SILL MAT DESIGN
FOR NARROW VEIN MINING

by

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THE UNIVERSITY OF BRITISH COLUMBIA

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Abstract

The crux of sill mat design is the estimation of vertical stress before sill mat failure. Current theories for estimating vertical pressure in rock and soils at shallow depths all produce a variety of answers. The current theories for estimating vertical stress do not account for changes in stope geometry. For example, the footwall, hangingwall, and ore widths change drastically from lift to lift in some narrow vein gold mines. Consequently, a different approach is suggested. The new approach is based on Terzaghi's (1948) Arching Theory and the first order linear differential equation:

$$d\sigma_v/dz = (\gamma - 2c/w) - (2K\sigma_v \tan(\phi)/w)$$

The solution to the above equation is proposed:

**Marcinyshyn’s Arching Theory Summation Series (MATSS) for Determining Vertical Pressure in Soil/Backfill.**

$$\sigma_v = \left[ (\gamma w_v - 2c_v) / 2K_v \tan(\phi_v) \right] + \left[ 1 / 2 \right] \sum_{i=1}^n \left\{ (\gamma_i w_{v,i} - 2c_{v,i}) / (K_{v,i} \tan(\phi_{v,i})) \right\} e^{-x}$$

where,

- $\chi = 2 \left[ (K_v z_v / w_v) \tan(\phi_v) + (K_{v,i} z_{v,i} / w_{v,i}) \tan(\phi_{v,i}) + ... + (K_i z_i / w_i) \tan(\phi_i) \right]$
- $K_v = \text{coefficient of lateral earth pressure of bottom segment}$
- $w_v = \text{width of bottom segment (cemented backfill, meters)}$
- $c_v = \text{cohesive strength of bottom segment (cemented backfill, Pascals)}$
- $z_v = \text{length of bottom segment (cemented backfill, meters)}$
- $\gamma_v = \text{unit weight of bottom segment (cemented backfill, N)}$
- $\gamma_o = 0$
- $c_o = 0$
- $q_o = 0$
- $z_o = 0$.

The above approach has the flexibility to handle changes in stope geometry, material properties, and different geotechnical environments (i.e. saturated and unsaturated backfill).
The work herein is based on work carried out at the Snip Mine near Stewart, British Columbia. The cable sling support system for sill mats that the Snip Mine currently uses is working well. Cable slings consist of high tensile steel cable installed with split sets that are cemented into boreholes. After reviewing other sill mat designs, the cable sling system should continue to be used at the Snip Mine. A site specific formula derived from MATSS, cable sling properties, stope dimensions, and other site characteristics can be used to determine cable spacing. The procedure outlined in this thesis can be used to design cable sling support for any sill mat in a similar mining environment. For the Snip Mine, the spacing of cable slings is calculated as,

Snip Mine Cable Spacing Formula

\[
\text{Cable Spacing} = (1.66 \times 10^{-4}) H \left[ 1 / (x_{\text{span}})^2 + 1 / (x_{\text{strike}})^2 \right] \text{ meters.}
\]

where,

\[
H = \text{maximum cable strength, N}
\]

\[
= \begin{cases} 
184,000 \text{ N for 1.27 cm (1/2 inch) diameter cable} \\
258,000 \text{ N for 1.6 cm (5/8 inch) diameter cable}
\end{cases}
\]

\[
x_{\text{span}} = \text{length of span at sill mat, m}
\]

\[
x_{\text{strike}} = \text{strike length of sill mat, m.}
\]

The constant \((1.66 \times 10^{-4})\) is based on the following Snip Mine site characteristics:

- Internal angle of friction \((\phi)\) = \(34^\circ\)
- Stope dip = \(52^\circ\)
- Stope height \((z)\) = \(5 \times \text{span}\)
- Backfill unit weight = \(15,000 \text{ N/m}^3\)
- Negative pore water pressure = \(1,500 \text{ Pa}\)
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- UBC Rock Mechanic Geotechnical Group

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Dedication

This Master's Thesis is dedicated to all Canadians who fought in World War II - the sacrifices they made grants us the freedom and peace that allows our pursuit of knowledge to be fulfilled.
Introduction

Sill mats are backfill supports for underground mining. Generally, cut and fill mining is a mining method incorporating sill mat design. Backfill is placed in mined out stopes as a slurry or paste material. Beneath a backfilled stope is a sill pillar and underlying the sill pillar is another backfilled stope. Figure 1 illustrates the location of the sill mat with respect to the stopes and sill pillar. The sill mat may or may not be anchored into the hangingwall and footwall - it depends on the sill mat design.

Figure 1: Generalized Sill Mat - A schematic vertical cross section of cut and fill mining utilizing a sill mat to support backfill.
In order to remove the sill pillar, the backfill in the stope above the sill pillar must be supported. The design supporting the vertical pressure exerted by the backfill is called a sill mat. This report looks at the current sill mat designs that are used in underground mining. At the Snip Mine near Stewart, British Columbia (Figure 2), sill mats are employed in cut and fill mining.

Figure 2: Snip mine location in British Columbia.

The Snip Mine utilizes cable slings as the support method for their sill mats. After reviewing current sill mat designs and accounting for site characteristics of the Snip Mine, it is suggested that the cable sling method of support for sill mats be continued. In this report, testing of the cable sling support method results in a better understanding of the technical aspects of cable slings.

Improved understanding of sill mat design is a direct result of predicting vertical stress in backfill. A new method of predicting vertical stress in backfill is presented in this report. Marcinysyn's Arching Theory Summation Series (MATSS) predicts the vertical stress exerted on sill mat support structures. MATSS is based on the arching theory model of Terzaghi (1948).
Terzaghi (1948). Employing MATSS, Snip Mine site characteristics, and cable sling properties, a mathematical equation (Cable Spacing Formula) based on cable strength, span length, and strike length estimates the spacing that should be used for cable slings. If a sill mat has already been installed, the formula can be used to estimate the maximum span that should be exposed beneath a sill mat that is supported by cable slings. Consequently, the mining of larger sill pillars would have to be modified to extract panels that will expose sill mats at widths less than or equal to that value determined by the Snip Mining Panel Formula:

\[
\frac{1}{(\text{panel width})^2} = \frac{\text{Cable Spacing}}{\left(\frac{\text{H}}{\#A}\right)} - \frac{1}{(\text{panel length})^2}
\]

where,
- cable spacing = installed cable spacing of cable slings (m)
- H = tensile breaking strength of cables (N)
- panel length = long dimension of panel to be extracted (m)
- \#A = constant.

\#A is based on new vertical stress determined from vertical stress (\(\sigma_{\text{MATSS}}\)) as outlined in Section 7.0. This number replaces 1.66E-4 of the Snip Mine Cable Spacing Formula.
1.0 Literature Review

The literature search was conducted by reviewing current electronic library services and through personal communication. Each mine has its own particular construction method for sill mats; in addition, the method of calculating vertical stress on a sill mat from backfill varied from mine to mine. Presented in chapter 1 is a summary of all information gathered that is pertinent to sill mat construction in underground mining in a low stress environment.

1.1 Information Services

The literature search for information on current Sill Mat Design was completed on the following electronic library services:

- COMPENDEX
- NTIS
- GEOREF
- METADEX
- MINDEX
- MNT
- Q&L.

The above information was supplemented with internal documents that were donated from companies. Information was found in various journal's and periodicals. All information on current sill mat design is summarized from the literature search.
1.2 Sill Mats

Sill Mat information is summarized from the following 15 mines and provides the data base for the literature review:

- INCO, Thompson Mine, Manitoba
- HUDSON BAY MINING AND SMELTING, Rod and Stall Lake Mines, Manitoba
- FALCONBRIDGE, Lockerby Mine, Sudbury Operations, Ontario
- DICKENSON MINE LTD., Red Lake, Ontario
- PLACER DOME, Detour Lake Mine, Ontario
- PLACER DOME, Campbell Mine, Red Lake, Ontario
- ECHO BAY MINES, Lupin, Northwest Territories
- COMINCO LTD., Snip Operations, B.C.
- TVX GOLD, Casa Berardi Mines, La Sarre, Quebec
- ROYAL OAK MINES, Giant Mine, Yellowknife, N.W.T.
- SOCIETE PENARROVA, Noailhac - Saint - Salvy Zinc-Lead Mine, France
- KINROSS GOLD CORPORATION, Kirkland Lake Operations, Ontario
- MAGMA-SUPERIOR MINING, Magma Copper, Arizona
- THE ZINC CORP. AND NEW BROKEN HILL CONSOLIDATED, LTD., Broken Hill, New South Wales, Australia.

The reports on each of these mines ranged from detailed engineering reports to generalized information gathered from conversations with mining personnel. The format of the reports were published papers and internal documents.

1.2.1 Sill Mat Design Methodology

Sill mats can be designed in two ways - experience and engineering. Clearly, the best way to design a sill mat is a combination of the two. Engineering a sill mat (O'Hearn et. al, 1989) can be accomplished by:

- solid mechanics theory
- numerical modeling
- physical modeling.

1.2.2 Mining Methods

Sill mats are used for sill pillar recovery and artificial roof support in underhand cut and fill mining. Figure 1.2.2.A illustrates the various mining methods that are using sill mats; however,
this is not to suggest that these are the only mining methods in which sill mats are used. Sill mats can be used where economy and safety dictate proper application of a design.

The mining methods that use sill mats are:

- Vertical Crater Retreat
- Longhole
- Mechanized Cut and Fill
- Conventional Cut and Fill
- Underhand Cut and Fill.

Combining the cut and fill mining methods (Figure 1.2.2.B) illustrates that cut and fill is the most likely mining method to apply the use of sill mats.

**Figure 1.2.2.A**: The mining methods that use sill mats versus their occurrence in the data base.

**Figure 1.2.2.B**: The cut and fill versus other mining methods that use sill mats.
1.2.3 Stope and Pillar Geometry

The dimensions are illustrated in Figure 1.2.3.A and are defined as follows:

- **Stope Span** - horizontal distance from hanging wall to foot wall
- **Stope Height** - height of stope parallel to dip
- **Pillar Span** - horizontal distance from hanging wall to foot wall
- **Pillar Height** - height of pillar parallel to dip.
The stope and pillar geometry associated with sill mat applications are listed in Table 1.2.3.A.

Table 1.2.3.B: Stope and pillar geometry from various mines using sill mats.

<table>
<thead>
<tr>
<th>Stope Dimensions</th>
<th>Pillar Dimensions</th>
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<tbody>
<tr>
<td>Span</td>
<td>Height</td>
</tr>
<tr>
<td>m</td>
<td>m</td>
</tr>
<tr>
<td>15</td>
<td>100</td>
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<td>40</td>
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<td>67</td>
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<td>40</td>
</tr>
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</table>

As illustrated by Table 1.2.3.A, there is a variety of stope and pillar dimensions that involve sill mats. There are three design stages surrounding the design and construction of a sill mat. First, the stope and pillar geometry are a function of the mining method, ore body, stress, and rock mass characteristics. Next, the design and installation of a sill mat is based on the stope and pillar geometry. Finally, the extraction of ore (UC&F) or sill pillars under a sill mat is a function of the sill mat strength. Each of these design stages must consider the other design stages for an efficient mining cycle.

The two distinctive characteristics of sill pillars can be defined. Sill pillars can support the hangingwall and footwall of a stope to prevent convergence, dilution, or rock bursts. On the other hand, sill pillars can be used to support the overlying backfill, in which case they become part of the sill mat design. A mining method may use sill pillars for both reasons.

The span of a sill mat that can be exposed determines the mining method that can be used. From the data collected it is evident that the mining method under a sill mat does not differ from the mining method used during extraction of other portions of the ore body. If the mining method differs - for instance, Cut and Fill and Longhole - both methods are used in the regular
mining cycle. From an economic point of view, using the same mining method and equipment is the most cost effective.

1.2.4 Sill Mat Construction Materials

The materials (Figure 1.2.4.A) predominantly used in sill mats are:

- timber
- backfill
- miscellaneous - metal straps, brackets, bolts, etc.

The above materials correspond to the construction of bulkheads, fill fences, sill mats and other supports for backfill. Cable slings and welded wire mesh are the next most popular items used in sill mat construction. In general, these are used to support the backfill

![Building Materials versus Occurrence](image)

**Figure 1.2.4.A: Building Materials versus Occurrence**

and timber in sill mats and bulkheads. Chain link is also used as support, but it is not as popular most likely due to cost. Geotextiles have been used in some applications. Flexible cable is used in some operations despite the lower load bearing capacity compared to cable slings (high tensile steel cable). In this case, a sill mat design is adapted to the load capacity being aided or transferred to other sill mat material.
Backfill is used in all cases involving sill mats. The backfill types displayed in Figure 1.2.4.B are:

- tailings
- rockfill
- smelter slag
- aggregates
- cemented backfill.

Cement is not necessarily added when rockfill or smelter slag is used for backfill. Cemented backfill is generally used when mill tailings or aggregates make up the backfill material.

Aggregates are used in an underhand cut and fill mining method when high level of stability for a sill mat is required.

Figure 1.2.4.B: Backfill types versus occurrences.
1.2.5 Ore Recovery

When sill mats are installed, 100 % ore recovery is expected. In a few cases (Figure 1.2.5.A), the ore recovery is not 100 %. In both cases, longhole mining is used to recover a sill pillar. Recovery of large pillars at deep levels can be hindered by stress problems and rockbursts. Remnant pillars, left in place to prevent overlying backfill from cavitating into working stopes, also reduces ore recovery in sill pillars. Some dilution may occur from adjacent backfilled stopes; in spite of dilution, ore recovery of 100% can be achieved. Other than isolated cases, sill pillar ore recovery should be close to 100% with proper planning.

Figure 1.2.5.A: Ore recovery versus occurrences.
1.2.6 Numerical Modeling

The following numerical modeling codes have been used to model sill mats:

- FLAC
- BEAP3D
- GENA3D
- MUDEC
- MAP3D
- UDEC.

FLAC is a finite difference program which uses a non-linear model and can handle large grid deformations. It is capable of simulating arching effects and excavation sequences (Tansey, 1993). A FLAC model can be used to analyze sill mat pressure and backfill behavior. Also, it can be used to determine footwall and hangingwall movements. FLAC can simulate a slip plane interface; the interface allows grids on either side of it to slide past one another (CANMET, 1989).

GENA3D is a three dimensional finite element modeling program used to approximate mining geometry and the distribution of stresses (Tansey, 1993).

MUDEC is a distinct element program which can simulate static and dynamic loading conditions on a sill mat. Backfill - ore interfaces should be finely discretized in order to allow the failure surface to form its own course with as little dependence on block size as possible. This is to prevent failures occurring along block interfaces in the numerical model (CANMET, Falconbridge, 1989). Boundary velocity conditions can be set to zero to simulate the effect of fill resting against a solid rock interface. Other boundaries can be made viscous to allow shock waves due to blasting to pass through an interface.

MAP3D (Wiles, 1991) is a three dimensional boundary element program which simulates rockmass response and predicts displacements, stresses and safety factors around rock slopes, open pits, tunnels, and underground excavations in rock (Mine Modeling Limited, 1993). A similar three dimensional boundary element program used for stress analysis is BEAP3D (CANMET, 1991).
UDEC is an explicit, distinct element code which solves the equations of motion in finite difference form (Cundall, 1974). Unlike finite elements, distinct elements may interact with any of the other elements and can experience large scale rigid body translations and rotations. The geometry of distinct elements is defined by the spacing and orientation of joints or layers in the material mass being modeled, with each element corresponding to an individual block of material. (O'Hearn et. al, 1989). Clearly, this code is well suited to sill mat modeling where layers of various tailing:cement ratios are used.

1.2.7 Instrumentation

Instrumentation can be installed in rock and backfill to gather information about sill mat performance. In rock, instrumentation is installed in the hanging wall and foot wall to determine the influence of a sill mat on parameters such as wall convergence and stress. The same instrumentation can be installed in a rock sill pillar to determine its performance as mining approaches the sill pillar. Soil instrumentation (civil/geotechnical) is applied to backfill monitoring. When possible, instrumentation is adapted to each situation to improve the performance of the instrument. From the data base, instrumentation and its uses at mine sites were determined and are listed below:

- Strain and stress cells - measure deformation which can be related to rock stress.
- Ground Movement Monitors - inexpensive, reusable single point extensometers for measuring rock movement within 10 meters of hole collar. Deformation read out from a LVDT.
- Geophones - measure peak particle velocity during blasts, rock bursts, or other seismic events in rock.
- Multipoint extensometers - wire/rod/magnetic extensometers used for measuring rock and soil deformation.
- Glotzl pressure cell - measures backfill pressure.
- Load cells - used to measure the load on supports.
- Piezometer - used to measure water pressure.
- Penetration Cone - used to determine internal angle of friction and shear strength.
- Texam pressure meter - measurement for modulus of elasticity.
- USBM Ultrasonic Measuring Device - used to monitor fill failure profile.

The above is a summary - but not a limited list - of available instrumentation that can be applied to sill mat design. It must be noted that the installation of each instrument is adapted to the application as required. In addition, the design of simple and effective forms of
instrumentation other than those listed above, should be utilized to obtain quality information that can be used to refine current sill mat design.

1.2.8 Engineering Considerations

The engineering design for a sill mat may consider one or more of the following parameters:

- backfill strength and the load it exerts on a support structure.
- cemented backfill strength and if it is not the support structure, the load it exerts on a support structure.
- support structure strength.

The behavior of a support structure, when it contains engineered materials such as lumber or cable, is predictable as long as the material properties do not change over time. Conversely, the engineering properties and behavior of backfill and cemented backfill are uncertain even though material testing is done. The unpredictable behavior of cemented and uncemented backfill is due to the variables involved in placement of the material.

1.2.8.1 Backfill

The performance of uncemented backfill within inclined stopes have been addressed by two authors - Mitchell (1991) and Blight (1984).

Blight (Figure 1,2,8,A) defines the vertical stress of backfill on a support structure as:

\[
\sigma_z = \frac{\omega \gamma \sin \beta}{(2 \kappa \tan \phi')}
\]

Note: assumes cohesion = 0 and negative pore pressure = 0 for maximum load on sill mat.

where,

- \( \omega \) = stope width
- \( \gamma \) = unit weight of fill
- \( \gamma_w \) = unit weight of water
- \( \beta \) = inclination of stope
- \( \phi' \) = drained friction angle
- \( \kappa \) = coefficient of earth pressure.
Figure 1.2.8.A: Vertical stress model in soil (Blight, 1984).

