamic loading of the mechanical bolts and TSL support system resulted in failure zones, damaged zones and intact
zones. This allowed for identification of the dynamic limit for failure and the dynamic limit for damage. The dynamic limit for failure is the minimum peak particle velocity that would cause failure of the support system, whereas the dynamic limit for damage is the minimum peak particle velocity that would cause damage to the support system. It is anticipated that the dynamic limit for damage would be lower than the dynamic limit for failure.

Based on the data analysis, as described in Appendix D: Data Analysis - Sample Calculation, the functional dynamic limits of the mechanical bolts and TSL system are determined. For the mechanical bolts and TSL system, the dynamic limit for failure is 1500 mm/s and the dynamic limit for damage is 1300 mm/s. The confidence level associated with these values is 95%.

In some cases, it is relevant to consider increased dynamic loading intensities and the associated risk to the support system. With the results of the resampled simulation trials, it was possible to correlate a range of potential peak particle velocities with associated failure risk levels. In doing so, the potential for failure at a given peak particle velocity can be identified. Table 4-2 lists varying dynamic load intensities with respect to risk for the mechanical bolts and TSL system.

<table>
<thead>
<tr>
<th>Risk Rating</th>
<th>Dynamic Limit for Failure</th>
</tr>
</thead>
<tbody>
<tr>
<td>5%</td>
<td>1500 mm/s</td>
</tr>
<tr>
<td>50%</td>
<td>2050 mm/s</td>
</tr>
<tr>
<td>95%</td>
<td>2600 mm/s</td>
</tr>
</tbody>
</table>

4.1.3 Seismic Severity

As mentioned in Data Collection, the severity of a dynamic event can be assessed via the volume of ejected material, best quantified by measuring the
zones of broken rock using either cavity scanning or other laser measurement
techniques. For Wall C-3, the Mensi scanner calculated 0.31-m$^3$ of ejected rock,
resulting a 37% void space in the test wall area. Wall C-3 also demonstrated a
bulked volume of 0.04-m$^3$, indicating that the support system was effective in
maintaining stability within that zone (highlighted in Figure 4-3). Figure 4-3 is a
plot of the total wall area, highlighting the resultant ejected and bulked volumes.

![Figure 4-3: Wall C-3 Resultant Ejected and Bulked Volumes](image)

In applying this information to support design, a relationship between vibration
levels and resultant dynamic severity is useful. In correlating the volume
calculations and the vibration contour plots, it was determined that rock ejection
occurred with peak particle velocities ranging from 900-mm/s to 3000-mm/s.
Subsequently, bulking occurred over a wall area in which peak vibrations ranged
from 1200-mm/s to 2400-mm/s. It should be noted that in both dynamic severity
zones, the ejected zone and the bulked zone, potentially unstable wedges of
similar size and orientation were present, as mentioned earlier.

Based on the results, the increased peak vibration levels measured in the
ejection zone (in excess of 2400-mm/s) were sufficient to cause failure of the
supported wedge. In the bulked zone, the results demonstrate that wedge
movement may occur at 2400-mm/s, however the energy is not sufficient to cause the supported wedge to fail. With regards to support design, it is proposed that an upper level of 2400-mm/s be imposed as the dynamic limit for mechanical bolts and TSL when dealing with similarly blocky ground conditions.

Although the opportunity was not available in this test program, it would have been interesting to determine the dynamic load intensity required to cause failure of a similar wedge formation under unsupported conditions. In so doing, a clear understanding of the provisional support capacity offered by the mechanical bolts and TSL could be determined.

4.1.4 Seismic Violence

The testing plan included the measurement of ejection velocities as a means of quantifying seismic violence for each of the walls, as discussed in 3.4 Seismic Violence. Unfortunately, due to technical difficulties, there were no ejection velocity measurements for Wall C-3.

4.2 Wall A-1

Wall A-1 is in the mechanized Cut and Fill area on the 90-m level of the 175 Orebody. This test wall is situated within the ore itself; a zone which is blocky and has a tendency to be more jointed. The support system installed on this sample wall consists of conebolts, #4 gauge wire mesh and 50-mm of dry shotcrete.

4.2.1 RockMass Structure

Structural mapping of Wall A-1 identified three predominant joint sets, which are listed in Table 4-3. All three joint sets are described as persistent, irregular and planar.
Table 4-3: Joint Sets for Wall A-1

<table>
<thead>
<tr>
<th>Joint Set 1</th>
<th>Dip</th>
<th>Dip Direction</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>78</td>
<td>163</td>
</tr>
<tr>
<td>Joint Set 2</td>
<td>68</td>
<td>077</td>
</tr>
<tr>
<td>Joint Set 3</td>
<td>15</td>
<td>347</td>
</tr>
</tbody>
</table>

Analysis of the joint set data in GDA identified a $J_v$ value of 13.7 (number of joints per cubic meter) and an estimated RQD of 71 for Wall A-1. Using Deere’s Rock Quality Designation, the engineering quality of the rock for Wall A-1 is classified as fair. It should be noted that Wall A-1 was found to have the poorest rockmass quality rating in comparison to the other test walls.

Figure 4-4: Structurally Controlled Failure – Wall A-1

The majority of the failure surfaces within Wall A-1 appeared to be along pre-existing joint planes (See Figure 4-4), however it should be noted that the depth of failure is limited to the depth of the blasthole - remnants of the blasthole were visible. All but one of the conebolts remained in the wall following the blast, however in their post blast state the conebolts proved not suitable for providing
support. Fly rock was measured to be thrown a maximum distance of 13-ft, while the dislodged cone bolt was ejected a distance of 23-ft.

This wall suffered the greatest amount of damage/failure, as compared to all of the other test walls, with approximate failure/damage over 75% of the wall area. A number of factors contributed to the unexpected degree of test wall damage. In addition to the structural effects mentioned above (undoubtedly a significant factor), the high amount of failure/damage is also linked to the fact that this wall was the first to be loaded. Based on existing blast data for the 175 OB rock type, analytical calculations determined that 15 primers would achieve the desired test wall design, however the load effects on this particular wall demonstrated that 15 primers provided a dynamic load in excess of the desired energy levels. Hence, this wall was, in a sense, used as a 'trial-and-error' test site in terms of perfecting the loading mechanism. In reaction to the Wall A-1 result, the remaining walls were loaded with 8 primers and included spacing between the primers for two of the test walls. This revised load configuration was successful in achieving the test wall design.

Another factor contributing to the unexpected damage levels is the actual location of the blasthole with respect to the test wall. Survey measurement of the blasthole and the test wall indicate that the blasthole was situated within the top third of the test wall rather than in the center of the test wall, as originally planned, altering the spatial distribution of the dynamic energy waves hitting the wall. Also, the burden spacing at the collar was much shorter than desired (2-cm as compared to 100-cm), meaning the load source was much closer to the test wall than intended. Drilling of the blasthole in the correct location was physically impossible due to structure location at the collaring point and the severe irregularity of the wall itself; the "ideal" corner of the test wall was missing, as it had fallen out as a wedge in the excavation of the test site, leaving an irregular shaped surface for the test. It is quite likely that the increased proximity of the blasthole to the test wall is the reason for the increased failure/damage to the support.
With the source of the dynamic load situated closer to the test wall, it was predicted that higher dynamic load intensities would be imparted onto the test wall and hence result in a greater amount of damage. The results of the analysis, however, do not support this hypothesis. The wall did demonstrate elevated levels of damage / failure, yet the associated dynamic load intensities measured in those areas were not significantly higher, hence contradicting the expected result. Due to the 'trial-and-error' nature of the Wall A-1 test (which was the first to be blasted), the results are not included in performance determination of this particular support system.

4.2.2 Dynamic Intensity

As with Wall C-3 above, the rockmass conditions of Wall A-1 are very blocky and hence the results are most applicable to a blocky rockmass.

Although the majority of the test wall failed under the dynamic loading, there was a zone along the bottom of the test wall, which did not fail. Within this zone, both damaged and intact samples were available to allow for the identification of the dynamic limit for failure and the dynamic limit for damage. Based on the data analysis (completed in the same manner as Wall C-3 and described in Appendix D: Data Analysis - Sample Calculation), the functional dynamic limits of the conebolts, mesh and shotcrete system are determined: the dynamic limit for failure is 1225 mm/s and the dynamic limit for damage is 950 mm/s. The confidence level associated with these values is 95%.

As was done for Wall C-3, the dynamic load intensities with associated risk ratings were determined. This allows for the inclusion of a risk factor in designing a support system for dynamic loading. Table 4-4 lists varying dynamic load intensities with respect to risk for the conebolts, mesh and shotcrete system.
Table 4-4: Dynamic Limit for Failure with Associated Risk Rating

Conebolts, Mesh and Shotcrete

<table>
<thead>
<tr>
<th>Risk Rating</th>
<th>Dynamic Limit for Failure</th>
</tr>
</thead>
<tbody>
<tr>
<td>5%</td>
<td>1225 mm/s</td>
</tr>
<tr>
<td>50%</td>
<td>1275 mm/s</td>
</tr>
<tr>
<td>95%</td>
<td>1350 mm/s</td>
</tr>
</tbody>
</table>

4.2.3 Seismic Severity

For Wall A-1, the Mensi scanner calculated a total of 1.53-m$^3$ of ejected rock, resulting in an 88% void space within the test wall area. Figure 4-5 is a screen capture of the Mensi scan, illustrating the ejected area from Wall A-1. Energy levels measured within this ejected zone range from 1550 – 3150 mm/s. Of the remaining intact wall area, the Mensi scanner did not detect any bulked volume. There are two possible reasons for this – the rockmass behind the support system incurred no damage (no new fractures were generated) and/or the support system was capable of suppressing any joint propagation / dilation that may have occurred in this area. The range of energy levels for the intact/damaged zone are 1000 – 1500 mm/s for the damaged samples and 500 – 1000 mm/s for the intact samples.

Figure 4-5: Wall A-1 Resultant Ejected and Bulked Volumes
As was done for Wall C-3, the correlation between the dynamic load intensities and the ejected volume was examined. For Wall A-1, it was determined that rock ejection occurred with peak particle velocities ranging from 1550-mm/s to 3150-mm/s. Peak particle velocities below 1500-mm/s had no effect on the rockmass or the support system (i.e. no ejection and no bulking).

The distinct transition zone in performance at the 1500-mm/s energy value also happens to spatially coincide with a structural feature. The joint plane highlighted in Figure 4-4 appears to be a significant boundary in support response. Above the joint plane, the wall is failed with significant rock ejection, while below the joint plane, the wall is intact or damage. It is suggested that the zone below the joint plane may have experienced a less severe response due to the constraining effects of the floor. Also, survey measurements of the vertical placement of the blasthole with respect to the wall height show that the blasthole was situated closer to the top of the wall, as mentioned earlier. Hence, due to spatial location of the blasthole with respect to the wall area, the top half of the wall experienced greater dynamic load energy as compared to the bottom half of the wall. The combination of these two factors along with the effects of rockmass structure resulted in a distinct support response boundary for Wall A-1.

4.2.4 Seismic Violence

Seismic violence was to be quantified through measurement of the rock ejection velocity. Like Wall C-3, technical difficulties prevented any accurate measurement of ejection velocity for Wall A-1.

4.3 Wall A-2

Wall A-2 is the second wall in the mechanized Cut and Fill area on the 90-m level of the 175 Orebody. This test wall is situated directly across from Wall A-1 and is also within the ore itself. The support system installed on this sample wall consists of cone bolts and #4 gauge wire mesh.
4.3.1 RockMass Structure

Structural mapping of Wall A-2 identified three predominant joint sets, which are listed in Table 4-5. All three joint sets are described as persistent, irregular and planar.

Table 4-5: Joint Sets for Wall A-2

<table>
<thead>
<tr>
<th>Joint Set</th>
<th>Dip</th>
<th>Dip Direction</th>
</tr>
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<tbody>
<tr>
<td>Joint Set 1</td>
<td>70</td>
<td>242</td>
</tr>
<tr>
<td>Joint Set 2</td>
<td>90</td>
<td>297</td>
</tr>
<tr>
<td>Joint Set 3</td>
<td>75</td>
<td>157</td>
</tr>
</tbody>
</table>

Analysis of the joint set data in GDA identified a $J_v$ value of 6.2 (number of joints per cubic meter) and an estimated RQD of 95 for Wall A-2. Using Deere’s Rock Quality Designation, the engineering quality of the rock for Wall A-2 is good. The favourable orientation of this wall with respect to the structural features of the rockmass is the basis for the high rockmass quality rating. This is in stark contrast to the nearest test wall, Wall A-1, which was found to have the poorest rockmass quality rating of all test sites.

The dynamic load imparted onto Wall A-2 (eight primers with 76-mm spacing between each primer) resulted in failure or damage over approximately two thirds of the wall area (See Figure 4-6).
The majority of the failure surfaces within Wall A-2 appeared to be along existing joint planes, indicating that structure was a dominant factor in wall and support failure.

In observing the support system itself, some elements of the system failed as a result of the blast. At the toe end of the wall, the load had caused the two sheets of screen to separate mid-way up the wall and peel away from the rock surface. In terms of the conebolts, all bolts remained in the wall however the support capacities of the bolts nearest to the toe appear to have very little or no residual support capacity. The collar end of the test wall suffered very little damage, with the support system still providing support capacity in that zone.

4.3.2 Dynamic Intensity

As with Wall C-3 and A-1 above, the rockmass conditions of Wall A-2 are very blocky and hence the results are most applicable to a blocky rockmass.

The outcomes from the loading of Wall A-2 were ideal, delivering the perfect post-blast test wall configuration. The post-blast wall contained samples from each of the required support performance categories (intact, damaged and failed) allowing for the identification of the dynamic limit for failure and the dynamic limit
for damage. Based on the data analysis, the functional dynamic limits of the conebolts and #4 gauge mesh were determined: the dynamic limit for failure is 2125 mm/s and the dynamic limit for damage is 1925 mm/s. The confidence level associated with these values is 95%.

The dynamic intensities with associated risk ratings were determined using the identical procedure as per Wall C-3 and A-1. This allows for the inclusion of a risk factor in designing a support system for dynamic loading. Table 4-6 lists varying energy levels with respect to risk for the conebolts and #4 gauge mesh.

Table 4-6: Dynamic Limit for Failure with Associated Risk Rating

<table>
<thead>
<tr>
<th>Conebolts and #4 Gauge Mesh</th>
<th>Risk Rating</th>
<th>Dynamic Limit for Failure</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>5%</td>
<td>2125 mm/s</td>
</tr>
<tr>
<td></td>
<td>50%</td>
<td>2350 mm/s</td>
</tr>
<tr>
<td></td>
<td>95%</td>
<td>2525 mm/s</td>
</tr>
</tbody>
</table>

4.3.3 Seismic Severity

For Wall A-2, the Mensi scanner calculated a total of 0.50-m$^3$ of ejected rock and a resulting void space equal to 29% of the test wall area. Of the intact wall area, the Mensi scanner captured a bulked volume of 0.24-m$^3$. Figure 4-7 is a screen capture of the Mensi scan, illustrating both the ejected volume and the bulked area for Wall A-2.
Dynamic intensities measured within the ejected zone range from 2200 – 3200 mm/s. In the bulked zone, dynamic intensity measurements ranged from 1750 – 3200 mm/s. Within the bulked zone, the rockmass and support system are damaged but are contained within the wall, and hence may possibly offer some residual support capacity. With this observation, it suggested that the conebolts and mesh can be subjected to dynamic intensities of 1750-mm/s up to 3200-mm/s and still offer some residual support capacity.

In considering the ejected and bulked volumes, the actual bulk volume for this wall is likely smaller than the Mensi scan bulk volume measurement. During post-blast scanning, pieces of the failed (peeled) screen obstructed the true final location of the test wall (See Figure 4-6). The screen obstructions were captured in the scan and were included as the final wall location for determining wall bulking. Hence, the bulk volume includes an air gap between the failed (peeled) screen and the true screen / wall location. For this reason, it is suggested that the actual bulked volume is less than the measured 0.24-m³ from the scan.

Of the remaining intact wall area (approximately 1/3 of the wall area closest to the collar), the Mensi scanner did not detect any bulked volume, indicating that either the rockmass behind the support system incurred no damage (no new fractures were generated) and/or the support system was capable of suppressing
any joint propagation / dilation that may have occurred in this area. The range of
dynamic intensities for this intact / damaged zone is 1600 – 3000 mm/s for the
damaged samples and 400 – 2550 mm/s for the intact samples.

4.3.4 Seismic Violence

Seismic violence was to be quantified through measurement of the rock ejection
velocity, however measuring the ejection velocity proved to be a challenge.
Fortunately, with Wall A-2, some success was realized in collecting ejection
velocities.

Three string potentiometers were installed mid-way up the test wall in three
separate locations. Figure 4-8 is a sketch of the velocity measurement locations
for Wall A-2.

A plot of ejection velocities versus time is presented in Figure 4-9.

Figure 4-8: Sketch of Wall A-2
Ejection Velocity Measurement Locations

Figure 4-9: Velocity vs.
Time Graph for Wall A-2
From the graph it can be seen that Channels 0 and 2 accelerate slower than Channel 4. This can be explained by the fact that the two channels were monitoring ejection velocities near conebolts. The conebolts slow the rock down as it is ejected. Once the bolts fail, velocity measurements become erratic and no further information can be used with confidence. The erratic measurements are likely due to ejected rock hitting the potentiometer's wire and/or the interaction of the piece of ejected rock attached to the end of the wire with other pieces of ejected rock. The maximum ejection velocities before significant disturbance to the signals are 2.8, 3.7 and 7.6 meters per second Table 4-7.

<table>
<thead>
<tr>
<th>Potentiometer</th>
<th>Maximum Velocity (m/s)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Ch. 0</td>
<td>2.8</td>
</tr>
<tr>
<td>Ch. 4</td>
<td>3.7</td>
</tr>
<tr>
<td>Ch. 2</td>
<td>7.6</td>
</tr>
</tbody>
</table>

Though the bolts near channels 0 and 2 had an effect on rockmass acceleration, they did not appear to have any effect on maximum velocity attained. Ejection velocity measurements near channel 2, which did have a conebolt installed nearby, measured a maximum velocity of 7.6 m/s; a value twice as big as the other two velocity measurements. Unfortunately, with such a minimal amount of information available, it is difficult to determine whether the conebolts and mesh are effective at reducing ejection velocities.

### 4.4 Wall D-4

Wall D-4 is situated in the footwall of the orebody on the 90-m level. On this sample wall, a composite support system was tested. The composite system consists of 50-mm plain shotcrete overlain with a thin spray-on liner. In earlier testing (Heilig, 2001), this form of support system was shown to have significant dynamic capacity.
4.4.1 **RockMass Structure**

Structural mapping of Wall D-4 depicts eight structural features that are visible along the test wall. From these structures, three predominant joint sets were identified (See Table 4-8). All three joint sets are described as non-persistent, irregular and planar.