If the fill is settling sufficiently in relation to its boundaries to develop its full shear strength, $K$ becomes equal to the coefficient of sliding wall friction (Blight, 1984):

$$K_w = 1 - \sin^2 \phi' / (1 + \sin^2 \phi')$$

Negative pore pressure results from drained fill and this is accounted for by the following formula (Blight, 1984):

Negative Pore Pressure ($\mu$) = $- [\omega \sec(\beta) \gamma_w] / 2$
Mitchell defines the vertical stress of backfill on a support structure as:

\[ \sigma_v = \frac{\gamma L}{2 \kappa \tan \phi} \]

where,
- \( \gamma \) = the unit weight of fill
- \( \kappa \) = coefficient of earth pressure. Assumed to be unity in the absence of knowledge on the vertical stress distribution.
- \( L \) = horizontal stope width
- \( \phi \) = backfill friction angle.

\[ \sigma_v = \left( \frac{\gamma A}{\tan \phi} \right) \left[ 1 - e^{-2Ko\tan \phi} \right] \]

where,
- \( A \) = \( \frac{2ab - a^2}{4b} \)
- \( a \) = span
- \( b \) = strike
- \( \gamma \) = unit weight of backfill
- \( \phi \) = friction angle
- \( z \) = depth below surface of backfill
- \( Ko \) = \( 1 - \sin \phi \).

Figure 1.2.8.B: Sill mat failure models (Mitchell, 1991)
Coates (1965) uses a similar equation to Knutsson; however, the horizontal area of the sill mat is used to calculate the hydraulic radius. Coates equation for vertical stress is:

$$\sigma_v = \gamma b / [2 K \tan(\phi) (b/L + 1)]$$

where,
- $b =$ span
- $L =$ strike
- $\gamma =$ unit weight of backfill
- $\phi =$ friction angle
- $K =$ coefficient of lateral earth pressure.

Pakalnis (1995) uses an equation derived from the geometry of an assumed arched surface. The arched surface is illustrated in Figure 1.2.8.C

![Figure 1.2.8.C: Schematic arched surface (Pakalnis, 1995).](image)

The arched load per meter along strike is defined as:

$$\text{Arched Load} = (0.25 W^2 \tan(B)) \gamma$$

where,
- $W =$ span
- $\gamma =$ backfill density
- $B =$ arch angle (Figure 1.2.8.C)

Relating the arched load to Blight's Equation (1984), results in:

$$\tan(B) = 2/(K \tan(\phi))$$
Clearly, the vertical stress determined by most authors is dependent on the friction angle, stope width, coefficient of lateral earth pressure, and the density of the backfill; in addition, each of the different models incorporate the model used by Terzaghi (1948) - Figure 7.1.A in section 7.0.

1.2.8.2 Cemented Backfill

Cemented backfill studies for sill mats appear to concentrate on failure analysis. O'Hearn (1989) and Mitchell (1991) have used models to determine failure modes of cemented sill mats. Mitchell (Figure 1.2.8.B) suggests failure modes are governed by the dimensions of the sill mat. A wide thin sill mat is susceptible to flexural failure due to low tensile strength of cemented tailings. Using standard flexural formulae for a fixed end uniformly loaded beam, failure is predicted when (Mitchell, 1991):

\[(L/d)^2 > \frac{2(\sigma_t + \sigma_c)}{\omega}\]

where,
- \(L\) = horizontal stope width
- \(d\) = vertical height of cemented sill
- \(\sigma_t\) = tensile strength of cemented sill
- \(\sigma_c\) = horizontal confining stress
- \(\omega\) = uniform loading which should include the self-weight of the sill mat.

A thick, narrow sill mat might be more prone to caving or undergo side wall shear failure. Assuming that caving would extend to a stable arch of height \(L/2\) (semi-circular arch), then all nonreinforced sills would be formed to a depth, \(d > L/2\) and caving would develop when (Mitchell, 1991):

\[L \gamma > 8 \sigma_t / \pi = 2.5 \sigma_t\]

From equilibrium, block sliding of the sill due to side shear failure occurs when (Mitchell, 1991):

\[(\sigma_v + d \gamma) > 2 \left( \frac{\tau_f}{\sin^2 \beta} \right) (d/L)\]

where,
- \(\tau_f\) = shear strength of the fill-wall rock interface
- \(\beta\) = dip angle.

A fourth failure mode is suggested by Mitchell. When the shearing resistance at the hanging wall contact is low due to poor quality hanging wall rocks and/or low hanging wall dip angles,
torsional failure is most likely to develop. Rotational failure would develop when (Mitchell, 1991):

\[(\sigma_v + d\gamma) > (d^2\sigma_t) / [2L(L - d\cot\beta)\sin^2\beta]\]

The previous analyses are derived for nonreinforced, cemented tailing sill mats. Reinforcements (steel wire, mesh, or geogrid systems) can be used to improve the tensile or flexural performance of the mat. Reinforcing elements could also be fastened to the walls, particularly the hanging wall, to prevent wall shear or tortional failures (Mitchell, 1991).

Laboratory centrifuge modeling produced some interesting conclusions. Even with a high degree of wall roughness, the hanging wall contact provides little resistance to slippage resulting in rotation about the foot wall contact. A thicker sill mat results in caving failure in the cemented backfill. Models with vertical walls generally exhibit flexural, rather than torsional, failure as a result of the better shearing resistance in the hanging wall contact; however, this improvement appears to be offset by larger vertical stress (\(\sigma_v\)). Consequently, this modeling suggests that the dip of the stope does not appear to be a major factor in sill mat design (Mitchell, 1991). Other points to note from the model study are (Mitchell, 1991):

- Steel members used for support in the models failed as a result of large flexural deformations of the sill mat. Wooden members failed in shear and did not provide as much support to the sill mat as the steel did.

- Experience has shown that high cement content nonreinforced sill mats are successful when there is good wall rock anchorage and the closure strains are sufficiently low that lateral crushing of these relatively stiff mats is not a problem.

- Models with wire reinforcement failed by sill mat cracking followed by rotation and rupture of the reinforcing wires.

- Using timber mats beneath cemented sill mats may be a good solution to cases where high closure displacements may cause crushing of the cemented sill mat.

- In cases where crushing is not a problem, the use of anchored wire mesh reinforcements in the cemented mat would be an effective alternative to the timber mat.

- Models indicate plain sills may be stable for spans up to 5 meters as long as an effective hanging wall contact can be provided.
• Sill rotation (torsional shear) appears to be a prevalent failure mode for stope dips of 70 degrees or less.

O’Hearn uses a different approach to sill mat modeling employing Voussoir beam analysis (Beer and Meek, 1982). The voussoir beam analysis assumes an abutment load distribution and a parabolic-shaped thrust line (Figure 1.2.8.C). There are three failure modes (O’Hearn et. al, 1989) possible in the Voussoir beam:

1. compressive failure in the arch.
2. shear failure in the abutments.
3. slough or tensile failure beneath the arch itself.

A factor of safety is used to determine a compressive failure in the arch and is defined as:

\[ F \text{ of } S = \frac{\text{uniaxial compressive strength of the fill}}{\text{maximum longitudinal stress}} = \frac{\text{UCS}}{f_c} \]

For shear failure at the abutments, the shear stress is compared to a Mohr-Coulomb strength given by:

\[ 0.5 f_c n \tan(\phi) C \]

where,

- \( \phi \) = interfacial friction angle
- \( C \) = cohesion
- \( n \) = the load / depth ratio
- \[ n = \frac{3}{2} \left( 1 - \frac{z}{t} \right) \]
- \( f_c \) = \[ \frac{1}{4} \left( \frac{\gamma s^2}{nz} \right) \]
- \( s \) = span
- \( \gamma \) = density of beam material
- \( z \) = lever arm (Figure 1.2.8.C)
- \( t \) = beam thickness.
Figure 1.2.8.C
Voussoir Beam Model
(Beer and Meek, 1982)
An estimate of the potential slough material (due to vibration from blasting or rock bursts) may be obtained from a consideration of the material volume beneath the theoretical arch (O’Hearn, 1989). This is given by:

\[ 0.677 t (1 - n) s \]

where,

\[ t = \text{dimension in Figure 1.2.8.C} \]
\[ s = \text{span}. \]

From this, the weight of material to be supported in the event of sloughing can be calculated. It is suggested this is the load which a timber beam support system should be designed to carry.

The two different models of Mitchell and O’Hearn result in similar failures. Both models predict:

- a beam failure - compression or tensile strengths exceeded.
- a slough or caving failure - tensile strength exceeded.
- a shear failure - at stope wall sill mat interface.

In addition, Mitchell suggests that a rotational failure also occurs when a shear failure - as defined above - occurs.
1.2.8.3 Sill Mat Support Materials

Support structures for sill mats - when the last level of support is not cemented backfill -
are built from the following items:

- lumber
- high tensile strength cables
- flexible cable
- welded wire mesh
- chain link mesh.

An engineering design can be based on backfill, cemented backfill, and the above materials. It is
the above materials that are better suited to an engineering design since the quality of backfill and
cemented backfill is questionable. However, degradation of the above materials may result from
the natural underground environment in a backfilled stope. A sill pillar below a sill mat which
incorporates one or more of the above materials should be removed as soon as possible to
maintain the integrity of the sill mat strength.

Lumber

Stulls are the timber beams that lie across the span of a sill mat providing support to the above
sill mat. Mitchell (1991) and Pakalnis (1995) have used beam design to determine the support
capabilities of stulls.

Mitchell considers the stulls to act as independent beams with free end conditions. Also, he
considers bending compatibility between the mat and the cemented sill which results in the
equation:

\[ q = \frac{w}{1 + \left(5S(EI)_{\text{sill}} / (EI)_{\text{stull}}\right)} \]

where,

- \( w \) = \((\sigma_v + d\gamma)\), the total unit loading.
- \( q \) = the unit loading supported by the stull.
- \( S \) = stull spacing.
- \( EI \) = section modulus.

For longitudinal shear at failure:

\[ \sigma_v L S > 1.3 \text{ (tA)s} \]

where,
(tA)s = shearing resistance (sectional area times the allowable shear strength of the stuff).

L = horizontal stope width.

For flexural failure:

\[ w \left[ \frac{S L^2}{t^3} \right] > 0.785 \sigma_s \]

where,
\[ \sigma_s = \text{the tensile (flexural) strength of the stuff material.} \]
\[ t = \text{the diameter of the stuff.} \]

Pakalnis (1995) follows guidelines in the Canadian Institute of Timber Construction resulting in the following equations:

Internal Bending Moment of a Beam (M_x) = \[ \frac{wL^2}{8} \]
Note: greatest moment at center of beam.

Maximum Normal Stress at Center of Beam (Fb) = \[ \frac{M_x (h/2)}{I_{\text{square}}} \]
Note: equation not corrected for units circular beam = \[ \frac{M_x (d/2)}{I_{\text{circle}}} \]

where,
\[ b = \text{width of beam (in)} \]
\[ h = \text{height of beam (in)} \]
\[ d = \text{diameter of beam (in)} \]
\[ l = \text{length of beam (ft)} \]
\[ w = \text{uniform load (lb/ft)} \]

Moment of Inertia
\[ I_{\text{square}} = \frac{bh^3}{12} \]
\[ I_{\text{circle}} = \frac{\pi d^4}{64} \]

Note: these equations are for solid beams

The lumber properties of the stuffs must be known to design for support capabilities; however, the change in stuff properties while sitting within the sill mat must be taken into account when undercutting the sill mat.
Cable Force Diagrams

Cable support for sill mats can utilize high strength tensile cable, flexible cable, or other suitable cable. When cable slings are installed high tensile strength cable is used. Pakalnis (1995) calculates tension based on the geometry of the cable sag described by equations given in Nash (1978). Figure 1.2.8.3.A illustrates the geometry of a cable sling when deflected by a vertical load.

\[
d = \frac{wa}{8H} \\
T = \frac{1}{2} wa \left[ 1 + \left( \frac{a^2}{16d^2} \right) \right]^{0.5} \\
I = a \left[ 1 + \frac{8}{3} (d/a)^2 - \frac{32}{5} (d/a)^4 + \frac{256}{7} (d/a)^6 - \ldots \right]
\]

where,

- \(d\) = cable sag (ft or m)
- \(w\) = load (lb/ft or N/m)
- \(a\) = span (ft or m)
- \(H\) = tension at midpoint (lb or N)
- \(T\) = tension at supports (lb or N)
- \(I\) = length of cable (ft or m).

Figure 1.2.8.3.A: Cable sling deflection geometry based on Nash (1978).
The force diagram for support cables that is used by Bharti Engineering Associates (Tansey, 1993) for sill mat design is illustrated in Figure 1.2.8.3.B. The vertical distance \( y \) is the amount of sag in the cable. The sag is critical to the tensile force in the cable, wire rope, or wire mesh and is dependent on the type and gauge of the material used.

\[
\text{Shear Force at End Support} \quad W = wx \\
\text{Tensile Force at Cable Center} \quad T_0 = \frac{wx^2}{(2y)} \quad \text{From Moment about B.} \\
\text{Tensile Force at End Support} \quad T = \sqrt{(T_0^2 + W^2)}^{0.5} \\
\text{Angle of Cable at End Support} \quad \theta = \tan^{-1}\left(\frac{W}{T_0}\right) \\
\text{Half Length of Cable} \quad S = \int_{c}^{b} \left[1 + \left(\frac{dy}{dx}\right)^2\right]^{0.5} dx
\]

where,
\( w \) = uniform line load
\( x \) = horizontal distance center (point C) to end cable, wire rope, wire mesh (point B)
\( y \) = vertical deflection or sag at center (point C) of cable, wire rope, wire mesh
\( T_0 \) = tensile force at cable, wire rope, wire mesh center (point C)
\( T \) = tensile force at end support (point B)
\( W \) = shear force at end support (point B).

**Figure 1.2.8.3.B:** Cable force diagram based on Tansey (1993).
In both cases - Pakalnis and Bharti - the geometry of the cable is dependent on lagging, welded wire mesh, or other materials; however, these systems provide a good approximation to the loads that can be expected when cables are installed as a supporting structure in a sill mat. The cable system can be used when mining the final lift of a stope under a sill mat. Undercutting the sill mat exposes the cable. The sag in the cable will indicate the tension in the cable; thus, the supported load can be determined as long as the initial installation loads are known and proper anchoring of the cables has been maintained.

**Cable, Wire Rope, and Wire Mesh Strength**

The shear and tensile strengths of cable, wire rope, and wire mesh that has been used by Bharti Engineering Associates (Tansey, 1993) are given below.

**Tensile Resistance**

\[ \text{Tensile Resistance} \quad T_r = \phi A F_y \]

where,
- \( \phi \) = strength reduction factor (used 0.9)
- \( A \) = x-sectional area of cable or wire rope
- \( \phi \) = x-sectional area of single layer of wire mesh of the corresponding cable, wire rope spacing
- \( F_y \) = yield strength of cable, wire rope, or wire mesh.

**Shear Resistance**

\[ \text{Shear Resistance} \quad V_r = 0.6 \phi_b A F_u \]

where,
- \( V_r \) = factored shear resistance of cable or wire rope
- \( \phi_b \) = factored shear resistance of a single layer of wire mesh
- \( \phi_b \) = reduction factor (used 0.67)
- \( A \) = x-section area of cable, wire rope
- \( A \) = x-sectional area of single layer of wire mesh of the corresponding cable, wire rope spacing
- \( F_u \) = tensile strength of cable, wire rope, or wire mesh.

The shear resistance above is compared to the wall bearing resistance below and the lesser of the two is used for design.

**Factored Wall Bearing Resistance**

\[ \text{Factored Wall Bearing Resistance} \quad B_r = 1.4 \phi_c A f_{c'} \]

where,
- \( \phi_c \) = reduction factor (used 0.6)
- \( A \) = area of bearing (taken as \( d \times 5d \))
- \( d \) = diameter of the anchored cable, wire rope
- \( f_{c'} \) = compressive strength of the wall rock.
Anchor Length of Cable, Wire Rope in Grout

\[ L_d = \frac{0.019 \ A \ F_y/(f_{c'}^{0.5})}{(2 - 400/f_y) \times 0.8} \]

where,
- \( A \) = x-sectional area of cable, wire rope
- \( f_y \) = yield strength of cable, wire rope
- \( f_{c'} \) = compressive strength of grout.

Due to the nature of the cable installation in Bharti’s design (Tansey, 1993), the shearing effect of the supported load becomes important and is accounted for in the previous equations. Conversely, the installation method for cable slings reduces the probability of shearing since cable movement will occur. Tensile failure is the most likely failure mode cable slings will undergo.

**Cable Spacing**

Once the load bearing capacity of support cables is known, the cable spacing must be determined. The following formula is used by Bharti Engineering Associates (Tansey, 1993) for the longitudinal or transverse cable spacing:

\[ S^2 < \left( \frac{V_f V_r}{V_f V_r} \right)^2 + \left( \frac{T_f T_r}{V_f V_r} \right)^2 \]

From,

\[ (S \times V_f V_r)^2 + (S \times T_f T_r)^2 < 1.0 \]

where,
- \( S \) = cable, wire rope spacing
- \( V_f \) = unit shear force acting on cable, wire rope at anchor point
- \( T_f \) = unit tensile force acting on cable, wire rope at anchor point
- \( V_r \) = shear resistance of individual cable, wire rope
- \( T_f \) = tensile resistance of individual cable, wire rope.

The above equation takes into consideration the shear forces of the Bharti sill mat design (CANMET, 1993). The cable sling sill mat design (Pakalnis, 1995) deals only with the tensile support strength of cables. Since a cable sling is allowed to move at all points along the span (including the anchor point), tensile failure of the cable is the most likely failure mode. As a result, the cable spacing is determined by:

1. finding the backfill load along the span.
2. finding the cable strength along the span.
The backfill load is found by using Blight’s vertical stress formula and correcting for the area of influence along strike that a cable is supporting. The cable tension is found using the equations given in Nash (1978). Equations for both steps are previously discussed. Clearly, step 1 and 2 require knowledge of the cable spacing and sag, respectively. Thus, an iterative calculation is completed to determine cable spacing.

**Wire Mesh Layers**

The number of layers of wire mesh required is (Tansey, 1993):

\[
N > \left[ \frac{(T_f/T_r)^2 + (V_f/V_r)^2}{0.5} \right]
\]

\[
\text{From, } \left[ \frac{T_f}{N \cdot T_r} \right]^2 + \left[ \frac{V_f}{N \cdot V_r} \right]^2 < 1.0
\]

where,

- \(N\) = required number of layers of wire mesh
- \(T_f\) = tensile force acting on wire mesh at end support
- \(V_f\) = shear force acting on wire mesh at end support
- \(T_r\) = tensile resistance of single layer of wire mesh
- \(V_r\) = shear resistance of single layer of wire mesh.

The number of wire mesh layers must be calculated in conjunction with the cable, wire rope spacing since the tensile and shear forces are dependent on the spacing used.

**1.3 Summary of Literature Search**

Experience and engineering are key elements to sill mat design. The most likely mining method for sill pillar recovery under a sill mat is cut and fill. Sill mats have been applied to a variety of stope and pillar geometry. The property of the following materials must considered for sill mat design:

- backfill
- cemented backfill
- support materials.

When sill mats are used, ore recovery should be 100%. Numerical and laboratory modeling have been used as valuable tools for sill mat design and instrumentation is used for monitoring sill mat performance. However, experience and engineering have to be applied to sill mat construction to ensure the stability of a sill mat and the safety of the workmen underneath it.
2.0 Snip Mine Geotechnical Assessment

The Snip mine geotechnical assessment is divided into three categories: geological stress, structure, and rockmass. For stress determination, MAP3D (Wiles, 1994) is used to simulate the rockmass. Structural geology from field data is analyzed and presented using DIPS (Hoek and Diederichs, 1989). The Rock Mass Rating (RMR) developed by Bieniawski (1973) is used to classify the rockmass. The stress, structure, and rock mass rating are used to determine if any aspects of sill mat design are affected by the rock characteristics.

The Snip ore body lies just below the surface of a mountain (Figure 2.0.A). All areas of the mine are within 500 meters of the surface.

Figure 2.0.A: Snip mine long section looking northeast. The mine lies just below the surface of the mountain slope.
Figure 2.0.B is a general cross section of the Snip ore body. The cross section looks northwest. It is evident from this cross-section that there are two zones, the main ore body and the secondary ore body - called the Twin Zone and 150 Zone respectively. The 150 Zone lies in the footwall of the Twin Zone. The Twin Zone is a 0.5 to 15 meter thick shear structure (Brown, 1994) and is generally mined by mechanized cut and fill. The 150 Zone is less than 5 meters thick and averages 2-3 meters thickness and is mined by conventional cut and fill.
2.1 Stress

MAP3D simulates rockmass response in three dimensions and predicts displacements, stresses and safety factors around rock slopes, open pits, tunnels and underground excavations in rock. The model formulation is based on the efficient Indirect Boundary Element Method using constant intensity "fictitious force" and displacement discontinuity type boundary elements (Wiles, 1994).

A simplified mine model is used for numerical stress modelling and is illustrated in Figure 2.1.A.

Figure 2.1.A: Simplified numerical mine model that reduces computer run times.
The dimensions of the stopes and pillars for the numerical model are listed in Table 2.1.A. These dimensions typify the general mechanized and conventional stopes of the Snip underground mine. The sill pillar in the numerical model is between 100 and 200 meters below the sloped mountain surface.

<p>| Table 2.1.A: Stope and pillar dimensions used in the numerical stress model. |
|-----------------------------|-----------------------------|</p>
<table>
<thead>
<tr>
<th></th>
<th>Conventional</th>
<th>Mechanized</th>
</tr>
</thead>
<tbody>
<tr>
<td>Span</td>
<td>3 meters</td>
<td>10 meters</td>
</tr>
<tr>
<td>Strike</td>
<td>50 meters</td>
<td>100 meters</td>
</tr>
<tr>
<td>Dip</td>
<td>45 degrees</td>
<td>45 degrees</td>
</tr>
<tr>
<td>Vertical Height</td>
<td>40 meters</td>
<td>40 meters</td>
</tr>
<tr>
<td>Vertical Pillar Height</td>
<td>3 meters</td>
<td>6 meters</td>
</tr>
</tbody>
</table>

There are three materials defined for the Snip Mine numerical stress model - graywacke wall rock, sill pillar ore, and backfill. Backfill is placed in the upper and lower stopes for the numerical model - backfill properties based on CANMET (1990). The ore material is weaker than the host material graywacke. The material properties for the three materials is listed in Table 2.1.B.

| Table 2.1.B: Rock, backfill, and cemented backfill properties for the numerical stress model based on rock mass rating and similar rock types described by Hoek and Brown (1980). |
|-----------------------------|-----------------------------|
| Material Elastic Properties |
|                            | Young's Modulus | Poisson's Ratio |
| Graywacke                  | 50 GPa            | 0.25            |
| Ore Pillar                 | 10 GPa            | 0.25            |
| Backfill                   | 0.06 GPa          | 0.25            |
| Material Strength Properties |
|                            | UCS  | m | s   | Friction Angle |
| Graywacke                  | 150 MPa | 10 | 0.1 |
| Ore Pillar                 | 40 MPa | 0.5 | 0.0001 |
| Backfill                   |       |   | 35° |
Stress analyses on the Snip Mine model are run under two stress conditions in order to determine if rockbursting is or will become a potential problem for sill pillar removal. The first stress analysis is run with the horizontal stress equal to the vertical stress. The next stress analysis is run with the horizontal stress equal to 3 times the vertical stress. The results are listed in Table 2.1.C.

<table>
<thead>
<tr>
<th>Sill Pillar Stress</th>
<th>Mechanized Stope</th>
<th>Conventional Stope</th>
</tr>
</thead>
<tbody>
<tr>
<td>Stress Parameters</td>
<td>MPa</td>
<td>MPa</td>
</tr>
<tr>
<td>Horizontal = Vertical</td>
<td>20-30</td>
<td>40-70</td>
</tr>
<tr>
<td>Horizontal = 3 x Vertical</td>
<td>80 - 100</td>
<td>180-200</td>
</tr>
</tbody>
</table>
Figure 2.1.B is a plot of the major principle stress ($\sigma_1$) for the stress analysis $\sigma_h = \sigma_v$. There is a stress buildup at the hangingwall and footwall since the graywacke is a higher modulus material. This characteristic disappears for stress analysis for $\sigma_h = 3\sigma_v$. The ore zone/graywacke interface is not as apparent with a higher horizontal stress. In both stress models there is a significant increase in sill pillar stress in the conventional stopes. This can be attributed to the smaller pillar and stope dimensions. Stress in smaller stopes is less likely to be shed elsewhere; in comparison, larger stopes will tend to shed the stress over larger areas thus reducing stress buildup. With $\sigma_h = 3\sigma_v$, stress in the conventional stopes surpasses the uniaxial compressive strength of graywacke (150 - 200 MPa); consequently, rockbursting could become a potential problem. Other indications of high horizontal stress would be yielding of the softer ore zone. Yielding of the ore zone has not been seen at the Snip mine. Since rockbursting or pillar yielding has not been encountered at the Snip Mine, the insitu stress regime is not expected to be $\sigma_h = 3\sigma_v$.
An important aspect of backfill pressure that concerns sill mat design is illustrated in Figure 2.1.C. An area of low backfill stress occurs just above the sill pillar - Figure 2.1.C is a plot of the major principle stress axis ($\sigma_1$). Clearly, the horizontal stress is reduced above the sill pillar. This would reduce the passive lateral coefficient of earth pressure ($K_p$).

**Figure 2.1.C:** Low backfill stress ($\sigma_1$) distribution above sill pillar. The major principal stress ($\sigma_1$) is the horizontal stress.
2.2 Structure

Structural geology data are plotted using DIPS (Hoek and Diederichs, 1993). DIPS is a software program that plots structural geology on a stereonet and allows for pole plot contouring. Pole plot contouring on a polar stereonet for all levels in the Snip mine is illustrated in Figure 2.2.A.

Figure 2.2.A: Polar stereonet plot of structural geology. Equal angle, lower hemisphere, Fisher pole plot.

Major pole #1 has a dip of 44° and a dip direction of 176°. The dip direction corresponds to 86° strike direction. Since all measurements are taken with respect to mine north (29° east of true north), this results in a strike direction of 115° true north. This relates to the orientation of
the ore body and is close to that documented in previous reports (Brown, 1994): 40-60° dip and 120° strike.

Joint set #2 and #3 are two other major joint sets and do not form a wedge. A third joint is required to form a wedge in the hangingwall or footwall of a stope. The occurrence of this is dependent on a random joint at the proper orientation. To date wedges have not been a problem. Failure of stope walls will most likely occur from parallel joints to the ore body. Since stopes are backfilled, sill mat design should not be affected by wedge failure in the hangingwall and footwall. In the back of a stope, a wedge can form from joint set #1, #2, and #3. But up to this time, a wedge of this nature has been controlled by local support. However, as mining progresses to new areas, structure should be considered as a design parameter for further sill mats.

2.3 Rockmass

Rockmass Rating (RMR) data was collected around areas of previous sill mats (Tables 2.3.A). In general, the mine site RMR for the hangingwall and footwall range in value from 60% to 80% with an average from 70% to 75%. "Good" (Bieniawski, 1976) hangingwall and footwall RMR indicates excellent cable sling anchoring for sill mat design.

<table>
<thead>
<tr>
<th>LOCATION</th>
<th>HW</th>
<th>FW</th>
<th>BACK</th>
</tr>
</thead>
<tbody>
<tr>
<td>3852 East Side</td>
<td>60</td>
<td>74</td>
<td>40</td>
</tr>
<tr>
<td>3764 West Side</td>
<td>64</td>
<td>80</td>
<td>50</td>
</tr>
<tr>
<td>3461 West Side</td>
<td>75</td>
<td>77</td>
<td>43</td>
</tr>
<tr>
<td>3242 West of Fault</td>
<td>57</td>
<td>75</td>
<td>45</td>
</tr>
<tr>
<td>3049 West, near SUB station</td>
<td>62</td>
<td>54</td>
<td>48</td>
</tr>
<tr>
<td>3049 East, near SUB station</td>
<td>60</td>
<td>60</td>
<td>60</td>
</tr>
<tr>
<td>SUB station</td>
<td>65</td>
<td>65</td>
<td>65</td>
</tr>
<tr>
<td>2647 West side.</td>
<td>67</td>
<td>72</td>
<td>55</td>
</tr>
<tr>
<td>2647 East side.</td>
<td>66</td>
<td>66</td>
<td>60</td>
</tr>
<tr>
<td>2236, East side.</td>
<td>70</td>
<td>72</td>
<td>67</td>
</tr>
<tr>
<td>2236, West side.</td>
<td>65</td>
<td>65</td>
<td>52</td>
</tr>
<tr>
<td>4867 Sill Mat</td>
<td>70</td>
<td>69</td>
<td>62</td>
</tr>
<tr>
<td>2656 Sill Mat</td>
<td>75</td>
<td>80</td>
<td>65</td>
</tr>
</tbody>
</table>
The histogram in Figure 2.3.A illustrates the distribution of rock mass rating for the Snip mine. The ore zone containing the Biotite Spotted Dike (BSU) unit has an RMR ranging from 25% to 60% averaging 45-50% RMR. Where the BSU is the dominant feature, the ground is usually poor and is characterized by low RMR values. The RMR can be linked to the stress concentration in a sill pillar. The higher the RMR, the higher the stress concentration that can be expected. Conversely, a lower RMR will result in higher strain values; thus, more deformation of the sill pillar can be expected when mining the final sill pillar. However, the induced mining stress at the Snip mine site is not one of the primary concerns as discussed in section 2.1.

Figure 2.3.A: Rock mass rating (Bieniawski, 1976) distribution for Snip mine.
2.4 Rock Mechanics Summary

The rock mechanics characteristics of the Snip Mine - stress, structure, and rock mass - favor the installation of sill mats. The stress concentration in sill pillars is substantially lower than the rock strength. At higher stress concentrations, the weaker ore should yield reducing the potential for rockbursts. Large scale wedges are unlikely; furthermore, backfilling prevents the movement of rock into the stopes. The favorable RMR of the hangingwall and footwall aid in the installation of cable slings for the sill mats - the wall rock provides a strong anchor point for the cable slings. However, if cable slings are installed in a large wedge, excess wall movement may cause problems; conversely, some wall movement is required to mobilize shear strength in backfill. Any rock weakness within the ore zone is already handled by bolting and screening. Based on the rock mechanics characteristics, mining below a sill mat should not cause great concern if the sill mat is properly designed.
3.0 Current Sill Mat Design

The Snip Mine has installed 14 sill mats over a 6 year period, 1990 - 1996. Figure 3.0.A illustrates the sill mats that have been installed in the Twin Zone.

Figure 3.0.A: Mine long section of current and proposed sill mats for the Twin Zone (mechanized stopes).
Various designs were constructed until the use of cable slings appeared to give the best results for supporting sill mats. The current design is illustrated in cross section on Figure 3.0.B.

![Cross section of current sill mat design](image)

**Figure 3.0.B**: Cross section of current sill mat design.

### 3.1 Sill Mat Installation

After the undercut of a new stope is mined, a sill mat is constructed above the sill (stope floor). The first part of sill mat construction is the installation of cable slings which are installed along the span and strike directions of a stope (Figure 3.1.A). Perforated plastic pipes (Big O pipes) are installed below the cable slings to aid in water drainage of the sill mat. A cable sling
Figure 3.1.A: Schematic plan view of a stope with the cable sling layout.

consists of high tensile cable anchored into rock with a split set and cable grip (Figure 3.1.B). Grout cartridges (Figure 3.1.C) are placed into a drillhole before the split set is driven in with a jackleg. The amount of grout cartridges used to cement each split set depends on the hole condition and the ability of the crew to load the grout cartridges in the holes. In general, 5-7 grout cement cartridges are used in 2.4 meter (8 feet) holes with 1.27 centimeter (1/2 inch) diameter cable and 7-9 grout cartridges in 3 meter (10 feet) holes with 1.6 centimeter (5/8 inch) diameter cable. Drill hole diameters for split sets are 3.18 cm (1 1/4 inch) and 3.80 (1 1/2 inch) for 1.27 and 1.6 centimeter cable, respectively. At locations where the span cables intersect the strike cables, Crosby clamps are used to attach the cables to one another (Figure 3.1.D). The spacing of the cable slings to date has been 1.5-2.0 meters; currently, spacing has been determined by practical production parameters and not engineering design.
At anytime during the cable sling installation, bulkheads and fill fences will be built when it is most convenient. For example, a fill fence at the far end of a stope - away from the stope access - will be built before most of the cable slings are installed to prevent carrying materials across the cable slings. Carrying materials across the cable slings is an inconvenience more than a design constraint.
Figure 3.1.C: Grout cartridges installed in bore hole before split set installation.

Figure 3.1.D: Crosby clamps used to anchor strike cables to span cables.
After the cable slings are installed, 10 cm x 10 cm (4 inch x 4 inch) weld wire mesh is laid on the cable slings. The weld wire mesh is attached to the cables with TYCLIP™ (Tannant, 1995, Figure 3.1.E). Once the weld wire mesh is attached to the cable slings, a geotextile is placed. The geotextile is run up the hangingwall and footwall for a distance that is about 1/2 meter above the height of cemented backfill which is poured to a height of 1.2 meters (4-5 feet) above the cables.

![Figure 3.1.E: A TYCLIP™ (Tannant, 1995) attaching the weld wire mesh to a cable sling.](image)

The geotextile is held in place with nails pinning it to a messenger cable (Figure 3.1.F).

Before the cemented backfill is poured, a bulkhead is usually built with a hole to allow for decanting of the cemented backfill (Figure 3.1.G).
Figure 3.1.F: Illustration of geotextile, messenger cable, and nail.

Figure 3.1.G: Water decanting through a bulkhead.
3.2 Cable Sling Testing

Testing of the cable sling system is done to ensure confidence in the installation procedure. Two tests are performed to determine the performance of the cable sling system. First, a pull test is carried out on the anchor system (Figure 3.1.B, section 3.1) which consists of a split set, cable grip, high tensile cable, and grout. This test ensures that the anchor system in the hangingwall and footwall utilizes the cable breaking strength and is not the weak point of sill mat design. Next, a cable deflection test is completed on a cable sling installed across a drift; as a result, a curve illustrating load versus deflection can provide an indication of cable tension when mining under a sill mat. In this manner, predicting the breaking point of a cable sling would enable workers to foresee sill mat failure.

3.2.1 Cable Pull Test

The current sill mat design utilizes the Scott Cable Sling™ System (Figure 3.1.B, section 3.1). The strength of this system was determined; namely, the anchor strength and whether or not the cement grout is utilizing the high tensile cable strength. In other words, is the grout the weakest part of the cable sling system. To determine the above information, a cable pull test was designed using a 535 kN (60 ton) hollow hydraulic jack. The general cable pull test setup is pictured in Figure 3.2.1.A.

Results from the pull test are listed in Table 3.2.1.A. Cables # 1,2,3 are installed with the current cable sling design - 2.4 meter (8 feet) holes, 1.8 meter (6 feet) split set, 0.6 meter (2 feet) of cable past cable grip. Cable #2 broke due to binding with spacer in between the hydraulic jack and wall rock - unnatural influence on cable breaking load. In case the current installation procedure is not utilizing the cable strength, cables #4,5,6 are installed with 2.7 meter (9 feet) holes, 1.8 meter (6 feet) split set, and 0.9 meter (3 feet) of cable past the cable grip. All tests indicate current cable sling anchor design is utilizing full breaking strength of the cable.
Figure 3.2.1.A: Cable pull test for determining anchor strength of cable sling system.

Table 3.2.1.A: Cable pull test breaking loads.