<table>
<thead>
<tr>
<th>Joint Set</th>
<th>Dip</th>
<th>Dip Direction</th>
</tr>
</thead>
<tbody>
<tr>
<td>Joint Set 1</td>
<td>88</td>
<td>102</td>
</tr>
<tr>
<td>Joint Set 2</td>
<td>80</td>
<td>320</td>
</tr>
<tr>
<td>Joint Set 3</td>
<td>60</td>
<td>185</td>
</tr>
</tbody>
</table>

Analysis of the joint set data in GDA identified a $J_v$ value of 6.2 (number of joints per cubic meter) and an estimated RQD of 95 for Wall D-4, indicating a good quality rockmass.

Post blast examination of this test wall indicated that blasting energy played the dominant role in wall / support failure; rockmass structure was not a significant factor. Visual observations of the post-blast test wall clearly identified the half-barrel of the remnant blasthole and failure surfaces appeared to be jagged and “fresh” rather than along existing structural planes.

The effects of the dynamic loading were concentrated at the toe end of the test wall, where approximately 15% of the wall area was categorized as failed (See Figure 4-10).
From this area, a minimal amount of rock was ejected along with some small sections of the composite support. In the majority of the ejected pieces, the shotcrete was separated from the TSL and ejected to the ground on its own, while the TSL remained adhered to the wall. Two pieces of the ejected material, however, remained intact (as a composite system where TSL was still adhered to the shotcrete) with an average size of 46-cm x 20-cm. A preliminary conclusion derived from this outcome is that the TSL can withstand increased dynamic intensity as compared to shotcrete - likely due to the increased tensile strength of the TSL. At intermediate dynamic intensity levels, the shotcrete fails (cracks and detaches from test wall) yet the TSL is capable of remaining intact, due to its high tensile strength. The contrast between the two reactions (TSL maintaining tensile strength while shotcrete experiences loss of tensile strength) likely contributes to the loss of adhesion between the shotcrete and TSL, where the shotcrete ejection forces are greater than the adhesion between the shotcrete and the TSL. To truly benefit from the increased tensile strength of the TSL in a composite system, it would be necessary to increase the adhesive strength between the shotcrete and the TSL such that it would match the tensile strength of the TSL. In doing so, the shotcrete would remain adhered to the TSL even once it has surpassed its tensile strength. Ideally, this increase in adhesive
strength would improve the dynamic limit for the composite system, by preventing rock ejection at increased intensity levels.

When the system is subjected to high dynamic intensities, the composite liner is ejected from the test wall as a whole (with TSL still adhered to the shotcrete). The two sections of composite liner that were ejected from the test wall, with the shotcrete and TSL still adhered to one another, demonstrated this situation. In this case, the intensity levels are sufficient to overcome the tensile strength of both the shotcrete and the TSL; hence the adhesive strength between the two systems is not mobilized. In this case, both the liner and the shotcrete are torn (exceeding the tensile strength), hence the composite system fails as a whole.

In essence, this test wall demonstrates that at intermediate dynamic intensity levels, TSL has greater tensile support capacity, while at high dynamic intensity levels, the composite system fails as a whole (shotcrete and TSL bonded together).

4.4.2 Dynamic Intensity

In contrast to the previously discussed walls, the rockmass conditions of Wall D-4 are relatively intact and massive and hence the results are most applicable to rockmasses of similar condition. For this reason a direct correlation between the results of this wall and the previously discussed walls is not possible – consideration of the rockmass type must be factored into the assessment and comparison.

Effects from blasting produced failed, damaged and intact samples along the test wall, allowing for the identification of the dynamic limit for failure and the dynamic limit for damage. Based on the data analysis, the functional dynamic limits of the composite system are determined as follows: the dynamic limit for failure is 2475 mm/s and the dynamic limit for damage is 2250 mm/s. The confidence level associated with these values is 95%.
As mentioned earlier, the shotcrete and TSL experienced a loss of adhesion during support failure, allowing the shotcrete to be ejected from the wall while the TSL remained secured to the composite system. Observations of this effect were concentrated in the zone classified as damaged. Given the dynamic limit for damage, listed above as 2250 mm/s, it can be predicted that the shotcrete fails at dynamic intensities below 2250 mm/s while the TSL is capable of maintaining some support at identical intensity levels (it continues to maintain tensile strength). Furthermore, the failed zones encompassed the two sections of composite liner, which were ejected as a whole from the test wall. Relating the observations of the failed scenario to the dynamic limit for failure of the composite system, it can be stated that both the shotcrete and the TSL fail when subjected to dynamic intensity levels equal to or greater than 2475 mm/s. This information may be of value in improving composite systems; where benefits can be derived by improving the adhesive strength of shotcrete to match the tensile strength of a TSL. Such an improvement may lead to a superior dynamic loading support system; a system where both components contribute to provide high dynamic limit capabilities.

The dynamic intensity values with associated risk ratings were determined for the composite liner, for the same reasons described in the previous sections. This allows for the inclusion of a risk factor in designing a support system for dynamic loading. Table 4-9 lists varying energy levels with respect to risk for the composite system.

<table>
<thead>
<tr>
<th>Risk Rating</th>
<th>Dynamic Limit for Failure</th>
</tr>
</thead>
<tbody>
<tr>
<td>5%</td>
<td>2475 mm/s</td>
</tr>
<tr>
<td>50%</td>
<td>2950 mm/s</td>
</tr>
<tr>
<td>95%</td>
<td>3100 mm/s</td>
</tr>
</tbody>
</table>
4.4.3 Seismic Severity

Wall D-4 was blasted without spacers between the primers and thus had more charge in the end of the hole as compared to Walls A-2 and C-3. Rock ejection began at the toe of the hole and continued for approximately one meter along the hole. Analysis from the Mensi scanner measured the amount of material ejected from the wall as 0.09 m$^3$. The scan also calculated the void area, which was found to equal 19% of the 5.5 m$^2$ area of the wall. Figure 4-11 is a screen capture of the Mensi scan for Wall D-4.

As was done for the above walls, the correlation between the vibrational energy and the ejected volume was examined. For Wall D-4, it was determined that rock ejection occurred with peak particle velocities ranging from 2600-mm/s to 3200-mm/s.

Of the remaining wall area, including both damaged and intact zones, the Mensi scanner detected 0.01-m$^3$ of bulked volume. It is suggested however that this bulked volume measurement may be inaccurate due to shadowing. The profile of this wall, in particular the blasted void space, was scanned twice due to shadowing from hanging pieces of TSL. The shadows cast by these hanging
pieces rendered the point cloud of the wall incomplete from a single vantage point. Merging of datasets from two vantage points was necessary in an attempt to overcome this problem.

Throughout the remaining intact section of the wall, no bulked volume was detected. For the same reasons as discussed with Wall A-2, the lack of bulked volume is likely attributed to: (1) the rockmass behind the support system incurred no damage (no new fractures were generated) and/or (2) the support system was capable of suppressing any joint propagation / dilation that may have occurred in this area. The range of dynamic intensities for the intact/damaged zone of the composite system is 2000 – 2900 mm/s for the damaged samples and 100 – 2250 mm/s for the intact samples.

4.4.4 Seismic Violence

Seismic violence was to be quantified through measurement of the rock ejection velocity, however measuring the ejection velocity proved to be a challenge. Wall D-4, however, was one of two walls that achieved some success in collecting ejection velocities.

Four string potentiometers were installed mid-way up the test wall in four separate locations. Figure 4-12 is a sketch of the velocity measurement locations for Wall D-4.

![Figure 4-12: Sketch of Wall D-4 Ejection Velocity Measurement Locations]

A plot of ejection velocities versus time is presented in Figure 4-14.
From the graph, two distinct irregularities in the data are identified. Firstly, it can be seen that Channels 1, 2 and 3 recorded instantaneous movement at the time of the blast, however Channel 4 did not record any significant movement until approximately 17 ms after the other three channels. This delayed reaction is likely attributed to either a delayed movement in this portion of the wall as a result of support retention capabilities or a piece of fly rock striking the potentiometer and causing it to come off the wall. The second anomaly highlighted within the data is the large velocity reversals recorded for Channels 1 and 2. It is difficult to speculate as to the cause of these velocity reversals, however it could be the result of a change in trajectory caused by the interaction with other blocks (fly rock). Given the incidence of anomalies identified within the data; confidence in the ejection velocities recorded for this test wall is low. Regardless, the maximum ejection velocities recorded by each of the four potentiometers are presented in Table 4-10.
Table 4-10: Wall D-4 - Recorded Ejection Velocities

<table>
<thead>
<tr>
<th>Potentiometer</th>
<th>Maximum Velocity (m/s)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Ch. 1</td>
<td>1.4</td>
</tr>
<tr>
<td>Ch. 2</td>
<td>2.6</td>
</tr>
<tr>
<td>Ch. 3</td>
<td>2.1</td>
</tr>
<tr>
<td>Ch. 4</td>
<td>3.8</td>
</tr>
</tbody>
</table>

4.5 Wall E-5

Wall E-5 is in the upper portion of the 175 OB, situated within the footwall of the orebody itself on the 50-m level (See Figure 4-1). The support system installed on this sample wall consists of conebolts and TSL.