<table>
<thead>
<tr>
<th>Test</th>
<th>Breaking Load</th>
<th>lbs.</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>N</td>
<td></td>
</tr>
<tr>
<td>1</td>
<td>197449</td>
<td>44280</td>
</tr>
<tr>
<td>2</td>
<td>142602</td>
<td>31980</td>
</tr>
<tr>
<td>3</td>
<td>186479</td>
<td>41820</td>
</tr>
<tr>
<td>4</td>
<td>219387</td>
<td>49200</td>
</tr>
<tr>
<td>5</td>
<td>208418</td>
<td>46740</td>
</tr>
<tr>
<td>6</td>
<td>186479</td>
<td>41820</td>
</tr>
</tbody>
</table>
3.2.2 Cable Deflection Test

When exposing a sill mat during mining of a sill pillar, the cable slings will deflect from the load of the backfill above. If the deflection of the cables is monitored (easily done by survey), the tension in the cables can be determined. If the cable tension is known, then the point at which the breaking strength of the cable is being approached is also known. This will aid in the safety of mining beneath a sill mat.

3.2.2.1 Theoretical Cable Tension

The theoretical cable tension is based on the theoretical strain in the cable since loads are not known. The theoretical cable strain is calculated from the geometry of the cable as it is being deflected. The following formula is used to calculate the theoretical cable strain ($\varepsilon$):

$$\varepsilon = \frac{L - a}{TCL}$$

where,

$L = 2 \times \left( \frac{a}{2} \right)^2 + d^2 \right)^{1/2}$

$a = \text{span across stope.}$

$d = \text{deflection.}$

$TCL = \text{total cable length - includes non-grouted cable in split set.}$

Note: use consistent units.

The theoretical load on the cable ($L_{ct}$) can be determined from the strain by the following formula:

$$L_{ct} = E \times A \times \varepsilon$$

where,

$E = \text{Young's Modulus}$

$A = \text{nominal wire cross-sectional area.}$

Note: use consistent units.

Young's Modulus and nominal wire cross-sectional area are supplied by the manufacturer (Table 3.2.2.1.A).
3.2.2.2 Measured Cable Tension

The measured cable tension is determined from a vertical point load used to deflect the cable (Figure 3.2.2.2.A).

![Point load cable deflection geometry.](image amatex)

**Table 3.2.1.A: High tensile cable specifications.**

<table>
<thead>
<tr>
<th>Property</th>
<th>1.27 cm (1/2 inch) diameter</th>
<th>1.60 cm (5/8 inch) diameter</th>
</tr>
</thead>
<tbody>
<tr>
<td>Ultimate Breaking Strength</td>
<td>184,000 N (41,000 lbs.)</td>
<td>258,000 N (58,000 lbs.)</td>
</tr>
<tr>
<td>Young's Modulus</td>
<td>200 Gpa</td>
<td>200 GPa</td>
</tr>
<tr>
<td>Nominal Cross Sectional Area</td>
<td>98.7E-6 m²</td>
<td>140.07E-6 m²</td>
</tr>
</tbody>
</table>
The measured cable load \( (L_{cm}) \) is calculated by:

\[
L_{cm} = \frac{P \cdot h}{2 \cdot d}
\]

where,

\[
P = \text{vertical load.}
\]
\[
h = (d^2 + (\text{span}/2)^2)^{1/2}
\]
\[
d = \text{deflection.}
\]

### 3.2.2.3 Theoretical versus Measured Cable Tension

The theoretical and measured cable tension is illustrated on Figure 3.2.2.3.A.

![Theoretical and measured load versus cable deflection.](image)

**Figure 3.2.2.3.A:** Theoretical and measured load versus cable deflection.

The theoretical curve is a third order polynomial equation due to the geometry of the point load model (Nash, 1978). The measured load is much higher than the theoretical load since the initial seating load of the jackleg tensions the cable immediately; therefore, an increase in cable tension due to installation load places the cable under a higher tension much sooner than theory
suggests (Figure 3.2.2.3.A). The initial installation tension of a cable sling due to the thrust of the jackleg used for the load/deflection test is approximately 2650 N (600 lbs.). 2650 N is subtracted from the measured load to account for the seating load. A simple calculation determines this initial load: the piston area multiplied by the mine air pressure gives the approximate initial load. The piston area of the jackleg used for cable sling installation is 0.0038 m² and the mine air pressure used to estimate the initial load is 689.5 kPa (100 psi). For cable slings in a sill mat, the initial load may be between 2200 N (500 lbs.) and 4450 N (1000 lbs.), depending on the ability of the operator to anchor the jackleg on the sill. Usually, two workmen are required to install cable slings.

One characteristic of the measured tension line is the rate of increase in the load at approximately 0.07 meters deflection. The increase in load can be explained by beam theory. A small load is required to deflect a horizontal beam. However, once the beam is deflected, the vertical component of the force vector is increased. As a result, initial tension of a horizontal cable sling is small. The cable continues to deflect at a certain rate until it reaches a critical point (i.e. 0.07 meters in the above test). For geometric reasons, the cable accepts more tension once it reaches the critical deflection point. Once the jump in tension occurs, the deflection resumes at a more consistent rate.

The cable broke because the split set dolly (Figure 3.2.2.3.B) was stuck in the split set causing the cable sling to bind at this point. Premature cable failure resulted from the split set dolly binding the cable sling. After this point in the test, the cable load increases at a higher rate with less deflection. This is expected since there is less cable supporting the load - a result of the broken strand.
Figure 3.2.2.3.B: Split set dolly used to install split sets with a jackleg. This photo shows the split set dolly wedged in the cable sling causing premature cable failure.

With better control on testing conditions, the determination of the cable load in a sill mat could be estimated from visible observations and survey data. Consequently, when the breaking strength of sill mat cables is being approached, safeguards to ensure continued mining beneath a sill mat can be implemented.
4.0 Backfill Properties

Waste rock from underground mining averaging -15 centimeters (6 inches) passing is used for backfill. In addition, mill backfill production is derived from two sources, namely, cyclone classification of flotation tailings and grinding of mine development waste. The mill feed hydraulic backfill is pumped at 60% solids to underground stopes. Hydraulic backfill typically accounts for approximately 65% of backfill and waste rock approximately 35% of backfill placed in mined out stopes. These values fluctuate according to requirements and constraints at the time of stope filling. If needed, hydraulic fill is derived by crushing and grinding mine development waste during periods of high backfill demand. Typically 32% by weight of the flotation tailings stream is recovered as hydraulic backfill via particle separation from a two stage cyclone circuit.

4.1 Size Distribution

Sieve analysis performed on backfill results in size distribution curves illustrated in Figure 4.1.A. The average 80% passing for the eight samples is 115 microns. A coarser product may add strength to the backfill (CANMET, 1990). Rockfill being used in some cases will improve the strength of backfill. The shape of the size distribution curves is consistent for all samples. The drawn out S shape indicates a reasonable well graded backfill size distribution; well graded particles lend themselves to improved strength. No relationship between the size distribution and the percolation rate of the samples is clear. The average percolation rate of the samples is 117 mm/hr.
4.2 Solids Density, Wet Bulk Density, and % Moisture

Backfill specific gravity averages 2.70 - 2.80. The wet bulk specific gravity of recently placed drained backfill is between 1.4 and 1.5. In addition, direct shear testing (section 4.3) samples are run at specific gravity's between 1.38 and 1.70, with an average specific gravity of 1.53 and a standard deviation of 0.05 (Wallace, 1996). As a consequence of the influx of water flushing through the backfill from subsequent overhand backfill lifts and the added compressive weight of overlying backfill, the density of the backfill should increase until some maximum is reached - possibly a maximum specific gravity of 1.7 that is achieved in the direct shear tests.

The moisture content is determined from fresh samples taken from a drained backfilled stope (Stope 150 - 300). The backfill was fully drained at the sample location and resulted in an average of 13.4 % moisture in the samples. This value compares favorably to the Moisture Content / Cement content graph (Figure 5.2.2.C, Section 5.2.2). The graph illustrates a moisture value between 10 and 15% for samples with low cement contents. Considering
moisture content, clearly, the cemented backfill laboratory samples taken from the 2656 cement backfill pour compare well to the noncemented backfill taken from the 300-150 stope.

Moisture Content is defined as:

\[ M \text{ (\%)} = \frac{\text{weight H}_2\text{O in moist sample}}{\text{weight moist sample}}. \]

### 4.3 Direct Shear Test

Direct shear tests on oxidized backfill (31 samples) indicates an average friction angle of 34.1 degrees and apparent cohesion of approximately 2.0 kPa. Direct shear tests on unoxidized backfill (5 samples) indicates an average friction angle of 31 degrees and apparent cohesion of approximately 3.0 kPa. Figure 4.3.A is a plot of all shear samples. Since oxidized backfill is the most likely insitu condition due to moisture underground and high porosity (51%), the insitu properties are estimated as 34.1 degrees friction angle and 2.0 kPa apparent cohesion. True cohesion is from clay particles; since the size distribution is coarse and chemical mineralogy does not indicate a high clay content, the apparent cohesion is due to negative pore water pressure (Wallace, 1996).
4.4 Predicted In situ Backfill Properties

The predicted values for in situ backfill properties based on testing are estimated as:

<table>
<thead>
<tr>
<th>Property</th>
<th>Value</th>
<th>#samples</th>
</tr>
</thead>
<tbody>
<tr>
<td>Size Distribution</td>
<td>Well graded</td>
<td>8</td>
</tr>
<tr>
<td>P80</td>
<td>115 microns</td>
<td>8</td>
</tr>
<tr>
<td>Percolation Rate</td>
<td>117 mm/hr</td>
<td>8</td>
</tr>
<tr>
<td>Bulk Density</td>
<td>1.4 - 1.8</td>
<td>40</td>
</tr>
<tr>
<td>Solids Density</td>
<td>2.7 - 2.8</td>
<td>6</td>
</tr>
<tr>
<td>% Moisture</td>
<td>10 - 15 %</td>
<td>6</td>
</tr>
<tr>
<td>Friction Angle</td>
<td>34°</td>
<td>31</td>
</tr>
<tr>
<td>Cohesive Strength</td>
<td>2.0 kPa</td>
<td>31</td>
</tr>
<tr>
<td>Porosity (solids density = 2.75)</td>
<td>0.51</td>
<td>40</td>
</tr>
</tbody>
</table>

Figure 4.3.A: Direct shear tests (Wallace, 1996).
5.0 Cemented Backfill Properties

Samples were taken during a cemented backfill pour for the 2656 sill mat. Figure 5.0.A shows the location of the 2656 sill mat in relation the other stopes in the 150 Zone. Tests were performed on the samples in order to determine backfill properties; in addition, insight into characteristics and behaviour of cemented backfill during cemented backfill pours for a sill mat was gained. Furthermore, the amount of cement being retained in the sill mat can be estimated.

![Figure 5.0.A: Mine longitudinal of the 150 Zone with the 2656 sill mat.](image-url)
5.1 Decanting and % Solids to 2656 Sill Mat

The mill feed backfill to the 2656 cemented backfill pour averaged 60% solids and lasted for 5 3/4 hours. At a feed rate of 31m$^3$/hr, approximately 90 cubic meters of material was delivered to the sill mat with 30 cubic meters of the material lying below the cable slings. This material acts as a blast cushion during mining of a sill pillar. The volume of solids below the support cables resulted from holes in the geotextile. The geotextile is a continuous filament, needle-punched, nonwoven polypropylene material (Layfield Plastics, 1993). The material is very robust and strong (Table 5.1.A); however, holes a few centimeters in diameter allow the backfill material at 60% solids to flow through with little resistance.

<table>
<thead>
<tr>
<th>Property</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Grab Tensile</td>
<td>936 N</td>
</tr>
<tr>
<td>Elongation</td>
<td>50 %</td>
</tr>
<tr>
<td>Trapezoidal Tear</td>
<td>379 N</td>
</tr>
<tr>
<td>Puncture Strength</td>
<td>446 N</td>
</tr>
<tr>
<td>Mullen Burst</td>
<td>2758 kPa</td>
</tr>
<tr>
<td>Opening Size (No. 70 US Sieve)</td>
<td>210 micro meters</td>
</tr>
<tr>
<td>Permittivity</td>
<td>1.3 1/s</td>
</tr>
<tr>
<td>Permeability (k)</td>
<td>0.3 cm/s</td>
</tr>
<tr>
<td>Weight</td>
<td>285 g/m$^2$</td>
</tr>
<tr>
<td>Thickness</td>
<td>105 mm</td>
</tr>
</tbody>
</table>

During the cemented backfill pour, a vortex (drainage pool) formed in the backfill at three different locations - these were evidence of leaks in the geotextile. Drainage pools may have occurred in other locations but not with the strong pool characteristics of other locations.

During the decanting of cemented backfill, clear water drains off the top first (Figure 3.1.G, section 3.1). This is followed by clouded water containing the cement. Finally, if allowed to decant further, cemented backfill material will flow over the decant fence. In general, decant water over the fence is clear for approximately 5 minutes. At 10 minutes, once finer particles (cemented and fine backfill) have been decanted, another board is added to the decant fence.
Water flow was also decanted through 2 “Big O” (Figure 1, Introduction) pipes and an internal decant tower which will decant backfill on subsequent cut and fill lifts above. The “Big O” discharge was slightly cloudier than the overflow of the decant boards at the fence and the internal decant tower; clearly, the “Big O” pipes are performing adequately.

5.2 Cemented Backfill Properties

During the cemented backfill pour, the mill feed backfill, cemented backfill, and the water bleeding from the cemented backfill pour are sampled to determine the cement loss from the cemented backfill pour. The cemented backfill is tested for Cement Content, Uniaxial Compressive Strength, Young’s Modulus, Dry and Wet Density, and Percent Moisture.

5.2.1 Cement Content in Samples

Samples for cement content testing are taken during the cemented backfill pours of sill mats 2656 and 3049. The samples taken during the 2656 sill mat pour (Table 5.2.1.A) were taken from the east end within the sill mat (Samples #1-5), one from the drift outside the bulkhead (Sample #10), decant material from the west side of the sill mat (i.e. at the bulkhead, Samples #14-19), and mill slurry (Samples #11-13) and mill water (Samples #23-25). Mill slurry and water samples are taken before cement is added. The location of each of these samples is noted on Figure 5.2.1.A. After two and half months of setting, a hole was cut in the bulkhead of 2656 for samples (Samples #7-9). Sample 6 is a completely dry sample taken in 1991 during the 3049 sill mat pour (Figure 3.0.A).
### Table 5.2.1.A: Cemented backfill assay values.

<table>
<thead>
<tr>
<th>Sample</th>
<th>I.D. #</th>
<th>I.D. Name</th>
<th>CaO%</th>
<th>Cement%</th>
<th>Corrected</th>
<th>background</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>1</td>
<td>B/F #1</td>
<td>9.6</td>
<td>14.8</td>
<td>0.0</td>
<td>14.9</td>
</tr>
<tr>
<td>2</td>
<td>2</td>
<td>B/F #6</td>
<td>11.9</td>
<td>18.7</td>
<td>3.8</td>
<td>14.9</td>
</tr>
<tr>
<td>3</td>
<td>3</td>
<td>B/F #3</td>
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<td>15.9</td>
<td>1.0</td>
<td>14.9</td>
</tr>
<tr>
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<td>4</td>
<td>B/F #4</td>
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<td>5</td>
<td>B/F #5</td>
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<td>21.1</td>
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<td>6</td>
<td>6&quot; 3049</td>
<td></td>
<td>17.5</td>
<td>27.3</td>
<td>12.4</td>
<td>14.9</td>
</tr>
<tr>
<td>7</td>
<td>11</td>
<td>Bulkhead Below</td>
<td>19.3</td>
<td>30.3</td>
<td>15.4</td>
<td>14.9</td>
</tr>
<tr>
<td>8</td>
<td>12</td>
<td>Bulkhead Above</td>
<td>22.4</td>
<td>35.2</td>
<td>20.3</td>
<td>14.9</td>
</tr>
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<td>14.9</td>
</tr>
<tr>
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<td>20</td>
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</tr>
<tr>
<td>12</td>
<td>21</td>
<td>M #5</td>
<td>9.4</td>
<td>14.8</td>
<td>0.0</td>
<td>14.9</td>
</tr>
<tr>
<td>13</td>
<td>22</td>
<td>M #4</td>
<td>9.7</td>
<td>15.3</td>
<td>0.0</td>
<td>14.9</td>
</tr>
<tr>
<td>14</td>
<td>23</td>
<td>B/W #2</td>
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<td>39.5</td>
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</tr>
<tr>
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<td>24</td>
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<td>25</td>
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<td>35.3</td>
<td>20.4</td>
<td>14.9</td>
</tr>
<tr>
<td>17</td>
<td>26</td>
<td>B/W #1</td>
<td>23.8</td>
<td>37.4</td>
<td>22.5</td>
<td>14.9</td>
</tr>
<tr>
<td>18</td>
<td>27</td>
<td>B/W #6</td>
<td>20.1</td>
<td>31.7</td>
<td>16.8</td>
<td>14.9</td>
</tr>
<tr>
<td>19</td>
<td>28</td>
<td>B/W #5</td>
<td>21.7</td>
<td>34.2</td>
<td>19.3</td>
<td>14.9</td>
</tr>
<tr>
<td>20</td>
<td>29</td>
<td>Nov. 29</td>
<td>12.8</td>
<td>20.1</td>
<td>5.2</td>
<td>14.9</td>
</tr>
<tr>
<td>21</td>
<td></td>
<td>Backfill</td>
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<td>14.9</td>
</tr>
<tr>
<td>22</td>
<td></td>
<td>Bondar Clegg Assay</td>
<td>9.0</td>
<td></td>
<td></td>
<td>14.9</td>
</tr>
</tbody>
</table>

*** mill slurry solids to GEHO pump assay 14.9% cement on day of pour, this is used for background! average of Samples M #4,5,6.

---

**Figure 5.2.1.A:** Cemented backfill sample locations as taken during the 2656 cemented backfill pour (long section).
The mill slurry samples (#11,12,13) taken the day of the 2656 sill mat pour assay CaO at 9-10% - this gives an average background cement reading of 14.9% cement. In other words, CaO from the ore and mill refining process gives a background reading of cement even though no cement has been added. This value (14.9%) is subtracted from all assay values to obtain a corrected cement content. Sample 21 (only backfill) and sample 22 (independent assay by Bondar Clegg) support a background CaO level of about 9%.

Samples 7 and 8 taken from the bulkhead hole show a cement content of 15.4 and 20.3 percent respectively. Sample 7 is taken just below the geotextile and weld wire mesh, and sample 8 is taken just above the geotextile and weld wire mesh. Sample 7 is part of the blast cushion and sample 8 is part of the sill mat which has to remain in place once the sill pillar is mined. Sample 9 is taken from inside a piece of geotextile hanging just outside the bulkhead at about 6 feet from the sill. Similar to Sample 8, it has a cement content of 20.8 percent cement. Comparing samples 8 and 9 (both within the geotextile) to sample 7 (beneath the geotextile) - both 8 and 9 have higher cement values. Sample 7 may have been lower if cemented backfill had not leaked through the geotextile during the cemented backfill pour. Evidently, the geotextile may be providing a good barrier for preventing cement flow out of a sill mat.

Samples taken at the bulkhead (Samples #7-9, 14-19) and the sample (#10) taken near the bulkhead in the drift all show a concentration of cement ranging in value from 15 to 20 percent. The samples taken during the pour from within the sill mat (Samples #1-5) range in value from 0 to 6.2% (cemented backfill is pumped in at 7.6% (12:1) cement for this pour). Clearly, cement is accumulating near the bulkhead. In other words, the cement concentrates in the direction of drainage and away from the pour zone. This could be a result of the cement (high CaO content) having a low density compared to backfill (high metal sulphide content). Upon settling, the backfill acts like a dense medium and the cement floats. In much the same manner as a concentration table separates the dense particles from light particles in a fluid medium, the flow action of the cemented backfill at 40% water can easily separate the cement from the backfill.
Further evidence for this washing of cement from the backfill is illustrated in Figure 5.2.1.B. The cement values of the sill mat (Samples #1-5) and the bulkhead (Samples #14-19) samples are plotted against time at which they were taken during the pour. The lower line on the figure is the sill mat sampling and the top line is the bulkhead sampling. For the first 2 1/2 hours the cement in the sill mat samples increased and the cement in the bulkhead overflow decreased. The final sample at the bulkhead was taken after a longer period of decanting; as a result, it may have been skewed to a higher cement content. If this is the case, then the cement content at the bulkhead consistently decreased with time indicating more cement remaining in the sill mat as the pour continues.

At the start of the pour the slurry in the stope is allowed to flow freely in the stope and drain towards the bulkhead (Figure 2.1.C); thus, cement washed out of the sill mat occurs more freely at the start of the pour and can accumulate at the bulkhead. As more backfill accumulates in the stope, travel paths for decanting become more restricted; consequently, cement is more likely to settle out and remain in the stope.
Starting the pour at the higher elevation at the west end of the 2656 stope allowed water and cement to flow freely to the east end (bulkhead). A completely sealed geotextile (no holes) may alleviate some of this problem; however, a blast cushion is still required for sill mat design and should be incorporated in all sill mats. As a result, 15 centimeter holes in the geotextile - similar to those that were present in the 2656 sill mat - cut at 10 meter intervals should allow for a blast cushion to form beneath the support cables.

Cement must be inhibited from being washed from a cemented backfill pour. A solution to this problem may be to start the cemented backfill pour at the lowest point of the sill mat. In the case of the lower 2656 sill mat, the lowest point would be at the bulkhead. Starting the pour at the bulkhead would build up the sill mat at the bulkhead - this is also the location of the decanting of water. Once the level of the sill mat reach the top of a decant board, the pour should shift to a point further away from the decant location. This will result in a more restricted travel path for decanting water; thus, cement will have more opportunity to settle in the sill mat as previous evidence suggests.
This method of pour can be achieved without adversely affecting production. Victaulic T couplings with an Open/Close valve could be installed at the backfill pipe joints. This does not increase the installation labour since couplings are installed at pipe joints - just a different coupling must be installed. The 2656 sill mat cemented backfill pour could have had two T couplings installed resulting in 3 pour locations (Figure 5.2.1.D). One pour location near the bulkhead, one pour location in the middle of the stope. Each one of these locations having a T coupling. The third pour location would be the end of the pipe when both T couplings are shut off. During the cemented backfill pour, the backfill worker in the stope can easily walk along the cat walk to open or shut the valves as required.

![Diagram of proposed T coupling locations for a sill mat pour (long section).]

Evidence to date suggests the method for cemented backfill pours at Snip Mine are working adequately. Having to pump cemented backfill at 60% solids results in difficult control over cement placement in a sill mat. However, minor changes to the pour method as suggested above may increase the amount of cement retained in a sill mat.
5.2.2 Uniaxial Compressive Strength and Young’s Modulus

During the 2656 cemented backfill pour, five samples were taken from the sill mat (Figure 5.2.1.A) and subsequently used for uniaxial compressive testing. An additional sample taken from the stope access just outside the bulkhead was tested. An older, completely dry sample was also tested from sill mat 3049. These samples correspond to samples #1-6, and 10 (Table 5.2.1.A). The samples ranged in length from 15 to 25 centimeters (6-10 inches) and had a diameter of 15 cm (6 inches). The samples were stored in plastic bags with a couple of inches of water - this moisture kept the samples wet to mimic underground conditions. After 3 months of curing time, the samples were prepared for uniaxial compressive testing. Due to the weak nature of the samples care was taken not to bump or move the samples excessively.

In a manner similar to concrete sample preparation, sulfur caps were placed on the ends of the samples. The sulfur caps ensure a flat surface for uniaxial testing. In order to place a sulfur cap on each end of the samples, compressed air (<200 kPa) was gently applied to each end of the samples - this help dry the surface. A small coating of oil was applied to the end surface after the compressed air. The melted sulfur proved to be hydrophobic; thus, the thin coating of oil provided an excellent bond surface for the sulfur. Once sulfur caps were on each end of the specimens, they were tested immediately. Promptness of this entire process ensured the moisture remained in the samples for uniaxial compressive testing. Sulfur capping and uniaxial compressive testing of the seven specimens was completed over 3 days. Samples not tested after sulfur capping were placed back in moist plastic bags to maintain water content.
A sample stress strain curve is illustrated in Figure 5.2.2.A. The maximum uniaxial compressive strength of each sample is listed in Table 5.2.2.A.

![Typical stress/strain curve for cemented backfill UCS test.](image)

**Figure 5.2.2.A:** Typical stress/strain curve for cemented backfill UCS test.

<table>
<thead>
<tr>
<th></th>
<th>UCS (kPa)</th>
<th>%cement</th>
</tr>
</thead>
<tbody>
<tr>
<td>B/F 1</td>
<td>309</td>
<td>0</td>
</tr>
<tr>
<td>B/F 3</td>
<td>217</td>
<td>1</td>
</tr>
<tr>
<td>B/F 4</td>
<td>438</td>
<td>5.7</td>
</tr>
<tr>
<td>B/F 5</td>
<td>185</td>
<td>6.2</td>
</tr>
<tr>
<td>B/F 6</td>
<td>356</td>
<td>3.8</td>
</tr>
<tr>
<td>Drift</td>
<td>109</td>
<td>20.4</td>
</tr>
<tr>
<td>3049</td>
<td>877</td>
<td>12.4</td>
</tr>
</tbody>
</table>

**Table 5.2.2.A:** Maximum uniaxial compressive strength of cemented backfill samples from 2656 sill mat.

The average maximum uniaxial compressive strength for the samples taken from the 2656 sill mat is 300 kPa; in comparison, the completely dry sample from the 3049 sill mat has a strength nearly three times that - 877 kPa. However, the 2656 cement backfill was poured at 12:1 (backfill:cement by weight) and the 3049 was poured at 6:1. The drift sample is extremely weak (109 kPa) and behaves more plastically as the stress/strain curve illustrates (Figure 5.2.2.B). In all likelihood, this is due to the moisture content (section 5.2.3) increasing the pore water pressure during compression. No correlation is found between cement content and uniaxial...
uniaxial compressive strength; however, the number of samples tested is small. A correlation may become evident for a larger sample population.

The modulus of elasticity (E) for the uniaxial compressive samples is presented in Table 5.2.2.B. Swan (1985) suggests the following empirical formula for calculating E:

\[
E = 0.21 \text{UCS}^{1.44}
\]

where,

- \( E \) = modulus of elasticity ( Gpa ),
- \( \text{UCS} \) = uniaxial compressive strength ( MPa ).

From Table 5.2.2.B, this equation provides a reasonable estimate for E and is used to compare results to the UCS tests.
Table 5.2.2.B: Modulus of Elasticity for the cemented backfill samples.

<table>
<thead>
<tr>
<th></th>
<th>Lab test</th>
<th>Swan</th>
<th>Lab test</th>
<th>Swan</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Str. Max./Strain</td>
<td></td>
<td>Linear</td>
<td>Canmet</td>
</tr>
<tr>
<td>B/F 1</td>
<td>0.015 GPa</td>
<td>0.039 GPa</td>
<td>0.039 GPa</td>
<td>0.008 GPa</td>
</tr>
<tr>
<td>B/F 3</td>
<td>0.013 GPa</td>
<td>0.023 GPa</td>
<td>0.052 GPa</td>
<td>0.009 GPa</td>
</tr>
<tr>
<td>B/F 4</td>
<td>0.041 GPa</td>
<td>0.064 GPa</td>
<td>0.101 GPa</td>
<td>0.024 GPa</td>
</tr>
<tr>
<td>B/F 5</td>
<td>0.008 GPa</td>
<td>0.018 GPa</td>
<td>0.026 GPa</td>
<td>0.004 GPa</td>
</tr>
<tr>
<td>B/F 6</td>
<td>0.030 GPa</td>
<td>0.047 GPa</td>
<td>0.108 GPa</td>
<td>0.026 GPa</td>
</tr>
<tr>
<td>Average</td>
<td>0.018 GPa</td>
<td>0.037 GPa</td>
<td>0.063 GPa</td>
<td>0.013 GPa</td>
</tr>
<tr>
<td>hi/lo out</td>
<td>0.018 GPa</td>
<td>0.036 GPa</td>
<td>0.065 GPa</td>
<td>0.014 GPa</td>
</tr>
<tr>
<td>Drift 1st</td>
<td>0.002 GPa</td>
<td>0.009 GPa</td>
<td>0.015 GPa</td>
<td>0.007 GPa</td>
</tr>
<tr>
<td>3049</td>
<td>0.191 GPa</td>
<td>0.174 GPa</td>
<td>0.191 GPa</td>
<td>0.174 GPa</td>
</tr>
</tbody>
</table>

The four columns in Table 5.2.2.B give the elastic modulus in four ways:

Column 1: \( \sigma_{\text{max}}/\varepsilon \) Strain (\( \varepsilon \)) at maximum stress (\( \sigma_{\text{max}} \)). This generates an elastic modulus characterizing the property over the entire range of stress - an averaging effect.

Column 2: Swan equation using the maximum stress (\( \sigma_{\text{max}} \)).

Column 3: \( \sigma_{\text{linear}}/\varepsilon \) Strain (\( \varepsilon \)) at maximum linear stress (\( \sigma_{\text{linear}} \)). The maximum linear stress is defined from the first part of the stress/strain curves (i.e. Figure 5.2.2.A) which generally appear linear. The linear portion of the stress/strain curves continue to an average UCS strength of 150 kPa for the 2656 sill mat samples. Clearly, at approximately half the average maximum uniaxial compressive strength the cemented backfill properties change to a weaker material. At this point cement bonds are broken and the cemented backfill enters the plastic zone from the elastic region - irreversible damage. After this point, as more stress is applied, the modulus of elasticity gradually decreases until it reaches zero at the maximum uniaxial compressive stress.

Column 4: Swan equation using the maximum linear stress (\( \sigma_{\text{linear}} \)).

For the 2656 sill mat samples, the average modulus of elasticity is 0.018 GPa and 0.065 GPa for the maximum stress and linear stress portions of the stress/strain curves. The drift sample is much lower at 0.002 GPa (maximum stress) and 0.015 GPa (linear) due to its lower strength and plastic behaviour.
5.2.3 Dry and Wet Density, Moisture Content, Porosity

The dry and wet density, moisture content, and porosity are determined from the samples in Table 5.2.1.A. Samples #1-6, 10, and a dry 10 centimeter sample from the 3049 sill mat pour are cylinders similar to those used in concrete testing (16 centimeter diameter). The dry and wet density, as well as the moisture content are calculated from the wet UCS samples (Table 5.2.1.A, Samples - #1-5,10) and the wet bulkhead samples (Table 5.2.1.A, Samples - #7-9).

The dry density is calculated for 2 dry samples from the 3049 sill mat pour (Table 5.2.1.A - #6).

The wet density formula (Samples #1-5,10) and the dry density formula for samples from the 3049 pour is:

\[
\rho_{\text{wet}} = \frac{W_c}{V_c}
\]

where,

\[
W_c = \text{UCS cylinder weight}
\]
\[
V_c = \text{cylinder volume, } (\pi r^2 h).
\]

The dry density of the wet UCS samples is calculated as follows:

\[
\rho_{\text{dry}} = \frac{W_c (100 - \%H_2O)}{V_c}
\]

where,

\[
\%H_2O = \text{moisture content}.
\]

The moisture content is determined from any sample size by:

\[
\%H_2O = \frac{(W_{\text{wet}} - W_{\text{dry}})}{W_{\text{wet}}}
\]

Porosity is defined as:

\[
\eta = 1 - \left(\frac{\rho_{\text{dry}}}{\rho_{\text{solid}}}\right)
\]

where,

\[
\rho_{\text{solid}} = 2750 \text{ kg/m}^3 \text{ for Snip Backfill.}
\]

The average wet density of the UCS samples (cemented backfill cylinders) is 2180 kg/m³ at an average moisture content of 16.9% moisture. Visual observations of the sill mat 12 - 24 hours after completion of the cemented backfill pour suggests 15 to 20% moisture for the majority of the cemented backfill in the sill mat. Figure 5.2.3.A illustrates a linear relationship between cement and moisture content. The moisture content of samples having little or no cement falls between 10% and 15%.
Data from 6 samples having cement contents from 9 to 14 % cement were added to the backfill samples to obtain Figure 5.2.3.A. The 6 samples were stored in plastic bags and cured for 1 week before testing the cement content. The graph compares favorably with plain backfill moistures of 13.4% (section 4.4). Furthermore, an increase in cement content increases the amount of water in cemented backfill. The insitu moisture content may be higher, in areas with concentrated cement, as indicated by the bulkhead samples (30.5% moisture). An average porosity of 34.1% supports the possibility of higher insitu moisture content for cemented backfill. If the moisture content is as high as 30% for insitu cemented backfill, then the wet density could be as high as 2400 kg/m$^3$ (CANMET, 1990). The average dry density for the 2656 sill mat samples is 1813 kg/m$^3$, similar to weak porous sedimentary rocks.

Figure 5.2.3.A: Moisture content versus cement content.
5.2.4 Predicted Insitu Cemented Backfill Properties

An estimate of insitu cemented backfill properties from testing and field observations are suggested as follows:

Table 5.2.4.A: Predicted insitu cemented backfill properties.

<table>
<thead>
<tr>
<th>Property</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>1. Unconfined Compressive Strength (UCS)</td>
<td>300 kPa</td>
</tr>
<tr>
<td>2. Cohesion, C (1/15 UCS, Singh (1976))</td>
<td>20 kPa</td>
</tr>
<tr>
<td>3. Elastic Modulus</td>
<td>0.065 GPa</td>
</tr>
<tr>
<td>4. Moisture Content</td>
<td>15-30 %</td>
</tr>
<tr>
<td>5. Wet Bulk Specific Gravity</td>
<td>2.2-2.4</td>
</tr>
<tr>
<td>6. Dry Bulk Specific Gravity</td>
<td>1.6-1.8</td>
</tr>
<tr>
<td>7. Porosity</td>
<td>34 %</td>
</tr>
</tbody>
</table>

The Uniaxial Compressive Strength (UCS), wet bulk density, porosity, and elastic modulus are estimated from the average of samples 1 to 5 (Table 5.2.1.A). The cohesion of the cemented backfill is estimated as 1/15 the UCS value (Singh, 1976). Referring to Figure 5.2.2.C; clearly, areas in the 2656 sill mat containing a concentrated cement (i.e. near the bulkhead) may have a high moisture content. Areas containing cement near or below the 12:1 (7.6%) pour ratio should have a moisture content between 15 and 20%. Similarly, the moisture content of noncemented backfill was found to be 13.4% (section 4.4). The above cemented backfill properties compare favorably to those estimates given in CANMET (1990).
6.0 Instrumentation

The following instrumentation was chosen for installation and monitoring of sill mats at the Snip mine:

- Flat Jack (Pressure Cell)
- Vibrating Wire Piezometer
- Tense Meter
- Extensometer
- Geophone.

The pressure cell is a vibrating wire pressure transducer surrounded by fluid encased in two metal plates. A force on the plates induces a pressure in the fluid which changes the frequency of a vibrating wire. The frequency of the vibrating wire is calibrated to a fluid pressure. The purpose of the flat jack is to measure pressure exerted by backfill on cemented backfill. Also, a flat jack will be used to measure the pressure from the cemented backfill exerted on the cable slings. From this pressure, the load on the cable slings will be determined and compared to theory and tense meter data.

The piezometer is also a vibrating wire pressure transducer which is placed in the ground to measure water pressure. The piezometer is installed just above the cemented backfill to measure water pressure of the various stages of backfilling and mining.

The tense meter is a vibrating wire strain gauge installed on a cable to measure the strain on a cable. This strain can be related to the load (tension) on the cable.

The extensometer is a modified potentiometer normally used to read temperature - modified by CANMET. In this case, the modification uses the potentiometer to measure movement; specifically, the deflection of a span cable during backfill pouring will be measured.

The geophone is used to monitor for the Peak Particle Velocity (PPV) during a production blast. As mining advances towards a sill mat, a geophone installed within the sill mat will pick up the PPV of shockwaves due to a blast. Three geophones were installed oriented with respect to strike, span, and vertical directions.
6.1 Stope 4847 Instrumentation

A flat jack was installed below the cables in the 4867 west sill mat (Figure 3.0.A). The sill mat in 4867 has a span of 10 meters at the location of the flat jack.

6.2 Stope 2656 Instrumentation

Instrumentation for the 2656 stope (Figure 5.0.A) consists of the following:

- Flat Jack
- Piezometer
- Tense Meter
- Extensometer
- Geophone.

Installation of the extensometer and tense meter is seen in Figure 6.2.A. The location of each of these instruments relative to the sill mat construction materials is illustrated in Figure 6.2.B. The extensometer is installed on the center of the span cable (2.4 meters span) and the tense meter is installed on this same cable. The piezometer is installed near the hangingwall just above the cemented backfill. Flat jack #1 is installed on the weld wire mesh with flat jack #2 installed on top of the cemented backfill.