4.5.1 RockMass Structure

Structural mapping of Wall E-5 identified three predominant joint sets, which are listed in Table 4-11. Two of the joint sets are described as non-persistent, while the third joint set is characterized as persistent.

Table 4-11: Joint Sets for Wall E-5

<table>
<thead>
<tr>
<th>Joint Set</th>
<th>Dip</th>
<th>Dip Direction</th>
</tr>
</thead>
<tbody>
<tr>
<td>Joint Set 1</td>
<td>70</td>
<td>238</td>
</tr>
<tr>
<td>Joint Set 2</td>
<td>77</td>
<td>90</td>
</tr>
<tr>
<td>Joint Set 3</td>
<td>38</td>
<td>115</td>
</tr>
</tbody>
</table>

Analysis of the joint set data in GDA identified a Jv value of 7.7 (number of joints per cubic meter) and an estimated RQD of 90 for Wall E-5. Using Deere’s Rock Quality Designation, the engineering quality of the rock for Wall E-5 is classified as good.

The effects of loading on Wall E5 caused approximately 50% of the wall surface to become either damaged or failed (See Figure 4-14).
Similar to Wall D-4, blasting energy played the dominant role in wall / support failure for Wall E-5; rockmass structure was not a significant factor. Although clear identification of the half-barrel from the remnant blasthole was not possible, failure along “fresh” surfaces rather than along existing planes of weakness was evident. The failure surfaces appeared to be jagged and new rather than along existing structural planes. The ejected rock was relatively small in size (90% of muck pile consisted of pieces measuring less than 5-cm), however there were three larger pieces ejected from the wall. In terms of scatter, there was minimal scatter of the flyrock. The majority of the flyrock was ejected onto the adjacent wall.

The ground support experienced some failure, some damaged sections and a relatively large intact area. All of the conebolts remained in the wall following the blast, however in their post blast state, two of the conebolts appeared to be offering very little support capacity. The thin spray-on liner demonstrated unique support characteristics by continuing to support ejected rock pieces that were no longer attached to the wall. Such rock pieces adhered to the thin liner, which was still attached to the test wall, were prevented from being fully ejected of the
wall. These pieces appeared as “hanging loose” on the test wall and could be identified through the drummy sound they produced during post-blast scaling and also by the stretched appearance of the TSL at the location of the rock failure between the intact rock and the loose piece. Figure 4-14 depicts sections of the TSL still adhered to the test wall.

4.5.2 Dynamic Intensity

As with Wall D-4 above, the rockmass conditions for Wall E-5 are relatively intact and massive, hence the results are most applicable to rockmasses of similar condition. Due to the variation in rockmass type and for reasons discussed in Section 4.4.2, a direct correlation between the results of this wall and Walls A-1, A-2 and C-3 is not possible.

Failure of the test wall and support system was concentrated at the toe end of the wall. Further away from the toe, zones characterized as damaged and intact were available for analysis. Based on the data collected, the functional dynamic limits of the conebolt and TSL system are determined: the dynamic limit for failure is 1600 mm/s and the dynamic limit for damage is 1750 mm/s. The confidence level associated with these values is 95%.

As mentioned in the discussion on Wall C-3, it is anticipated that the dynamic limit for failure would be higher than the dynamic limit for damage of any given support system. With this particular wall, the resulting values do not reflect the expected trend. Under the failed category, samples with intensity values as low as 1550-mm/s were included. However, in the damaged category the lowest dynamic intensity value included in the sample group was 1850-mm/s. Flaws in the results are possible due to the nature of the data collection (visual estimation of failed versus damaged). Additionally, rockmass characteristics may also have contributed to the unexpected outcome. A rockmass with a pre-defined plane of weakness will more readily fail (at a lower intensity value) as compared to an intact rockmass. In this particular case, the damaged zones were likely situated within intact rock and hence it required a greater amount of energy to generate
failure, as compared to the failed zone, which may have been negatively affected by a rockmass structure. As demonstrated with this particular test wall, it is important to be reminded of the potential experimental flaws when interpreting the results.

As was done for the previous walls, the dynamic intensity values with associated risk ratings were determined for the support. This allows for the inclusion of a risk factor in designing a support system for dynamic loading. Table 4-12 lists varying energy levels with respect to risk for the conebolt and thin liner support system.

Table 4-12: Dynamic Limit for Failure with Associated Risk Rating

<table>
<thead>
<tr>
<th>Conebolts and TSL</th>
<th>Risk Rating</th>
<th>Dynamic Limit for Failure</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>5%</td>
<td>1600 mm/s</td>
</tr>
<tr>
<td></td>
<td>50%</td>
<td>1700 mm/s</td>
</tr>
<tr>
<td></td>
<td>95%</td>
<td>2025 mm/s</td>
</tr>
</tbody>
</table>

4.5.3 Seismic Severity

Like Wall D-4, Wall E-5 was blasted without spacers between the primers and thus had a concentrated charge at the toe of the hole, as compared to Walls A-1, A-2 and C-3. With the concentrated charge, the ejection began at the toe of the hole and continued for approximately 1.3-m along the length of the hole. The Mensi scanner calculated a total of 0.43-m$^3$ of ejected rock, resulting in a 49% void space within the test wall area. Figure 4-15 is a screen capture of the Mensi scan, illustrating the ejected area from Wall E-5. Dynamic intensity levels measured within this ejected zone range from 1550 – 3200 mm/s. Of the remaining intact and damaged wall area, the Mensi scanner detected a small bulked volume of 0.01-m$^3$. The ranges of dynamic intensities for the intact/damaged zones are: 1850 – 2600 mm/s for the damaged samples and 600 – 1850 mm/s for the intact samples.
As was done for the previous walls, the correlation between the dynamic intensity and the ejected volume was examined. For Wall E-5, it was determined that rock ejection occurred with peak particle velocities ranging from 1550-mm/s to 3200-mm/s. At peak particle velocities below 2600-mm/s, examples of damaged support without ejection are present, leading to the statement that dynamic intensity levels below 2600-mm/s may result in failure or damage. Intensities below 1550-mm/s had no effect on the rockmass or the support system (i.e. no ejection and no bulking), indicating a "safe" zone below the 1550-mm/s level. This outcome provides an idea of the dynamic performance capabilities for the conebolt and TSL support system.

4.5.4  Seismic Violence

Seismic violence was to be quantified through measurement of the rock ejection velocity. Like Walls A-1 and C-3, technical difficulties prevented any accurate measurement of ejection velocity for Wall E-5.
4.6 Wall Analysis Summary

The following table summarizes the results of the test wall analysis presented within this chapter.

Table 4-13: Wall Analysis Summary

<table>
<thead>
<tr>
<th></th>
<th>Wall C-3</th>
<th>Wall A-1*</th>
<th>Wall A-2</th>
<th>Wall D-4</th>
<th>Wall E-5</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Support Type</strong></td>
<td>TSL &amp; Mechanical Bolts</td>
<td>Conebolts, #4 Gauge Welded Wire Mesh, Shotcrete</td>
<td>Conebolts, #4 Gauge Welded Wire Mesh</td>
<td>Composite Liner (Shotcrete &amp; TSL)</td>
<td>TSL &amp; Conebolt</td>
</tr>
<tr>
<td><strong>Rockmass Type</strong></td>
<td>Blocky</td>
<td>Very Blocky</td>
<td>Blocky</td>
<td>Massive</td>
<td>Massive</td>
</tr>
<tr>
<td><strong>Jv</strong></td>
<td>10.0</td>
<td>13.7</td>
<td>6.2</td>
<td>6.2</td>
<td>7.7</td>
</tr>
<tr>
<td><strong>RQD</strong></td>
<td>82</td>
<td>71</td>
<td>95</td>
<td>95</td>
<td>90</td>
</tr>
<tr>
<td><strong>Dynamic Limit for Failure (5% risk)</strong></td>
<td>1500-mm/s</td>
<td>1225-mm/s</td>
<td>2125-mm/s</td>
<td>2475-mm/s</td>
<td>1600-mm/s</td>
</tr>
<tr>
<td><strong>Dynamic Limit for Damage (5% risk)</strong></td>
<td>1300-mm/s</td>
<td>950-mm/s</td>
<td>1925-mm/s</td>
<td>2250-mm/s</td>
<td>1750-mm/s</td>
</tr>
<tr>
<td><strong>Volume Ejected</strong></td>
<td>0.31-m³</td>
<td>1.53-m³</td>
<td>0.50-m³</td>
<td>0.09-m³</td>
<td>0.43-m³</td>
</tr>
<tr>
<td><strong>Volume Bulked</strong></td>
<td>0.04-m³</td>
<td>0.00-m³</td>
<td>0.24-m³</td>
<td>0.01-m³</td>
<td>0.01-m³</td>
</tr>
<tr>
<td><strong>Average Ejection Velocity</strong></td>
<td>-</td>
<td>-</td>
<td>4.7-m/s</td>
<td>2.5-m/s</td>
<td>-</td>
</tr>
</tbody>
</table>

* 'Trial and Error' test wall – see Section 4.2

In applying the test wall results and deriving support guidelines, it is important to maintain the context of the test program from which the results are based. There
are certain uncontrolled factors inherently present in the test program, which must accompany any support guidelines derived from this research. Specifically, the rock type (amphibolites and massive sulphide) and the rockmass structure should be considered when applying the results to a support design. Secondly, it is important to reinforce that there are a number of differences between each of the test walls themselves; even with the best attempt to have uniformity across the five test sites, hence a direct comparison between each support type (especially through visual assessment alone) is unsuitable. As a reminder to the reader, some of the differences between the walls include:

- Location of blasthole with respect to the test wall;
- Concentration and distribution of explosive energy over length of the blasthole;
- Rock type and rockmass characteristics

Aside from the above-mentioned experimental flaws, valuable data has been attained from the Dynamic Loading of Support test program. With the collected results, additional information is available to assist mine operations in the support design process. Chapter 5 will discuss TSL support design guidelines specific to dynamic loading conditions and will detail a support design approach for dynamic loading based on the results of this test program.
5 TSL SUPPORT DESIGN FOR DYNAMIC LOADING CONDITIONS

Espley (1999) proposed a support design approach for thin spray-on liners in response to wedge-controlled failures and stress-controlled failures. However, the design process for TSL support systems in response to dynamic load failures was not included and does not exist in the research to date. In fact, very little information exists pertaining to the design of any ground support system in response to dynamic load failures. For this reason, the author has proposed the following approach, specifically focusing on dynamic loading and thin spray-on liner support systems.