Figure 6.2.A: Extensometer and tense meter installation.
6.2.1 Instrumentation Data During 2656 Cemented Backfill Pour

The cemented backfill pour started at approximately 1:00 PM and ended at approximately 6:30 PM on the same day. The vertical pressure, cable tension, and cable deflection was determined from instrumentation and visual observations. Data from two pressure cells, Flat Jacks #1 and 2, measure the vertical pressure. An extensometer and a tense meter are installed on a cable sling installed along the span. Data from the instrumentation are stored in a CANMET PL-800 data logger (CANMET, 1995). Upon review of the data logger data, it was determined that the pressure cell (Flat Jack 1 and 2) and the piezometer data were erratic and unreliable. The frequency range of the vibrating wires may be out of the reading range of the data logger and/or the metal connectors used to connect the instrumentation to the data logger are causing short circuits because of moisture. The extensometer and tense meter produced data that are verifiable.

6.2.1.1 Vertical Pressure

The vertical pressure measurements from Flat Jack#1 are very erratic due to problems discussed previously. Also, the readings are substantially higher than what is predicted from above and in
some cases the readings exceed the pressure cell range of 6.895 kPa (100 psi); as a result, the data are unreliable.

During a backfill pour, vertical pressure is estimated from the following:

\[ P_v = \rho \times g \times H \]

where,

- \( \rho \) = density of cemented backfill
- \( g \) = gravity
- \( H \) = height of fill.

The above formula is used in conjunction with pressure readings from a pressure cell in the 4867 stope. The saturated backfill density is calculated as 2054 kg/m\(^3\) using the 68.3 kPa (9.9 psi) pressure reading from 4867 pressure cell - 3 meters saturated backfill and 0.55 meters drained backfill.

\[ 68,300 = \rho_{sat} (9.81)(3) + (1450)(9.81)(0.55) \]

Another approach uses the backfill density (1450 kg/m\(^3\)) and assumes the voids (porosity = 34%) are filled with water. A saturated density of 1943 kg/m\(^3\) is obtained. These estimates are reasonable; therefore, the pressure cell in 4867 stope is deemed to be accurate.

### 6.2.1.2 Cable Tension

The change in cable tension from the installed tension as indicated by the tense meter is illustrated in Figure 6.2.1.2.A. The total change in cable tension from the initial reading (12:00 PM) to the final reading (08:00 AM) is 36.3 kN (8,000 lbs). The cable tension increases at 1:45 PM. This rise in tension corresponds to a higher rate of increase in the deflection of the cable (Figure 6.2.1.2.B). A scatter plot of the change in tension versus the deflection is seen in Figure 6.2.1.2.C. The data is on the low end of the Tension/Deflection Graph (section 3.1) and an initial deflection due to initial heave of 7.6 centimeters (3 inches) causes a gap in data; as a result, a reasonable trend line cannot be drawn.

The tension in the cable rises until 3:15 PM. At this time, the cemented backfill leaking through the geotextile must be at a level that begins to support the material above the cables. This is 2
hours after the start of cemented backfill pour - more than enough volume of solids to fill the void below the cables (i.e. 2 hr. pour time @ 31 m$^3$ slurry per hour = 34 m$^3$ solids, volume below cables = 30 m$^3$). The support effect of the material below the cables results in a lower rate of increasing cable tension until a maximum change in cable tension of 36.3 kN is reached (Figure 6.2.1.2.A). This maximum cable tension is reached at the approximate time cemented backfill pour to the sill mat ceased - 6:30 PM.

![Figure 6.2.1.2.A: Cable tension versus time during the 2656 cemented backfill pour.](image)
Figure 6.2.1.2.B: Deflection versus time during the 2656 cemented backfill pour.

Figure 6.2.1.2.C: Cable tension versus deflection during the 2656 cemented backfill pour.
6.2.1.3 Cable Deflection

An initial deflection of 9.1 centimeters (3.6 inches) was measured during the early stages of the backfill pour by a rope attached to one of the cable slings and suspended vertically from the back of the 2656 stope. In addition, the extensometer measured 9 centimeters (3 1/2 inches) of movement at 1:45 PM (Figure 6.2.1.2.B) - this was a half hour after cemented backfill started reaching the 2656 sill mat. This movement was expected since an initial deflection in the cable sling system was observed during the cable sling deflection test (Figure 3.2.2.3.A, section 3.2.2.3). Evidence is beginning to point to an initial light seating load that causes early deflection in the cable sling system. This initial load does not tension the cable significantly; however, an inherent initial slack seems to be in the cable sling system in addition to a small load being able to deflect a horizontal member or cable.

The deflection readings taken at 3:00 and 3:15 PM may be erroneous data. These data points seem to suggest the cable being returned to its initial starting position; unlikely, unless the support of saturated fill below the cables exerts enough force to push up the cables and cemented backfill on top of the cables and geotextile. The maximum deflection of the extensometer is reached at 7:30 PM - one hour after pouring was stopped. This may be a result of compression and settling of the material below the cables. Further deflection in the cable is out of range of the extensometer.
6.3 Instrumentation Data For Subsequent Cut and Fill Backfills

After curing for 30 days, 6.1 meters (vertical) of hydraulic backfill was placed above Flat Jack #2 - above the cemented backfill. After the 2656 stope was allowed to dewater for 30 days, pressure readings were read manually (Rocktest vibrating wire readout) and resulted in the measurements listed in Table 6.3.A. Backfill was placed on the next lift bringing the total vertical height of backfill to 8.2 meters above Flat Jack #2. Instrumentation readings were taken 14 days later. More backfill was placed over the next few months as mining continued in the 2656 stope. When the backfill elevation reached 18.4 meters above Flat Jack #2, the instrumentation was read with an RS Technical digital readout unit. Three different vibrating wire readout units were used to

Table 6.3.A: Instrumentation readings for subsequent backfill levels.

<table>
<thead>
<tr>
<th>Date</th>
<th>Flat Jack #1</th>
<th>Flat Jack #2</th>
<th>Piezometer</th>
<th>Tense Meter</th>
</tr>
</thead>
<tbody>
<tr>
<td>18-Oct-95</td>
<td>10.1 kPa</td>
<td>8.6 kPa</td>
<td>-2.3 kPa</td>
<td>39 kN</td>
</tr>
<tr>
<td></td>
<td>(1.47 psi)</td>
<td>(1.27 psi)</td>
<td>(-0.34 psi)</td>
<td>(8800 lbs) data logger (Figure 6.3.A)</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>48 kN</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>(10,800 lbs) Rocktest</td>
</tr>
<tr>
<td>28-Nov-95</td>
<td>12.1 kPa</td>
<td>8 kPa</td>
<td>-1.5 kPa</td>
<td>error reading.</td>
</tr>
<tr>
<td></td>
<td>(1.76 psi)</td>
<td>(1.16 psi)</td>
<td>(-0.22 psi)</td>
<td></td>
</tr>
<tr>
<td>20-Feb-96</td>
<td>error reading.</td>
<td>18.3 kPa</td>
<td>7.8 kPa</td>
<td>40 kN</td>
</tr>
<tr>
<td></td>
<td></td>
<td>(2.65 psi)</td>
<td>(+1.13 psi)</td>
<td></td>
</tr>
</tbody>
</table>

obtain the data in Table 6.3.A; consequently, the data should be reliable for determining a mathematical model for predicting vertical stress on a sill mat.
Figure 6.3.A: Cable tension versus time for the 2656 sill mat. Data includes the cemented backfill pour and the 1st lift of backfill (6.1 vertical meters) placed on top of the cemented backfill.
7.0 Proposed Sill Mat Design

The critical factor of sill mat design is the estimation of vertical stress before the onset of sill mat failure. The current theories for estimating vertical stress in backfill do not account for fluctuations in stope geometry. For example, the footwall dip and ore widths change drastically from lift to lift in some narrow vein gold mines - this fact is encountered at the Snip mine. Consequently, a different approach to estimating vertical stress is suggested. The new approach is based upon Terzaghi’s (1948) Arching Theory and the first order linear differential equation:

\[ \frac{d\sigma_v}{dz} = (\gamma - 2c/w) - (2K\sigma_v \tan(\phi)/w) \]

The solution to the above equation is proposed:

**Marcinyshyn's Arching Theory Summation Series (MATSS) for Determining Vertical Pressure in Soil/Backfill.**

\[ \sigma_v = \left[ \frac{\gamma w_n - 2c_n}{2 K_n \tan(\phi_n)} \right] + \left[ \frac{1}{2} \sum_{i=1}^{n} \left\{ \frac{(\gamma_i w_i - 2c_i)}{(K_i \tan(\phi_i))} \right\} - \left\{ \frac{(\gamma_i w_i - 2c_i)}{(K_i \tan(\phi_i))} \right\} \right] e^{-x} \]

where,

- \( \chi \) = 2 \left[ \frac{(K_n z_n w_n) \tan(\phi_n)}{K_{n-1} z_{n-1} w_{n-1}} \tan(\phi_{n-1}) + \ldots + \frac{(K_i z_i w_i) \tan(\phi_i)}{K_{i-1} z_{i-1} w_{i-1}} \tan(\phi_{i-1}) \right]
- \( K_i \) = coefficient of lateral earth pressure of stope segment i (meters)
- \( z_i \) = length of stope segment i (meters)
- \( c_i \) = cohesion + negative pore pressure (Pascals)
- \( w_i \) = stope width of stope segment i, perpendicular to footwall (meters)
- \( \gamma_i \) = unit weight of soil/backfill of stope segment i, (N)
- \( \phi_i \) = soil/backfill friction angle of stope segment i (degrees)
- \( i \) = a stope segment
- \( n \) = total # of stope segments i
- \( K_n \) = coefficient of lateral earth pressure of bottom segment
- \( w_n \) = width of bottom segment (cemented backfill, meters)
- \( c_n \) = cohesive strength of bottom segment (cemented backfill, Pascals)
- \( z_n \) = length of bottom segment (cemented backfill, meters)
- \( \gamma_n \) = unit weight of bottom segment (cemented backfill, N)
- \( \gamma_0 \) = 0
- \( c_0 \) = 0
- \( q_0 \) = 0
- \( z_0 \) = 0.
7.1 Derivation of MATSS from Terzaghi's Model

Since backfill is similar to a fine to coarse grained sandy soil, the vertical backfill stress condition in a stope can be defined by soil arching theory and is given by the equation (Terzaghi, 1948, Figure 7.1.A):

\[ \text{equation I} \quad \frac{d\sigma_v}{dz} = \frac{(\gamma_{\text{BF}} - 2c/w) - (2K\sigma_v \tan(\phi)/w)}{\text{vertical stress,}} \]

where,

- \( \sigma_v \) = vertical stress,
- \( \gamma_{\text{BF}} \) = soil/backfill density,
- \( c \) = cohesion + negative pore water pressure,
- \( w \) = stope width perpendicular to footwall and hangingwall,
- \( K \) = coefficient of lateral earth pressure,
- \( \phi \) = soil/backfill friction angle.

Clearly, this is a first order linear equation of the form (Boyce and DiPrima, 1986):

\[ \text{equation II} \quad 0 = \frac{dy}{dz} + ay + b \]

where,

- \( \frac{dy}{dz} = \frac{d\sigma_v}{dz} \),
- \( y = \sigma_v \),
- \( a = 2K\tan(\phi)/w \),
- \( b = \gamma_{\text{BF}} - 2c/w \).

Consequently, the equation must have a solution of the form (Boyce, DiPrima, 1986):

\[ \text{equation III} \quad y = b_i e^{az} + b_n. \]

Terzaghi's solution to equation I in the form of equation III is:

\[ \text{equation IV} \quad \sigma_v = \left[ \frac{(\gamma_w - 2c)}{2K\tan(\phi)} \right] \left[ 1 - e^{-\frac{(Kz/w)\tan(\phi)}} \right] + qe^{-\frac{(Kz/w)\tan(\phi)}} \]

where,

- \( b_i = -(\gamma_w - 2c)/2K\tan(\phi) \)
- \( b_n = (\gamma_w - 2c)/2K\tan(\phi) \)
- \( a = 2K\tan(\phi)/w \)
- \( q = \) surcharge load on surface
- \( z = \) depth from q to vertical stress surface.
Unlike Terzaghi’s model, MATSS assumes more than one segment (Figure 7.1.B). The solution for equation I is derived by substituting the appropriate values into equation IV and results in the following equation:

\[ \sigma_v = \left(\frac{\gamma_i w_i - 2 c_i}{2 K_i \tan(\phi_i)}\right) + \left[1/2\right] \sum_{i=1}^{n} \left[\frac{(\gamma_i w_i - 2 c_i)}{(K_i \tan(\phi_i))}\right] e^{-x} \]

where,

- \(\chi\) = \(2 \left(\frac{K_i z_i}{w_i}\right)\tan(\phi_i) + \left(\frac{K_{i-1} z_{i-1}}{w_{i-1}}\right)\tan(\phi_{i-1}) + \ldots + \left(\frac{K_1 z_1}{w_1}\right)\tan(\phi_1)\)
- \(K_i\) = coefficient of lateral earth pressure of stope segment i
- \(z_i\) = length of stope segment i (meters)
- \(c_i\) = cohesion + negative pore pressure (Pascals)
- \(w_i\) = stope width of stope segment i, perpendicular to footwall (meters)
- \(\gamma_i\) = unit weight of soil/backfill of stope segment i, (N)
  \(\gamma_n = \gamma_{n-1} \ldots = \gamma_1\), for simplicity
- \(\phi_i\) = soil/backfill friction angle of stope segment i (degrees)
- \(i\) = a stope segment
- \(n\) = total # of stope segments i
- \(K_n\) = coefficient of lateral earth pressure of bottom segment
- \(w_n\) = width of bottom segment (cemented backfill, meters)
- \(c_n\) = cohesive strength of bottom segment (cemented backfill, Pascals)

Figure 7.1.A: Terzaghi’s (1948) vertical stress model for soils.
\begin{align*}
  z_n &= \text{length of bottom segment (cemented backfill, meters)} \\
  \gamma_n &= \text{unit weight of bottom segment (cemented backfill, N)} \\
  \gamma_0 &= 0 \\
  c_0 &= 0 \\
  q_0 &= 0 \\
  z_0 &= 0.
\end{align*}

Clearly, matching equation III and IV, the solution to equation I is determined:

\[
y = \sigma_v = b_1 e^{-ax} + b_n
\]

where,

\[
  b_1 = \left[ \left( \gamma_i w_i - 2c_i \right) / 2 K_i \tan(\phi_i) - \left( \gamma_{i-1} w_{i-1} - 2c_{i-1} \right) / 2 K_{i-1} \tan(\phi_{i-1}) \right]
\]

\[
  b_n = \left( \gamma_n w_n - 2c_n \right) / 2 K_n \tan(\phi_n)
\]

\[
  a = 2 \left[ (K_n z_n/w_n)\tan(\phi_n) + (K_{n-1} z_{n-1}/w_{n-1})\tan(\phi_{n-1}) + \ldots + (K_i z_i/w_i)\tan(\phi_i) \right]
\]

\[
x = z_n z_{n-1} \ldots z_i.
\]

Each segment in the Figure 7.1.B is separated from the next segment for illustrative purposes. Terzaghi's model assumes one vertical stress segment with one stress surcharge (q). MATSS assumes many segments - each segment has a surface surcharge (q_i) and a stress surface (q_{i+1}). The stress surface (q_{i+1}) becomes the surcharge load on the next segment below. In this way a variation of properties (i.e. density) can be modeled.
Figure 7.1.B: MATSS model related to Terzaghi's model for determining vertical stress soil/backfill.
The solution for vertical stress at surface two \((n = 2)\) from Figure 7.1.B by substituting appropriate values into equation IV (Terzaghi, 1948) is:

\[
\sigma_v = \frac{\gamma_2 w_2 - 2c_2}{2K_2 \tan(\phi_2)} \left[ 1 - e^{-\frac{(K_2 z_2/w_2)\tan(\phi_2)}{2}} \right] + q_1 e^{-\frac{(K_2 z_2/w_2)\tan(\phi_2)}{2}}
\]

where,

\[
q_1 = \frac{\gamma_1 w_1 - 2c_1}{2K_1 \tan(\phi_1)} \left[ 1 - e^{-\frac{(K_1 z_1/w_1)\tan(\phi_1)}{2}} \right] + q_0 e^{-\frac{(K_1 z_1/w_1)\tan(\phi_1)}{2}}
\]

\[
q_0 = 0, \text{ assuming air at top surface of soil/backfill.}
\]

Substitution \(q_1\) into \(\sigma_v\):

\[
\sigma_v = \frac{\gamma_2 w_2 - 2c_2}{2K_2 \tan(\phi_2)} \left[ 1 - e^{-\frac{(K_2 z_2/w_2)\tan(\phi_2)}{2}} \right] + \frac{\gamma_1 w_1 - 2c_1}{2K_1 \tan(\phi_1)} \left[ 1 - e^{-\frac{(K_1 z_1/w_1)\tan(\phi_1)}{2}} \right] e^{-\frac{(K_1 z_1/w_1)\tan(\phi_1)}{2}}
\]

Multiply terms:

\[
\sigma_v = \frac{\gamma_2 w_2 - 2c_2}{2K_2 \tan(\phi_2)} - \frac{\gamma_2 w_2 - 2c_2}{2K_2 \tan(\phi_2)} e^{-\frac{(K_2 z_2/w_2)\tan(\phi_2)}{2}} + \frac{\gamma_1 w_1 - 2c_1}{2K_1 \tan(\phi_1)} \left[ e^{-\frac{(K_1 z_1/w_1)\tan(\phi_1)}{2}} \right] e^{-\frac{(K_1 z_1/w_1)\tan(\phi_1)}{2}}
\]

Collecting similar terms:

\[
\sigma_v = \frac{\gamma_2 w_2 - 2c_2}{2K_2 \tan(\phi_2)} + \frac{\gamma_1 w_1 - 2c_1}{2K_1 \tan(\phi_1)} - \frac{\gamma_2 w_2 - 2c_2}{2K_2 \tan(\phi_2)} e^{-\frac{(K_2 z_2/w_2)\tan(\phi_2)}{2}} + \frac{\gamma_1 w_1 - 2c_1}{2K_1 \tan(\phi_1)} \left[ e^{-\frac{(K_1 z_1/w_1)\tan(\phi_1)}{2}} \right] e^{-\frac{(K_1 z_1/w_1)\tan(\phi_1)}{2}}
\]

Comparing \(\sigma_v\) to equation V (MATSS), the equation for vertical stress at surface 2 is given by the following equation:

\[
\sigma_v = \left[ \frac{\gamma_2 w_2 - 2c_2}{2K_2 \tan(\phi_2)} + \left[ \frac{1}{2} \right] \prod_{i=1}^{2} \left\{ \left[ \frac{\gamma_i w_i - 2c_i}{K_i \tan(\phi_i)} \right] - \left[ \frac{\gamma_i w_i - 2c_i}{K_i \tan(\phi_i)} \right] \right\} e^{-\chi}
\]

where,

\[
\chi = 2 \left[ \frac{K_2 z_2/w_2}{\tan(\phi_2)} + \frac{K_2 z_2/w_2}{\tan(\phi_2)} \right] + \left[ \frac{1}{2} \right] \prod_{i=1}^{2} \left\{ \left[ \frac{\gamma_i w_i - 2c_i}{K_i \tan(\phi_i)} \right] - \left[ \frac{\gamma_i w_i - 2c_i}{K_i \tan(\phi_i)} \right] \right\} e^{-\chi}
\]
7.2 Practical Application of MATSS

MATSS can be applied to the estimation of vertical pressure in civil, geotechnical, and mining engineering design. A mining application is presented below for determining the vertical load on a sill mat.