5.1 TSL Design Methodology: Dynamic Loading Conditions

5.1.1 Step One – Identify the Dynamic Load Source

The first step in the proposed design methodology with respect to dynamic loading is to identify the dynamic loading conditions that will affect the excavation stability. It is important to consider both nearby blasting activities as well as any potential seismic sources that may exist and could impart dynamic loading onto the excavation, such as point source or fault slip events. When assessing nearby blasting activities, information such as the blast energy (in the form of pounds per delay) and distance from the design excavation are required to calculate the dynamic load. Seismic source parameters such as source mechanism (i.e. strain burst, fault slip movement), proximity to design excavation and historical seismicity levels are useful in determining predicted dynamic load conditions.

5.1.2 Step Two – Calculate Dynamic Intensity Load

By using all of the information collected (as described in the first step), predicted dynamic intensity load, in the form of peak particle velocity, can be calculated at the location of the excavation. In the case where blasting activities are the
dynamic load source most likely to affect the excavation, the following scaling law can be used to calculate peak particle velocity (CRRP, 1995)

\[ PPV(\text{in/sec}) = A \left( \frac{D}{\sqrt{w}} \right)^b \]  

Equation 5

where D is the distance between the associated blasting activity and the excavation in feet; and w is the quantity of explosive in pounds per delay. The equation factors A and b are site-specific constants and vary depending on the rockmass characteristics and response to dynamic loading energy.

Alternatively, the dynamic loading from seismic sources may be of greater concern. In these situations, predicted dynamic energy values from seismic sources can be converted to peak particle velocities through the scaling laws. As mentioned earlier however, the scaling law is not a perfect solution; it has site-specific constants, which must be determined using available rockmass response data. The constants may change depending on the available data; hence the accuracy of the calculated energy levels is questionable. If considering a number of sources for dynamic loading, selection of the source expected to generate the highest peak particle velocity should be chosen for design purposes.

5.1.3 Step Three – Demand versus Capacity

The next step is to compare the dynamic limit of the support systems with the calculated dynamic intensity load anticipated to be imparted onto the excavation. It is within this step of the design process where more information on the dynamic limit of ground support products is needed and where the research completed within this thesis is most applicable. For ground support system selection, the dynamic load demand (or intensity) anticipated to be imparted onto the excavation is matched with the dynamic limit of a support system, such that the dynamic limit of the support system exceeds the anticipated dynamic load demand. Typical Factor of Safety (FS) values are to be applied when
determining which support system is most suitable for the anticipated loading conditions, as per the equation below.

\[
FS = \frac{\text{Dynamic Limit of Support System}}{\text{Dynamic Load Demand}}
\]

Equation 6

To further explain the TSL design process (for both static loading conditions and dynamic loading conditions as described above), Section 5.2 provides an example.

5.2 Application in Support Design

A unique proposed application for thin spray-on liners is to support the walls of raises (either bored or driven). In the past, many bored raises were left unsupported only to deteriorate over time (due to block instability, stress changes or seismic activity). It is suggested that a thin spray-on liner could provide the necessary support along the walls to give long-term stability.

At Inco's Copper Cliff North Mine, a full geotechnical assessment on the feasibility of using a thin liner to support a newly developed raise was undertaken (Moreau et. al., 2003). The raise was established as an escapeway (considered a permanent excavation) and therefore the support design must meet a factor of safety equal to 2.0 (Hoek et al., 2000). Components to the support assessment included a structural analysis, stress modeling using Phase2™ and analysis of the expected dynamic loading conditions.

The raise is circular (5-ft diameter) and dips at 78 degrees. The raise is 120-ft in length and is located at a depth of 4050 feet to 4200 feet. The host rock is a meta-sediment and is characterized as a good quality rockmass. There are three main joint sets running through the excavation which had to be considered in the evaluation.

Table 5-1 lists the basic analysis inputs.
Table 5-1: TSL Support Design Analysis (Moreau et al., 2003).

<table>
<thead>
<tr>
<th>Excavation Inputs</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Width</td>
<td>1.52 m</td>
</tr>
<tr>
<td>Height</td>
<td>36.56 m</td>
</tr>
<tr>
<td>Trend</td>
<td>305 °</td>
</tr>
<tr>
<td>Plunge</td>
<td>78°</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Critical Joint Sets</th>
<th>Dip</th>
<th>Dip Direction</th>
<th>Trace Length</th>
<th>Friction Angle</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>83</td>
<td>282</td>
<td>99</td>
<td>30</td>
</tr>
<tr>
<td>2</td>
<td>86</td>
<td>36</td>
<td>99</td>
<td>30</td>
</tr>
<tr>
<td>3</td>
<td>7</td>
<td>342</td>
<td>99</td>
<td>30</td>
</tr>
</tbody>
</table>

5.2.1 Rock Mass Structural Analysis

A structural investigation of the rock mass was completed. Field data was input to the *Geomechanical Design Analysis* (GDA) software in order to assess the size, shape and orientation of wedges around the excavation. One result from the analysis, which assumes a worst-case scenario, is the delineation of a 190-tonne wedge in the hangingwall of the raise, Figure 5-1. Although geometrically unlikely to fail by sliding into the raise, this wedge was used in defining the required thickness of the stand-alone liner support. Note: The GDA software also defined other wedges, more likely to fail by spalling activities; however, these wedges were of small size and geometry and were clearly supportable with a thin application of a thin spray-on liner.
During the site investigation, some recent wedge failure was clearly observed in the raise despite the new age of the excavation. In particular, a new wedge (3.2 tonnes) was dislodged from the hangingwall approximately 20 feet below the collar of the raise. According to the available geological data and discussions with the Ground Control Engineer (F. Malek, personal communication, 2003), the structures responsible for this wedge formation are repeated along the entire length of the hangingwall of the raise.

Upon completion of the data collection, an analytical wedge stability calculation was carried out using both the worst-case scenario (190 tonne wedge calculated from GDA) and the observed condition (3.2 tonne wedge viewed during the site investigation). There were two identified wedge failure mechanisms: (1) rotation (free fall) about its base and (2) sliding.

To maintain wedge stability, the liner had to be capable of holding the wedge in place through liner tension at the wedge perimeter (Espley, 1999). The first step in the calculation is to determine the wedge load demand that will be exerted about the wedge perimeter. Under the observed condition, the wedge load demand was calculated to be 5.5 kN/m. Under the worst-case scenario the wedge load demand was calculated to be as high as 21.8 kN/m.

Figure 5-1: Wedge Analysis Output from GDA Software (Moreau et al., 2003)
The load capacity of any TSL system is a function of its ability to support the load along the wedge perimeter (Espley et al., 2002). Laboratory testing has indicated that a typical TSL may have a load capacity value as low as 1.5 kN/m/mm but can range as high as 3kN/m/mm. For the purposes of comparison in this stability analysis, both values were used in the support design calculation as a worst and best-case estimate.

The equation used in TSL support design (for wedge stability analysis purposes only) is presented below.

\[
FS = \frac{\text{Load Demand}}{\text{Load Capacity} \times \text{Thickness}}
\]

Equation 7

The calculated load demand, the stated load capacity along with the appropriate factor of safety value were used in the equation to calculate the liner thickness that would be required to ensure wedge stability. Table 5-2 lists the support design thicknesses for both the observed wedge condition and the worst-case wedge condition.

<table>
<thead>
<tr>
<th>Table 5-2: TSL Design Thicknesses (Moreau et al., 2003)</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Observed Wedge</strong></td>
</tr>
<tr>
<td>Liner Capacity</td>
</tr>
<tr>
<td>F.S.</td>
</tr>
<tr>
<td>Liner Thickness</td>
</tr>
</tbody>
</table>

The TSL under consideration for the project was noted as having a liner capacity of 3kN/m/mm. With this information, it was concluded that a thickness of 4-mm is sufficient to ensure wedge stability, even under worst-case scenario conditions.
5.2.2 Stress Analysis

The in situ stresses in the area were assessed to determine their potential effects on the TSL supported raise. The analysis was carried out using Phase2™ from the University of Toronto and Inco's Sudbury Basin in situ stress data. The raise was modeled using existing mining influences (current stress state) and with the stress influence of future mining activities. The key results at a section along the raise are shown in Figure 5-2.

![Figure 5-2: Stress envelop surrounding the raise, with current mining and future extraction. (Moreau et al., 2003)](image)

Based on the stress analysis, three main issues were raised:

1. According to the brittle rockmass failure criterion, when stresses surrounding an excavation are greater than \( \frac{1}{2}\sigma_c \) (\( \sigma_c \) = uniaxial compressive strength of the rockmass), rockmass failure is initiated. From the stress analysis, stress levels immediately adjacent to the raise walls were found to be equal to \( \sigma_c \). When a TSL is applied to the raise walls, the liner will aid the rockmass in maintaining integrity and strength. However, the critical stress level at which a TSL supported rockmass begins to fail is unknown. This is possibly an area for further investigation.
The modelling analysis investigated the effects of future mining on the stability of the raise. According to the mine plan, a 280,000-ton block located 100 feet away from the raise will be excavated. This mining will alter the distribution and levels of stress around the raise and, as mentioned above, the effect of the TSL can be assumed to aid in supporting the raise yet the critical stress levels for liner failure needs to be defined.

When the wedge apex height is less than the depth of the stress failure envelope, there is concern for stability in unsupported ground. Measurements taken during the visual site investigation showed an observed wedge apex height of 0.6 metres. From the GDA analysis, the largest possible wedge would have an apex height of 0.9 metres. The depth of influence of the critical in-situ stresses around the raise ranges from 0.1 to 1 metre so there is a possibility that the wedge apex height is less that the depth of the stress failure envelope. In an unsupported raise, it is possible for wedges to fail as a result of the in situ stresses, since the stress failure envelope influence is greater than the wedge apex height. The addition of a thin liner, will aid to prevent this failure, with the thicknesses calculated in the previous section.

Despite some unknown reactions of thin liners to high stress redistributions, it is expected that a TSL has the capacity to support the raise under static conditions.

5.2.3 Dynamic Loading Conditions

Dynamic loading conditions in the raise were identified as a potential problem where it was suspected that the thin liner (as a stand-alone support system) would not have the required capacity. In the time frame between excavation of the raise and conducting the TSL support assessment, some wedges became loose and fell from the walls of the raise and one obvious factor leading to wedge failure was the blasting activity approximately 140 feet from the raise.
For long-term stability, the mine operation required a ground support system that would be capable of withstanding dynamic loading to preserve the integrity of the raise. Historically, seismicity in the area was fairly low; however it was suspected that the long hole stoping activities and associated large scale blasting plans of the mine operation would be the most likely source of any dynamic loading imparted onto the excavation. As per the mining layout, the minimum possible distance between the planned blast locations and the raise is 100 feet and blasts could be loaded up to 1300 lbs/delay (590 kg/delay) in this location.

Once the dynamic load source was determined, the dynamic energy imparted onto the excavation could be calculated. The calculation is described in Section 5.1.2.

Table 5-3: Peak Particle Velocities at the Raise Location (Moreau et al., 2003)).

<table>
<thead>
<tr>
<th>Weight per Delay (lbs)</th>
<th>Distance (ft)</th>
<th>( \frac{D}{\sqrt{w}} )</th>
<th>PPV (in/sec)</th>
<th>Source</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>1300.00</td>
<td>2.7735</td>
<td>19.5504</td>
<td>USBM</td>
</tr>
<tr>
<td>2</td>
<td>1300.00</td>
<td>2.7735</td>
<td>31.2806</td>
<td>DUPONT</td>
</tr>
<tr>
<td>3</td>
<td>1300.00</td>
<td>2.7735</td>
<td>13.2375</td>
<td>Stobie Formula</td>
</tr>
<tr>
<td>4</td>
<td>1300.00</td>
<td>2.7735</td>
<td>27.3382</td>
<td>Rogers (INCO specific)</td>
</tr>
</tbody>
</table>

The results in the above table are computed using different scaling laws, each derived under different site condition (see Section 5.1.2). The predicted peak particle velocity derived from the Inco specific model indicates a dynamic load energy of 27.3 in/sec (693.4 mm/s) in the vicinity of the raise.

According to the results presented in this thesis (see Table 4-13), the dynamic limit for a TSL and conebolt bolt system is 1750-mm/s, which is considerably higher in comparison to the anticipated dynamic loading energy of 693.4 mm/s. Given that the raise is a long term escapeway, a factor of safety of 2.0 is required in the design of the ground support system. Therefore;
\[ FS = \frac{1750 \text{ mm/s}}{693.4 \text{ mm/s}} = 2.5 \]

A comparison between the anticipated dynamic load in the raise and the dynamic limit for failure of a TSL and conebolt system indicates that the TSL/conebolt system is completely capable of maintaining support capacity under the anticipated dynamic loading conditions.

### 5.2.4 Operational Considerations

During the site inspection it was noted that the footwall side of the raise was completely wet. The water was not running down the rock surface, but by touching the rock the observer’s skin became wet. Figure 5-3 illustrates the rock surface condition.

![Footwall Rock Surface Condition of the Raise](image)

Figure 5-3: Footwall Rock Surface Condition of the Raise (Moreau et al., 2003).

Although the water was observed at the collar end of the raise, it was suspected that there would be water conditions along the footwall of the entire 120-ft raise. From past experience, high surface water content contributes to poor adhesion. Operational solutions, such as drainage holes, water shedding mechanisms and possible use of wicking materials were considered and it was concluded that the water was not an insurmountable issue.

### 5.2.5 Outcomes of the geotechnical assessment

The geotechnical assessment concluded that a thin spray-on liner with conebolts is a suitable support product for the raise. In terms of supporting potential wedges, the analysis indicated a thickness of 4-mm would be sufficient. Despite
uncertainty regarding the TSLs reinforcing capabilities under elevated stress levels, any stress induced wedge failures are supportable by the TSL through its high tensile strength capacity. Finally, the predicted dynamic loading levels were calculated to be lower than the ground support dynamic limit values, indicating that with such dynamic loads there was less than a 5% chance of support failure (see Appendix D: Data Analysis - Sample Calculation for details on the statistical interpretations).

Although engineering design proved that a thin liner with conebolts could be used, such a unique application requires the development of a quality control program to ensure such a system remains functional. Due to the long-term requirement for secondary egress via the raise, it was decided not to use a thin liner. The mine opted for use of a culvert to support the raise.

Although this project did not in fact get implemented, the geotechnical assessment was a useful exercise to apply a design strategy for TSL support systems, including the dynamic limit component. To date, there have been very few TSL support designs completed for application in an operating mine and this particular exercise was the first attempt by Inco to include the dynamic loading component. As a result of the exercise, the design process itself has been improved for future applications.
6  CONCLUSION AND RECOMMENDED FUTURE WORK

Based on the research contained within this thesis, thin spray-on liners do offer a number of advantages over traditional ground support tools. Unique support capabilities, such as high tensile strength, improved elasticity and above average dynamic loading capabilities make TSL support systems a viable ground support alternative. In addition to these beneficial support capabilities, operational advantages also exist with these unique systems. Considerations of economic, productivity and health and safety implications further strengthen the rationale for considering thin spray-on liners.

The support performance of TSLs with respect to dynamic loading was studied within this thesis. With the goal of improving TSL support design, the development and implementation of a large-scale in-situ test methodology to quantify the dynamic limit of TSLs was completed. Multiple factors and support performance criteria had to be taken into consideration through the test program methodology; factors such as the level of damage, seismic intensity, seismic severity, seismic violence and rockmass characteristics. As such, a means of measuring and quantifying these various factors was defined. Additionally, to fully examine dynamic load support performance, multiple data types were collected throughout the testing program. For analysis purposes, integration of the various data types and formats into a single meaningful support performance criterion was developed; from which the dynamic limit of the five support systems was identified.

Successfully, the dynamic limit for failure and the dynamic limit for damage of five composite ground support systems were derived. Through comparison of the dynamic limit values, TSLs are shown to have a potential support advantage. Of the five support systems tested, the composite liner consisting of shotcrete and TSL resulted in the highest dynamic limit for failure and damage. This preliminary outcome indicates that a composite system (which includes a TSL)
would be the most suitable ground support system for areas anticipated to be affected by high dynamic loading conditions. Although it is unsuitable to directly compare the performance of the five support systems tested (as indicated by the author herein), it does provide a reasonable approximation to their dynamic loading behaviour and also presents a proposed dynamic limit for failure that can be applied in improving support design.

One of the key accomplishments of this thesis is the presentation of the dynamic limit values for thin spray-on liners (Table 4-13: Wall Analysis Summary). The presentation of this table allows mine operations to more fully understand the support capacity of ground support systems, in particular those with thin spray-on liner included. Traditionally, TSL support design encompassed static loading considerations and potential stress induced loads. However, the author herein proposes the inclusion of dynamic loading as a third consideration to ensure excavation stability against seismic activity. Specifically, a TSL design methodology focusing on dynamic loading conditions is presented. The methodology includes: identifying the dynamic load source; calculating the anticipated dynamic load intensity; and then examining the demand versus the capacity, employing a factor of safety. Although the dynamic loading design methodology presented was specific to TSL, future research may prove the applicability of the design methodology to all ground support systems.

With this thesis, a large scale dynamic loading test methodology is proposed. In reflecting upon the methodology after having completed it, the author is aware of the limitation to the methodology. Consideration of any similar types of testing in the future should incorporate a greater attempt to have both homogeneous and identical types of materials for each of the various test sites. Homogeneous and identical rockmass materials would provide a more consistent basis for comparison. Secondly, there is some skepticism on the ability to properly simulate a dynamic event through the design blasting techniques proposed within. Further research into this aspect of the test methodology is merited in any future works. Additionally, the author has noted within the research some
discrepancies in both the monitoring and the calculating components when examining near-field versus far-field source locations. Any future work in the field of dynamic loading should carefully consider the near-field versus far-field debate, with the goal of quantifying any skew in results which may occur due to such variations.