7.2.1 Coefficient of Lateral Earth Pressure

The coefficient of lateral earth pressure, $K$, at any point, is defined as the ratio of the horizontal effective stress, $\sigma'_h$, to the vertical effective stress, $\sigma'_v$, at that point (Canadian Geotechnical Society, 1992):

$$K = \frac{\sigma'_h}{\sigma'_v}.$$

The proper coefficient of lateral earth pressure must be determined when applying MATSS. For normally consolidated soils, the coefficient of earth pressure at rest, $K_0$, is given approximately by the equation (Canadian Geotechnical Society, 1992):

$$K_0 = 1 - \sin(\phi^*)$$

where,

$$\phi^* = \text{effective angle of internal friction}.$$

Backfill pumped as a slurry in to a stope and then drained can be described as a normally consolidated soil; therefore, $K_0$ must define the initial stress state in backfill. As consolidation takes place the coefficient of lateral earth pressure increases because the horizontal stress increases and the vertical stress decreases. The consolidation of backfill takes place as a result of each backfill level being used as a working platform for the next mining lift. In addition, water flow from subsequent backfill lifts will rework the backfill increasing the consolidation effect. Upon dynamic loading (i.e. blasting) the lateral coefficient value will approach $K_0$. Other applications of MATSS (i.e. embankments) must determine if an active or passive condition exists and if changing conditions will alter selection of active, passive, or $K_0$ conditions.

In backfill placed against rigid retaining walls, compaction increases the earth pressure at rest, (values of $K_0$ in excess of 1.0), and even close to the passive condition have been observed
The passive condition is defined as a retaining wall designed to move into the soil it is retaining; conversely, the active condition is defined as a retaining wall that moves away from the soil it is retaining. For sill mat design in narrow vein mining a passive condition is present in a static condition. There are two points that support the use of a passive condition lateral earth pressure coefficient ($K_p$) for sill mat design. First, the Canadian Geotechnical Society (1992) defines the passive earth pressure as the maximum value of lateral earth pressure that can be mobilized by the relative motion of a structure moving against a soil mass. The convergence of a stope's hangingwall and footwall due to stress relaxation follows the above definition. Next, the Society elaborates further by stating:

".... compacted soil (having already been 'prestressed' by compacting work) can require very little or no movement to produce pressures approaching the full lateral pressure".

In addition to each backfill lift being used as a working platform for further lifts, the backfilling (water inflow) of each subsequent lift in cut and fill mining provides the necessary compacting effect required for little hangingwall/footwall convergence to maximize full lateral pressure. Consequently, the coefficient of lateral earth pressure in MATSS is passive. However, after a blast round during sill pillar mining, there may be enough force to disturb a sill mat and the backfill lying above. Thus, the coefficient of lateral earth pressure would be reduced. In this case, with maximum blast damage the coefficient of lateral earth pressure will approach $K_o = 1 - \sin(\phi)$. Figure 7.2.1.A illustrates this point.
After a blast takes place, further hangingwall and footwall convergence is likely. If backfill is disturbed during a blast, backfill reconsolidation takes place unless support material has also been damaged (i.e. cable slings). Therefore, the coefficient of lateral earth pressure would increase again approaching a passive K condition. After pillar removal, a dramatic increase in hangingwall / footwall convergence (i.e. high horizontal stress redistribution) may increase the passive K condition to a value higher than previously achieved.
The passive lateral coefficient of earth pressure is derived from Mohr's circle. The Mohr's circle for backfill in the passive case is illustrated in Figure 7.2.1.B. Solving the geometric relationship from the Mohr's circle results in an equation for determining $K_p$ (Wallace, 1996).

\[
\sin (\phi) = \frac{[ (\sigma_h - \sigma_v) / 2 ]}{[ (\sigma_h + \sigma_v) / 2 ]}
\]

\[
\sin (\phi) \cdot \sigma_h + \sin (\phi) \cdot \sigma_v = \sigma_h - \sigma_v
\]

\[
\sigma_h \cdot (\sin (\phi) - 1) = \sigma_v \cdot (-\sin (\phi) - 1)
\]

\[
K_p = \frac{\sigma_h}{\sigma_v} = \frac{(1 + \sin (\phi))}{(1 - \sin (\phi))}
\]

Figure 7.2.1.B: Derivation of $K_p$ from Mohr's circle (Wallace, 1996).
Figure 7.2.1.B assumes vertical wall support with the horizontal stress greater than the vertical stress. In underground mining, the changes in footwall dip must be considered. Figure 7.2.1.C (Canadian Geotechnical Society, 1992) illustrates the geometry of an embankment used for the calculation of the passive earth pressure coefficient. For MATSS, the embankment is represented by the footwall. This is critical for determining sill mat pressure. Undulations and dip changes in an orebody cause the change of the geometry of Figure 7.2.1.C. In addition, the geometry of the footwall must be used to determine the passive earth pressure coefficient. Using the hangingwall for $K_p$ calculations produces inconsistent results; in other words, the lateral earth pressure acts upon the footwall.

$$K_p^{0.5} = \frac{\sin(\beta+\phi)}{\sin(\beta)} \left\{ \left[ (\sin(\beta-\delta))^{0.5} - \left[ \sin(\delta+\phi) \sin(\phi+i) / \sin(\beta-i) \right]^{0.5} \right] \right\}$$

**Figure 7.2.1.C: $K_p$ derived for an embankment (Canadian Geotechnical Society, 1992).**

The angle ($\delta$) from the perpendicular of the footwall (Figure 7.2.1.C) at which the passive earth pressure acts is zero since the initial elastic movement direction on an open face will be normal to the face because the principle stress perpendicular to the face is zero.
7.2.2 Stope Segments, Segment Geometry (z, c_n)

Stope segments are determined from the dip of the footwall (Figures 7.2.2.A,B,C, Appendix I). As the dip of the stope changes a new segment should be added. The upper most segment is the first segment (i.e. i = 1). The first segment is the first segment below the surface (air) of the backfill. There is no limit to the amount of segments that can be used in MATSS; however, since MATSS converges as \( z \to \infty \), each case will have some depth \( (z) \) at which the number of detailed segments used should be reduced to save computation time. This statement agrees with arching theory and was explained by Blight (1984) whose method assumes \( z \) approaches infinity. Clearly, stope segments depths \( (z) \) are not infinite in length and neither is the sum of \( z_i \). Consequently, Blight (1984) can not be applied to sill mat design as it over estimates the vertical load significantly to the point of requiring an impractical support design. Also, Blight (1984) does not account for fluctuations in dip and variations in stope width. In addition, Blight does not account for different material properties - i.e. cemented and uncemented backfill.

Each segment width \( (x) \) is determined from a perpendicular line drawn from the center point of each footwall segment to the hangingwall (Figure 7.2.2.A,B,C). This method does not have to be used - the method can be modified to select a width \( (w) \) that best represents the average stope width of each segment. Clearly, a program that can determine the average width of the stope segment will lead to more accurate results (i.e. AutoLisp routine in ACAD). A more rigorous approach would determine an equation that typifies the line segment of the hangingwall and footwall. Find the distance between each of these functions for a stope width \( (w) \).

The depth \( (z) \) of each segment is not the vertical depth. In order for arching theory to apply, the depth \( (z) \) must be parallel to the footwall. The depth for each segment is the length of the footwall. Once again however, if a different method of determining a segment depth \( (z) \) due to the geometry of certain segments, then it should be incorporated. For example, Figure 7.2.2.A illustrates \( z_1 - z_5 \). The midpoint of the top and bottom limits of a segment are used to determine the \( z \) length of each segment.
The method for determining the stope segments is based on the footwall geometry. The length of segment widths (w) are influenced by the footwall and hangingwall geometry. Both w and z lengths should best represent the geometry of each segment. In civil applications (i.e. embankments with wall supports at more than one angle) as \( w \to \infty \), \( e^x \) approaches unity and arching theory is once again upheld.

**7.2.3 Cohesion, Negative Pore Pressure**

For mining backfill (uncemented), cohesion is generally zero. However, the oxidation of iron sulphides may increase this value above zero. In Canadian underground mining, moisture is generally present; as a result, negative pore water pressure is a factor. The negative pore water pressure is be used for cohesion (c) in MATSS. Negative pore water pressure is apparent cohesion and not true cohesion. For example, if negative pore water pressure = -1.0 kPa, then \( c = 1.0 \text{ kPa} \). In the field, negative pore water pressure should be determined from piezometers.

If backfill or soil is fully saturated, then MATSS still applies. MATSS in a saturated zone estimates effective stress. Water pressure must be added to MATSS to obtain full normal stress on a plane (sill mat). Consequently, moisture contents of backfills or soils should be determined. In general, the condition of the backfill above a sill mat is fully drained - mining beneath a sill mat occurs months to years after backfill has been placed.

In the saturated zone, the pressure in a static fluid is the same in all directions at any given point; thus, \( K_p \) approaches unity. Furthermore, in a saturated zone the shear strength is reduced to zero since the effective friction angle \( (\phi') \) equals zero (Figure 7.2.3.A). Thus, the simplified equation for \( K_p = (1 + \sin (\phi)) / (1 - \sin (\phi)) \) can be used.
In the saturated zone, the unit weight of the solid material is reduced by the force of buoyancy of the water. For example, a 20 kN soil has an effective unit weight of 10.2 kN - the force of buoyancy of water is 9.8 kN.

Cemented backfill has a cohesive strength, density, and other characteristics such as UCS strength that are different than uncemented backfill. The appropriate parameters are changed and separate stope segments should be employed in MATSS. Consequently, the cemented backfill portion of a sill mat will always be the nth segment in MATSS (Figure 7.1.B).

\[
K_p = \frac{1 + \sin(\phi')}{1 - \sin(\phi')}
\]
\[
= \frac{1 + \sin(0)}{1 - \sin(0)}
\]
\[
= 1.
\]
7.2.4 Application of MATSS to the 2656 Sill Mat

Figures 7.2.2.A,B,C, (Appendix I) illustrate the stope geometry for the 2656 conventional stope.

Figure 7.2.2.C: MATSS stope geometry for February, 1996 backfill level corresponding to a vertical pressure at Flat Jack #2.
The section between FJ#1 and FJ#2 is the cemented backfill. Refer to section 6.0 for instrumentation installation. The width (x) and depth (z) values for each stope segment are listed in Table 7.2.4.A (Appendix I). These values are used in MATSS (Table 7.2.4.B, Appendix I). There are 3 sets of data corresponding to 3 backfill elevations in Table 7.2.4.C as follows:

<table>
<thead>
<tr>
<th>Date</th>
<th>Level</th>
<th>MATSS</th>
<th>FJ#2 Measured and MATSS</th>
<th>Measured</th>
<th>Backfill Above Sill</th>
</tr>
</thead>
<tbody>
<tr>
<td>2656 October 1995</td>
<td>FJ#2</td>
<td>8.1</td>
<td>8.8</td>
<td>8.0</td>
<td></td>
</tr>
<tr>
<td>2656 November 1995</td>
<td>FJ#2</td>
<td>8.6</td>
<td>8.4</td>
<td>10.5</td>
<td></td>
</tr>
<tr>
<td>2656 February 1996</td>
<td>FJ#2</td>
<td>21.6</td>
<td>18.3</td>
<td>20.2</td>
<td></td>
</tr>
<tr>
<td>2656 March 1996</td>
<td>FJ#1</td>
<td>38.7</td>
<td>35.3 inst. error</td>
<td>20.2</td>
<td></td>
</tr>
</tbody>
</table>

Column 1 is calculated from MATSS. Column 2 is a calculation using the measured FJ#2 pressure and MATSS to calculate the pressure at FJ#1 - this procedure is completed to compare MATSS and a known vertical pressure reading (FJ#2) to FJ#1. Column 3 is measured pressure from instrumentation. Column 4 lists the vertical depth of backfill above the sill at the time of instrumentation readings. Comparing other methods to MATSS is difficult because each method makes various assumptions when applying arching theory to vertical stress. However, all methods use various forms of Equation IV and simplify the equation as described below.

Equation IV \[\sigma_v = \frac{(yw - 2c)}{2 K \tan(\phi)} + qe^{-Kz/w2\tan(\phi)}\]  

For example, all methods in sill mat design set the surcharge load (q) to zero since the top of the backfill is not loaded. The depth (z) is usually set to infinity to predict maximum stress. Furthermore, since uncemented backfill is a fine to coarse grained sand at most mines, cohesion is set to zero. Following these assumptions, the equation is reduced to:

\[\sigma_v = \frac{(yw)}{2 K \tan(\phi)}\]  

Similar to Blight and Mitchell, the above form of equation IV over estimates the maximum vertical stress on a sill mat.
In many cases $K$ is set to equal unity or $K = 1 - \sin(\phi)$. The maximum vertical pressure on the sill mat in 2656 based on the above equation with $\gamma = 15,000$ N, $K = 1$, $w = 2.4$, is 26.7 kPa. If $K = 1 - \sin(\phi)$ then the maximum vertical pressure is 60.5 kPa. Using the simplified MATSS model (backfill only) for the Snip mine, the maximum vertical pressure for a 2.4 meter span is:

$$3625.8 \times \text{Span} = 8.7 \text{kPa} \quad (\text{section 7.3.1}).$$

Clearly, too many simplifications overestimate the vertical stress in backfilled stopes. Table 7.2.4.D lists some of the other methods of calculating vertical stress for the February flat jack #2 reading of 18.3 kPa since backfill depth on this date approximates $z = \infty$. Most methods except MATSS and Pakalnis overestimate vertical stress.

<table>
<thead>
<tr>
<th>Method</th>
<th>Vertical Stress (kPa)</th>
<th>Water Pressure (kPa)</th>
<th>Comments</th>
</tr>
</thead>
<tbody>
<tr>
<td>Flat Jack #2</td>
<td>18.3</td>
<td>7.7</td>
<td>(measured)</td>
</tr>
<tr>
<td>Water Pressure</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>MATSS</td>
<td>13.9</td>
<td>21.6</td>
<td>parameters (Table 7.3.4.B, Appendix I)</td>
</tr>
<tr>
<td>Blight (Mitchell)</td>
<td>19.0</td>
<td>26.7</td>
<td>$\gamma = 15,000$ kN/m$^3$, $K = 1.0$, $c=0$</td>
</tr>
<tr>
<td>Blight (Mitchell)</td>
<td>27.0</td>
<td>34.7</td>
<td>$\gamma = 20,000$ kN/m$^3$, $K = 1.0$, $c=0$</td>
</tr>
<tr>
<td>Blight (Mitchell)</td>
<td>52.8</td>
<td>60.5</td>
<td>$\gamma = 15,000$ kN/m$^3$, $K_o = 1.0-\sin(\phi)$, $c=0$</td>
</tr>
<tr>
<td>Coates</td>
<td>45.8</td>
<td>53.5</td>
<td>$\gamma = 15,000$ kN/m$^3$, $K = 1/(1+2\tan^2(\phi))$, $c=0$</td>
</tr>
<tr>
<td>Knutsson</td>
<td>57</td>
<td>64.7</td>
<td>$\gamma = 15,000$ kN/m$^3$, $K_o = 1.0-\sin(\phi)$, $c=0$</td>
</tr>
<tr>
<td>Pakalnis</td>
<td>10.1</td>
<td>17.8</td>
<td>$\gamma = 20,000$ kN/m$^3$, $K = (1.0+\sin^2(\phi))/(1-\sin^2(\phi))$, $c=0$</td>
</tr>
</tbody>
</table>

The piezometer installed above the cemented backfill at the same elevation as FJ#2 provides good insight into the effects of water in the stope which has a direct effect on the vertical pressure exerted on a sill mat. The negative pore water pressure for the data at 8 meters backfill (Oct/95) is -2.344 kPa. For 10 meters (Nov/95) of backfill the negative pore pressure is reduced to -1.517 kPa - attributed to an increase in water content above the cemented
backfill. When more backfill is placed, more water is also introduced into the stope; thus, negative pore water pressure is reduced until it becomes saturated (positive water pressure).

The final data series indicates positive water pressure of 7.7 kPa. The water pressure is added to MATSS to estimate total vertical stress. The effective unit weight of the backfill is 10.2 kN. The reason for this is that effective stress represents the stress transmitted through the solid portion of a soil skeleton (Craig, 1994). The total normal stress on a plane is defined as:

\[ \sigma_v = \sigma_v' + \mu \]

where,

- \( \sigma_v \) = vertical stress (measure by a flat jack)
- \( \sigma_v' \) = effective stress in backfill (MATSS)
- \( \mu \) = water pressure.