Although this research work is likely not the final piece in allowing mines to proceed with widespread implementation, it is a step forward in TSL research.
7 REFERENCES


Overview of TSLs. Presents side-by-side test results, operational consideration / comparison with other support techniques, initial dynamic load testing using rock cores.


Comparison of the economic and productivity factors associated with bolt and screen, shotcrete and polymer lining underground rock support.


Description of the WSIB sponsored dynamic loading testing (large scale study) into TSLs, conducted through Queen’s University.


Review of TSL research in Canada up to the year 2000. Includes: comparison of physical attributes between different types of TSL; discussion on support and operational performance; operational and costs benefits; occupational health and safety considerations.

Ground support concept with thin spray-on liners and some laboratory testing results.


Comparison of the economic and productivity factors associated with bolt and screen, shotcrete and polymer lining underground rock support methods. A study of direct / indirect operating and capital costs and production factors, such as installation and set times and material handling issues.


Environmental air quality analysis for MDI (isocyanates) during application of TSL at the 175 OB - for the purposes of the Dynamic Loading of Support II trials.


Review of world-wide support practices in burst-prone ground; full-scale dynamic field tests on support systems; full scale impact tests on shotcrete and mesh; laboratory load-deformation tests on mesh, shotcrete and connections; empirical database of rock damage and support performance under actual rockburst conditions; establish relationship between source magnitude and distance, initial excavation conditions, and predicted support


Description of an iterative algorithm for modeling voussoir arch, as applied to rockmass stability assessments.

An extensive review of thin spray-on liners products for use as ground support in the hardrock mining industry. Research is categorized into the following: technical considerations, operational considerations and support considerations.


An Excel spreadsheet model of development costs (including material and labour) for development at Inco Ltd.


A short article on a similar dynamic loading field study to be conducted through the ACG and the University of Western Australia. Field study is slightly different from the one included in this thesis with regards to the support systems being tested (testing of fresh systems and existing or deteriorated systems).


An Excel spreadsheet model to assist in design of simulated seismic events.


A standard university statistics book. Acted as a reference for (1) data analysis in the Dynamic Loading of Support field trials and (2) data representation in the TSL economic model (time study trends).


Definition and description of rockburst damage classifications (minor, moderate, major)


Highlight important contributions to industry put forth in the Canadian Rockburst Support Handbook.


Provision of specialty user-define blocks for Extend Simulation Software. Inco requested a block to act as a roll-down menu for front-end model users.


Information on Polymers

Mansour Mining Inc. 2003 "MCB Modified Cone Bolt, Mine Mesh Strap, Yielding Rockburst Support". Company Product Literature.

Description, specifications, installation procedures.


Dynamic loading trials – observation and result of seismic severity (volume survey) and seismic violence (rock ejection velocity measurements).


Information and description of the 175 Orebody and the Research Mine

Laboratory testing of thin spray-on liners using coated drill cores and blasting techniques to simulate dynamic loading.


Description of self-supporting mechanism of rock and how ground support can enhance stability.


Information on rock blasting and explosives.


Comparison of the ejection velocities measured for a blast-induced (simulated) dynamic event versus the calculated ejection velocities for a true dynamic event (calculated based on peak particle velocities).


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Appendix A: Environmental Monitoring Program

An environmental monitoring program for isocyanates was included as part of the laboratory testing. The goal of the environmental monitoring was to determine the level of airborne isocyanates that are created when spraying RockWeb. The spraying was performed in a portable garage that was set-up just outside of Bay 22 at North Mine. The cross-sectional area of the garage is 64.8 sq. ft.. Inside the garage, four sampling locations were equipped with impingers for collecting the air quality samples during the spraying. Figure A 1 illustrates the site set-up and the location of the air quality sampling points.

![Figure A 1: Laboratory Test Site](image)

Just prior to spraying, the air velocity was measured at the front of the tent and at the back of the tent. The resultant air volume is calculated below in Table A 1.

<table>
<thead>
<tr>
<th>Measurement</th>
<th>Front</th>
<th>Back</th>
</tr>
</thead>
<tbody>
<tr>
<td>Air Velocity</td>
<td>200 ft/min</td>
<td>40 ft/min</td>
</tr>
<tr>
<td>X-Sectional Area</td>
<td>64.8 sq. ft.</td>
<td>64.8 sq. ft.</td>
</tr>
<tr>
<td>Air Volume</td>
<td>12,960 cfm</td>
<td>2,592 cfm</td>
</tr>
</tbody>
</table>
The impingers were analysed using colourimetric, Marcali method and NIOSH method P&CAM 142. Analysis results are presented in Table A 2.

### Table A 2: Environmental Analysis Results

<table>
<thead>
<tr>
<th>Sample Identification</th>
<th>Air Volume (l)</th>
<th>MDI</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>(mg)</td>
</tr>
<tr>
<td>1A+B</td>
<td>138</td>
<td>.008</td>
</tr>
<tr>
<td>2A+B</td>
<td>138</td>
<td>.021</td>
</tr>
<tr>
<td>3A+B</td>
<td>138</td>
<td>.009</td>
</tr>
<tr>
<td>4A+B</td>
<td>138</td>
<td>.020</td>
</tr>
</tbody>
</table>

Results from the environmental monitoring program show that the measured isocyanate levels for all sample locations in the portable garage were elevated above the regulatory TLV of 0.005 ppm. (See Excerpt from the Ontario Regulation respecting Isocyanates).

The samples 1A+B and 3A+B correspond to the two sample locations immediately adjacent to the open garage door (front of the garage); the other two samples correspond to the impingers that were placed at the back of the garage where the door was closed. Air quality data collected at the front of the garage showed significantly lower isocyanate levels as compared to those levels measured at the back of the garage. The variation in air volume between the front of the garage and the back of the garage is the most likely reason for the difference in isocyanate levels. At the back of the garage, there was insufficient air volume (2,592 cfm) to dilute the isocyanates. In contrast, the increased air volume at the front of the garage (12,960 cfm) was capable of diluting isocyanate levels to just above the TLV.

Brian Keen of Coleman/McCreedy East was consulted on the issues of typical mine air volumes and dilution of isocyanates underground. According to Keen, the standard airflows at Coleman/McCreedy East are in the range of 30,000 –
50,000 cubic feet per minute (cfm), however the worst case scenario could have airflows as low as 10,000 – 12,000 cfm. A comparison of these standard volumes with the airflows measured at the test site indicated that the portable garage creates an unrealistic underground environment. Inside the garage, airflow conditions were less than or equal to the worst-case scenario.

At the back of the garage where the air volume was extremely low (2,592 cfm), the resulting isocyanate levels were quite high, yet by increasing the air volume to 12,960 cfm the isocyanate levels drop to just above the TLV. In discussing this concept with Keen and drawing from his past experience with other isocyanate-bearing products, it was suggested that ventilation levels of 15,000 to 16,000 cfm would be sufficient to dilute the isocyanate levels to below the TLV. A note of interest: these volumes are actually lower than the typical mine air flow and are easily exceed in many underground headings.

Excerpt from the Ontario Regulation Respecting Isocyanates

The Ontario regulations for isocyanates, a designated substance, have set threshold limit values to protect workers from over-exposure. Section 4 subsections 1 and 2 of the Regulation Respecting Isocyanates (Reg. 842 under the Occupational Health and Safety Act) states:

Every employer shall take all necessary measures and procedures by means of engineering controls, work practices and hygiene practices to ensure that time-weighted average exposure of a worker to toluene diisocyanate (TDI), methylene bisphenyl isocyanates (MDI), hexamethylene 1, 6-diisocyanate (HDI) or isophorone diisocyanate (IPDI) is reduced to the lowest practical level and in any case shall not exceed 0.005 parts of the isocyanates per million parts of air by volume or 0.2 micromoles of the isocyanates per cubic metre of air.

Despite subsection (1), an employer shall ensure that the exposure of a worker to toluene (TDI), methylene bisphenyl isocyanates (MDI), hexamethylene 1, 6-diisocyanate (HDI) or isophorone diisocyanate (IPDI) shall not exceed 0.02 parts
of the isocyanates per million parts of air by volume or 0.8 micromoles of the isocyanates per cubic metre of air in any period of time.

According to the regulation, an individual may work in an environment that complies with subsection (1) and (2) without requiring the worker to wear respiratory equipment. However, if conditions exist where the isocyanate levels are above the regulated TLV, it is necessary for all workers in the exposure area to wear respiratory equipment.
Appendix B: Schematics – Air Quality Monitoring

Wall C-3
Ventilation Information – Auxiliary System
Air Velocity = 20.38 m/sec
Duct Diameter (outlet) = .9144 m
Air Quantity = 13.38 m$^3$/sec (28,330 cfm)

Wall D-4
Ventilation Information – main drift
(no auxiliary system for 9001 drift)
Air velocity = 1.05 m/sec
Cross Sectional Area of Roadway = 16.72 m
Air Quantity = 17.6 m$^3$/sec (37,200 cfm)
Wall E-5

Ventilation Information – Auxiliary System

Air velocity = 18.78 m/sec
Duct diameter (outlet) = 0.635 m
Air Quantity = 5.94 m³/sec (12,600 cfm)

86 ft (26 m)
Appendix C: Wedge Analysis Output

Wall A-1

Wedge weight = 11 tonnes
Failed along visible joint planes

Wall A-2
Wall C-3

Wedge weight = 0.3 tonnes

Failed along visible joint planes

J_v = 10
RQD = 82
Wall D-4

Structure did not appear to be a factor

$J_v = 6.2$

$RQD = 90$
Wall E-5

Wedge weight = 9 tonnes

Structure did not appear to be a factor

$J_v = 7.7$

$RQD = 90$
Appendix D: Data Analysis - Sample Calculation

The objective of the analysis is to determine the critical dynamic intensity values for the five ground support systems being tested, as listed in Table 2-3. Identification of critical intensity values requires the creation and examination of all three types of support responses possible in ground support (intact, damaged and failed). Information on support response alone however is not sufficient - the corresponding measured peak particle velocities for each wall are also required. The correlation of both pieces of information - the peak particle velocities and the support response – is the key to determining the critical intensity value for a support system.

Dividing the sample area into small sample sizes facilitates the data correlation. Each wall is segmented into 40 equal sized areas, which are referred to as 'samples'. The sample size of 0.15-m$^2$ is chosen on the basis that 40 such samples will fully encompass the area of the largest wall, Wall C-3. Each subsequent wall will be assessed using an identical sample size, shape, and layout to ensure equivalent sample weighting. An example of the sample layout is illustrated in Figure A 2: Analysis Sample.
The sample identification scheme is also illustrated in Figure A 2: Analysis Sample. For all walls, sample identification begins at the top of the wall on the toe-end (toe-end is always “A” and collar-end is always “H”). The scheme increases alphabetically along the length of the wall and numerically from the back (1) to the floor (5).

In all cases, with the exception of Wall C-3, the outermost samples encompassed more area than was actually defined by the wall, since the overall wall areas were smaller than the total sample area (i.e. sample area always equals 6.7-m$^2$ yet wall area is equal or less than 6.7-m$^2$). Therefore extrapolation was used to incorporate the remaining sample space into the analysis.

**Seismic Intensity Calculations for Wall C-3**

For ease of understanding, the analysis will be described in detail for one wall: Wall C-3 – Mechanical Bolts and Thin Spray-On Liner.

As discussed in the data collection process, a vibration contour plot was generated for Wall C-3. The sample grid was overlain onto the vibration contour
plot, dividing the vibration data into 40 samples. Each of the 40 samples was individually assigned average peak particle velocity values based on the number of contours crossing through that particular sample and the area of influence of any particular contour. The end product is an energy intensity grid with 40 peak particle velocity values, with each value representing a 0.15-m$^2$ area. Figure A 3: Energy Intensity Grid illustrates an example of the vibration contour plot overlain by the energy intensity grid for Wall C-3.

The second component of the analysis integrates the support response. As described in Section 3.1 Level of Damage, the support response is graphically represented in a scaled sketch. Post-blast photos of the wall and the support system are also available to strengthen the evaluation. Based on information from the sketches, photos and knowledge of the sample grid layout, a support response category is assigned to each of the 40 samples. The support response is assigned based on sample characteristics, as listed in Table A 3.
### Table A 3: Support Response Categories

<table>
<thead>
<tr>
<th>Support Response</th>
<th>Sample Characteristics</th>
</tr>
</thead>
</table>
| Intact           | All rock is retained by the support system  
                   | No visible damage to the support system  
                   | Bulking may be visible and support may appear to be stretched but is undamaged |
| Damaged          | Support is broken, cracked, ripped  
                   | Rock is contained by the support  
                   | Rock adhered to the support but not necessarily attached to the wall. |
| Failed           | Support is broken, cracked, ripped  
                   | Rock is ejected from the wall  
                   | Rock is no longer adhered to the support. |

The procedure for assigning support response is as follows:

1. Select sample ID and note sample location (with respect to surveyed wall points) on energy intensity grid.
2. Find and highlight corresponding sample area using surveyed wall points (i.e. point matching)
3. Determine support response as per Table A 3.
A summary table tabulating the sample ID, energy intensity value and corresponding support response classification is generated from the above components. A portion of the summary table for Wall C-3 is provided in Table A 4 for illustrative purposes.

Table A 4: Summary Table – Energy Intensity correlated to Support Response

<table>
<thead>
<tr>
<th>Wall 1</th>
<th>Blast #1</th>
<th>Grid Coord</th>
<th>PPV</th>
<th>Support Response</th>
</tr>
</thead>
<tbody>
<tr>
<td>A1</td>
<td>3200</td>
<td>Intact</td>
<td></td>
<td></td>
</tr>
<tr>
<td>A2</td>
<td>3200</td>
<td>Intact</td>
<td></td>
<td></td>
</tr>
<tr>
<td>A3</td>
<td>2900</td>
<td>Damaged</td>
<td></td>
<td></td>
</tr>
<tr>
<td>H3</td>
<td>2100</td>
<td>Intact</td>
<td></td>
<td></td>
</tr>
<tr>
<td>H4</td>
<td>1200</td>
<td>Intact</td>
<td></td>
<td></td>
</tr>
<tr>
<td>H5</td>
<td>200</td>
<td>Intact</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Statistical Data Analysis for Wall C-3

At this point in the analysis, the samples are grouped into three separate categories based on their support response classification: Intact, Damaged, and Failed. Each category is examined as a statistical population in an attempt to identify the probability distribution that best represents the sample data. For Wall C-3, the outputs of the statistical evaluations are as follows:

<table>
<thead>
<tr>
<th>Intact</th>
<th>Damage</th>
<th>Failed</th>
</tr>
</thead>
<tbody>
<tr>
<td>PPV (mm/s)</td>
<td>PPV (mm/s)</td>
<td>PPV (mm/s)</td>
</tr>
<tr>
<td>Mean</td>
<td>1482</td>
<td>Mean</td>
</tr>
<tr>
<td>St. Dev.</td>
<td>893</td>
<td>St. Dev.</td>
</tr>
<tr>
<td>Sample No.</td>
<td>17</td>
<td>Sample No.</td>
</tr>
</tbody>
</table>

Histogram plots of each category’s data set were created to facilitate identification of the probability distribution that would best represent the data. The histogram plots for each category are presented in Figure A 4: Histogram Plots for Wall C-3.
A visual examination of the histogram plots determined that the sample data did not suitably fit any probability distribution. Hence the raw data alone was used to identify the critical dynamic intensity values or dynamic limits.

As discussed in the objective, the goal of the analysis is to determine the dynamic limit for damage and the dynamic limit for failure. The analysis will focus firstly on identifying the dynamic limit for failure. In identifying the dynamic limit (for either damage or failure), it is important to include the intact sample data into the identification scheme. A plot of the intact sample data and the failed sample data is presented in Figure A 5.
The Fail/No Fail boundary should be somewhere in this interval.

Ideally, there would be a distinct intact / failure boundary within this plot; a boundary value where all the intact data falls to the left of the boundary value and all of the failed data falls to the right of the boundary value. This boundary value would be analogous to the dynamic limit for failure. Realistically however, this boundary value is not readily apparent from the plot. An approximation of such a boundary value can be achieved by averaging the maximum peak particle velocity for the intact samples and the minimum peak particle velocity for the failed samples. This approximation would apply whether there was overlap between the two sample series (as is the case for Wall C-3) or not. Consider a case where there is no overlap – the boundary value would be somewhere between the maximum ppv value for the intact series (where all of the series is located at the far left of the graph) and the minimum ppv value for the failed series (where all of the series is located at the far right of the graph).

For the data sets presented in Figure A 5, the maximum ppv for the intact sample data is 3200 mm/s and the minimum ppv for the failed sample data is 900 mm/s. Applying the averaging technique discussed above, the boundary value is 2050 mm/s. However this boundary value is based on a limited number of sample points. By resampling the data 2000 times, a distribution in the boundary value is achieved. Figure A 6 is a histogram of the resampled data simulation output.
Figure A 6: Dynamic Limit for Failure - Histogram

The histogram in Figure A 6 illustrates the variation in the potential boundary value or dynamic limit for failure. From this range, it is possible to develop a risk rating associated with the dynamic limit for failure. The risk rating is based on the percentile of simulation outputs (boundary values) at incremental peak particle velocity values within the range. For example, only 5% of the simulations (2000 trials) resulted in boundary values below 1500 mm/s. Table A 5 presents the dynamic limits for failure at risk ratings of 5%, 50% and 95%.

Table A 5: Dynamic Limit for Failure with Associated Risk Rating

<table>
<thead>
<tr>
<th>Mechanical Bolts and TSL</th>
<th>Risk Rating</th>
<th>Dynamic Limit for Failure</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>5%</td>
<td>1500 mm/s</td>
</tr>
<tr>
<td></td>
<td>50%</td>
<td>2050 mm/s</td>
</tr>
<tr>
<td></td>
<td>95%</td>
<td>2600 mm/s</td>
</tr>
</tbody>
</table>
Using the identical analysis technique as described above, the dynamic limits for damage at risk ratings of 5%, 50% and 95% are presented in Table A 6.

Table A 6: Dynamic Limit for Damage with Associated Risk Rating

<table>
<thead>
<tr>
<th>Mechanical Bolts and TSL</th>
<th>Risk Rating</th>
<th>Dynamic Limit for Damage</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>5%</td>
<td>1300 mm/s</td>
</tr>
<tr>
<td></td>
<td>50%</td>
<td>1850 mm/s</td>
</tr>
<tr>
<td></td>
<td>95%</td>
<td>2600 mm/s</td>
</tr>
</tbody>
</table>