In the application of MATSS, attention to water pressure is paramount. If negative pore pressure exists it is treated as an apparent cohesion in MATSS since it acts to reduce backfill load. Positive water pressure is associated with saturated backfill since external pressures such as wall convergence do not prevent water from flowing in any direction; in other words, water pressure in a stope is wholly dependent on water depth. Therefore, the water pressure in a saturated zone is added to effective stress to obtain total vertical stress (\( \sigma_v = \sigma_v' + \mu \)). In the saturated region, cohesion for MATSS is set to zero. The passive coefficient of lateral earth pressure is equal to unity for reasons outlined in section 7.2.3. Increased drainage above the cemented backfill will aid in sill mat stability and lower vertical pressure on a sill mat. However, most sill mats are fully drained before the sill pillar is mined.

This process is explained, referring to Figure 7.2.2.C. Segments 1-11 have an assumed cohesion of 1517 Pa - the last recorded negative pore pressure before saturation. Segment 12 and 13 are in the saturated zone; as a result, cohesion for MATSS is set to zero in these two segments and the unit weight of the backfill is reduced by the force of buoyancy (20 kN - 9.8 kN = 10.2 kN) to obtain the effective pressure on FJ#2. The unit weight of the cemented backfill (segment 13) is reduced to 14.2 kN (24 kN - 9.8 kN). The effect of water on the cemented backfill may also reduce the cohesion to zero since the porosity of the cemented backfill allows water to seep into the cemented backfill. Comparison of computed and
measured results is favorable as shown in Table 7.2.4.D above (21.6 to 18.3 kPa). If negative pore water pressure is increased for more segments above segment 11, then the predicted vertical pressure from MATSS will decrease. However, in the absence of knowledge about negative pore water pressure in backfill higher above the piezometer, only estimates can be made. MATSS calculates effective vertical stress in a saturated zones by considering the effective unit weight of backfill; in addition, MATSS calculates effective vertical stress in unsaturated zones since negative pore water pressure is treated as apparent cohesion. Comparing all measured versus predicted results, clearly, MATSS provides excellent estimation of vertical stress for sill mat design.

7.2.5 Application of MATSS to the 4867 Sill Mat

A pressure cell is located in the 4867 stope (Figure 3.0.A) where the span is approximately 10 meters. A pressure reading was taken when 3.55 meters of uncemented backfill was placed in the stope - 0.55 meters was drained and 3 meters was fully saturated (Figure 7.2.5.A).

\[
\text{\begin{tabular}{c}
\text{stope working surface} \\
\hline
\text{drained backfill} \\
\hline
\text{0.55 m} \\
\hline
\text{3 m} \\
\hline
\text{saturated backfill} \\
\hline
\text{pressure cell} \\
\hline
\text{sill pillar} \\
\end{tabular}}
\]

Figure 7.2.5.A: Uncemented backfill conditions for the 68.3 kPa (9.9 psi) pressure reading in the 4867 stope (vertical cross section).
The values used for MATSS are summarized in Table 7.2.5.A

<table>
<thead>
<tr>
<th>Table 7.2.5.A: MATSS parameters for 4867 pressure cell reading with 0.55 meters drained backfill and 3.0 meters saturated backfill.</th>
</tr>
</thead>
<tbody>
<tr>
<td>i=1, 0.55 meters drained backfill</td>
</tr>
<tr>
<td>K</td>
</tr>
<tr>
<td>Apparent cohesion</td>
</tr>
<tr>
<td>unit weight</td>
</tr>
<tr>
<td>friction angle (\phi)</td>
</tr>
<tr>
<td>depth (z)</td>
</tr>
<tr>
<td>width (x)</td>
</tr>
<tr>
<td>surcharge load (q)</td>
</tr>
</tbody>
</table>

The step by step calculation of MATSS with n=2 and the parameters listed in Table 7.2.5.B is given in Appendix I. The resulting effective stress (MATSS) on the pressure cell is 38.6 kPa. Adding the water pressure (3 m water = 28.4 kPa) to MATSS gives 68.0 kPa total vertical stress. MATSS correlates well to the pressure cell which measured 68.3 kPa.

7.2.6 Possible Errors

All parameters measured in the laboratory may vary from insitu values. For example, the cohesive strength of the insitu cemented backfill may vary from that tested in the laboratory; however, work performed for section 5.0 indicated values normal to industry standards in Canada (CANMET, 1990). The backfill density for MATSS is not corrected for variation with depth. Negative pore pressure may vary with depth as well and was not changed for MATSS calculations. Piezometers at various elevations in the stope would verify any negative pore pressure changes. Small changes in the insitu friction angle of the backfill may exist. In light of the above, any differences in the above parameters are not significant enough to adversely affect MATSS.

The effect of stope segments has a dramatic effect on accuracy if not enough stope segments are used. For example, the February stope segments had to be increased near the saturated zone since the effective stress above the saturated zone becomes more significant for determining effective stress. The unit weight is used to calculate the effective stress in the
saturated zone; thus, the effective stress above the saturated zone has a greater influence on the
total effective stress calculated by MATSS. For the 2656 sill mat MATSS iterations, there is
not a significant discrepancy between the MATSS calculated value and the instrumentation
results. Instrumentation on the other hand, can have numerous factors affecting its
performance underground. However, measurements were taken with 3 loggers throughout the
test period and error in instrumentation appears minimal. The similarity between MATSS and
instrumentation results indicates MATSS correlates well to field data.

7.3 Cable Sling Spacing
In order to determine the cable spacing for sill mat support, four levels of calculations are
executed. First, the vertical stress (MATSS, n=1 for uncemented backfill since insitu
properties of cemented backfill are questionable) for various stope spans is determined. Using
only backfill for calculations provides a conservative estimation for determining cable sling
spacing since the vertical pressure will be over estimated. Second, the maximum deflection
based on the cable properties and span is determined. Next, the support capability of the cable
slings is calculated. Finally, comparing the vertical load to support capability, a formula for
cable spacing is proposed.

7.3.1 Vertical Stress (MATSS)
Assuming various stope spans (perpendicular to hangingwall and footwall) and using the Snip
Orebody as a model, a vertical load employing MATSS for each span is determined. The dip of
a stope for this method is assumed to be 52 - 53 degrees - the maximum vertical stress is
realized from the backfill when the lateral coefficient of earth pressure is passive ($K_p$). The
effect of $K_p$ in Figure 7.3.1.A illustrates this point.
Figure 7.3.1.A: The effect of footwall dip on $K_p$.

At 52° - 53° $K_p$ is a minimum for a friction angle of 34°. $K_p$ for friction angles of 25° and 40° are also included in Figure 7.3.1.A. As the dip of the ore body increases, the friction angle becomes more of a factor for determining vertical pressure. The higher the friction angle the higher the coefficient of lateral earth pressure. A larger value for the coefficient of earth pressure results in a lower vertical pressure on a sill mat. In other words, materials with higher friction angles have better arching capabilities and tend to support themselves more than lower friction angle materials.

A simplified MATSS model is used to calculate vertical stress. The backfill is assumed to be drained with apparent cohesive strength due to negative pore water pressure (1.517 kPa - this value may be higher or lower; however, it is the last recorded value for negative pore water pressure). A stope height parallel (z direction) to the dip of the stope is assumed to have a minimum height of 200m. Values for iterations on various spans indicate that the effects of backfill influence sill mat loads from a distance of $z = 5x$. In other words, the load on a sill mat
is influenced by backfill that is a factor of 5 times the span distance along the z direction. For example, a span of 20 meters is affected by the influence of backfill 100 meters away in the z direction.

Figure 7.3.1.B: Backfill within distances of 5x have some effect on vertical pressure on a sill mat.

Figure 7.3.1.C illustrates the effects of depth (z) on the load of a sill mat with a 10 meter span. Clearly, after the depth is greater than 2 to 3 times the sill mat span, the effects of backfill are minimized. The curve is similar for all spans - only the vertical pressure changes.

Figure 7.3.1.C: Backfill within distances of 2-3x have the greatest effect on sill mat vertical pressure.
From the above assumptions material properties previously outlined (section 4.0, 5.0), the resulting graph for predicting vertical load on a sill mat is illustrated in Figure 7.3.1.D.

The vertical backfill pressure for Snip Mine sill mats can be summarized by the equation as determined by Figure 7.3.1.D.

\[ \sigma_{\text{vertical}} = 3625.8 \times \text{sill mat span (meters)} \text{ N/m}^2 \]

where,

\[ \text{sill mat span (meters)} \]

Two other conditions are illustrated on Figure 7.3.1.D - lower apparent cohesive strength (negative pore water pressure) and friction angle (\( c = 0, \phi = 25^\circ \)) and higher apparent cohesive strength and friction angle (\( c = 2300 \text{ Pa}, \phi = 40^\circ \)). The lower values correspond to the upper line and give a high vertical load - this is the worst case with no cohesive strength and a low friction angle. The lower line on Figure 7.3.1.D indicates a reduced vertical load for higher friction angles and negative pore water pressure. The apparent cohesion of 2300 Pa is selected since this was the highest recorded negative pore water pressure measured during the test period. These two properties become more important at larger spans as illustrated in Figure 7.3.1.D.
7.3.1.D. Clearly, the material properties of the backfill and the effect of water must be determined for each sill mat to calculate load. Sill mats installed in stopes with poor drainage characteristics will have significant increases in vertical load.

7.3.2 Maximum Deflection

The maximum deflection of a cable sling is determined from the span and cable properties. Both the 1.27 cm (1/2 inch) and 1.6 cm (5/8 inch) high tensile strength steel cable have the same maximum strain when the cables have the same stress since they are made from the same material, so this section holds true for both cables. The stress depends on the cable tension and cable cross sectional area; as a result, the tensions will be different in each of the cables, but the stress will be the same for the maximum strain. In other words, a given strain results in the same stress for 2 different cables made out of the same material. For example, two different diameter cables with a 3% strain have the same stress. In other words, Hooke's Law is independent of diameter \( \sigma = E \varepsilon \).

The parabolic length of cable based on the span and deflection as given by Nash (1978) is:

\[
\text{Equation A } \quad l = x \left[ 1 + \frac{8}{3} \left( \frac{d}{x} \right)^2 - \frac{32}{5} \left( \frac{d}{x} \right)^4 + \frac{256}{7} \left( \frac{d}{x} \right)^6 - \ldots \right]
\]

where,
- \( l \) = cable length due to deflection
- \( x \) = span
- \( d \) = deflection at center of cable.

Note: units of length must be consistent.

The cable length (l) is equal to the sum of the original length (x) and the amount of elongation (El) due to strain. Clearly,

\[
\text{Equation B } \quad l = x + El.
\]

From material properties (Van Vlack, 1987), the maximum strain (\( \varepsilon_{\text{max}} \)) is:

\[
\varepsilon_{\text{max}} = \frac{\text{Cable Tension Maximum}}{(E \times A)}
\]

where,
- \( E \) = Young’s Modulus
- \( A \) = Nominal Cable Cross-Sectional Area.
As a result, the maximum elongation for a given span is:

\[ E_l = x \cdot \varepsilon_{\text{max}}, \]

Equation B becomes,

\[ 1 = x + (x \cdot \varepsilon_{\text{max}}). \]

Substituting into Equation A, \( \varepsilon_{\text{max}} \) is defined as:

Equation C:

\[ \varepsilon_{\text{max}} = \frac{8}{3}(d/x)^2 - \frac{32}{5}(d/x)^4 + \frac{256}{7}(d/x)^6 - \ldots. \]

Solving the above equation for each span until the maximum deflection is obtained at each \( \varepsilon_{\text{max}} \) results in the following graph:

![Figure 7.3.2.A: Maximum deflection for high tensile steel cable based on the maximum strain for a cable sling installed with split set, cable grip, and cement grout.](image)

Figure 7.3.2.A is corrected for additional slack that appears to be a characteristic in the cable sling system. As described in section 3.1, some inherent slack allows for extra deflection in a cable sling. This fact works in favor of cable sling design - more deflection mobilizes more vertical support strength in the cable sling due to geometry. One reason for increased deflection is the amount of grout used for installation of a cable sling - grout does not cover the entire cable; therefore, the maximum strain is increased because more cable than the span length exists. In addition, some slack in the cable sling may be attributed to initial setting loads required to tighten the system so all components are straining and resisting the vertical load.
For the above reasons, Figure 7.3.2.A data is corrected by adding a factor of 0.02 meters deflection per meter span to correlate the graph to Cable Tension Vs. Deflection Graph from field testing (Figure 3.2.2.3.A in section 3.2.2.3). This results in a corrective strain value of 0.0008 m/m of Span.

\[ d_{\text{span}} = 0.0791 \times \text{span} \text{ meters} \]

### 7.3.3 Linear Vertical Support

The support of a parabolic shaped cable sling is defined by Figure 1.2.8.3.A (Nash, 1978) in section 1.2.8.3:

**Equation 7.3.3.A**

\[ \omega = \frac{8 d H}{x^2} \]

where,

- \( \omega \) = linear load per horizontal unit along span, N/m
- \( d \) = deflection, m
- \( H \) = maximum cable tension, N
- \( x \) = span, m.

The above equation accounts for the support of the span cable slings at the center of the span. A similar equation based on the tension at the ends of the cable from the same figure is:

**Equation 7.3.3.B**

\[ \omega = \frac{2 T}{[x \times (1 + (x^2 / 16 d^2))^{0.5}]} \]

where,

- \( \omega \) = linear load per horizontal unit along span, N/m
- \( d \) = deflection, m
- \( T \) = maximum cable tension, N
- \( x \) = span, m.

The maximum cable tension and various spans with corresponding maximum deflections are entered into both equations to determine the maximum linear load that each equation calculates. The maximum linear load along span as determined by a maximum cable tension at the ends of the cable (equation 7.3.3.B) is generally 5% lower than the maximum linear load calculated by the maximum cable tension in the center of the cable (equation 7.3.3.A). This fact is illustrated in Table 7.3.3.A.
Table 7.3.3.A: Maximum linear load at center and ends of a parabolic sagging cable (Nash, 1978).

<table>
<thead>
<tr>
<th>Span (x)</th>
<th>Center (H)</th>
<th>Ends (T)</th>
<th>Difference</th>
</tr>
</thead>
<tbody>
<tr>
<td>m</td>
<td>kN</td>
<td></td>
<td></td>
</tr>
<tr>
<td>5</td>
<td>32.7</td>
<td>31.1</td>
<td>4.89%</td>
</tr>
<tr>
<td>10</td>
<td>16.3</td>
<td>15.6</td>
<td>4.89%</td>
</tr>
<tr>
<td>15</td>
<td>10.9</td>
<td>10.4</td>
<td>4.89%</td>
</tr>
<tr>
<td>20</td>
<td>8.2</td>
<td>7.8</td>
<td>4.89%</td>
</tr>
</tbody>
</table>

A conservative design would use equation 7.3.3.B to obtain the maximum support along strike. Equation 7.3.3.A is used for sill mat design - it is corrected by a factor of 1/1.0489 to account for the difference between equation 7.3.3.A and 7.3.3.B.

Some allowance for the support of the strike cables must be included in sill mat design. Although the strike cable may not follow parabolic loading like the span cables, the total cable support of the cable slings (span and strike cables) is postulated:

\[ \omega_{\text{linear}} = \omega_{\text{span}} + \omega_{\text{strike}} \]

where,

\[ \omega_{\text{span}} = 8 \frac{d_{\text{span}} H}{(x_{\text{span}})^2} . \]

Since the deflection of the cables is controlled by the span cables, the support of the strike cables is:

\[ \omega_{\text{strike}} = 8 \frac{d_{\text{span}} H}{(x_{\text{strike}})^2} \]

As the strike length of a sill mat approaches the span length, the above hypothesis is true since the geometry of strike cables will be similar to the span cables (Figure 3.1.A, section 3.1). Substituting Equation 1a and 1b, the equation for linear load along span is defined as:

\[ \omega_{\text{linear}} = \left( \frac{1}{1.0489} \right) \left[ 8 \frac{d_{\text{span}} H}{(x_{\text{span}})^2} + 8 \frac{d_{\text{span}} H}{(x_{\text{strike}})^2} \right] \]

\[ = 7.63 \frac{d_{\text{span}} H \left[ 1 / (x_{\text{span}})^2 + 1 / (x_{\text{strike}})^2 \right]}{N/m} \]

where,

- \( d_{\text{span}} \) = maximum deflection based on span
- \( H \) = maximum cable strength
- \( x_{\text{span}} \) = sill mat span
- \( x_{\text{strike}} \) = sill mat strike length.
As the strike length increases, the term $1/ (x_{\text{strike}})^2$, will decrease until its support capabilities become insignificant. For example, Figure 7.3.3.A is a graph illustrating the importance of support capabilities of strike cables. At a Strike/Span Ratio of 2:1 the support carried by the strike cables becomes insignificant and the majority of the load is carried by the span cables (i.e. strike lengths greater than 20 meters have very little load carrying capabilities for a 10 meter span). A similar graph to Figure 7.3.3.A for various spans results in similar curves; however, the strike to span ratio at which the strike cables do not carry a significant load varies. For spans less than 10 meters the ratio is increased and for spans greater than 10 meters the ratio is decreased.
7.3.4 Cable Spacing

The vertical pressure (section 7.3.1) and linear vertical support (section 7.3.3) are used to calculate the cable spacing for a sill mat. For adequate design, the support must be equal to or greater than the load on the sill mat.

Load = Support.

Know,
Support along Span = \( \omega_{\text{linear}} \),
and Load along Span = \( \sigma_{\text{vertical}} \times (\text{Cable Spacing Along Strike}) \).

Therefore,
\( \sigma_{\text{vertical}} \times (\text{Cable Spacing Along Strike}) = \omega_{\text{linear}} \).

Thus,
Cable Spacing = \( \frac{\omega_{\text{linear}}}{\sigma_{\text{vertical}}} \)
= \( 7.63 \, d_{\text{span}} \, H \left[ \frac{1}{(x_{\text{span}})^2} + \frac{1}{(x_{\text{strike}})^2} \right] / [3625.8 \, (x_{\text{span}})] \).

recalling, \( d_{\text{span}} = 0.0791 \, x_{\text{span}} \)
then,
Cable Spacing = \( 7.63 \, (0.0791 \, x_{\text{span}}) \, H \left[ \frac{1}{(x_{\text{span}})^2} + \frac{1}{(x_{\text{strike}})^2} \right] / [3625.8 \, (x_{\text{span}})] \)

**Cable Spacing** = \( (1.66 \times 10^{-4}) \, H \left[ \frac{1}{(x_{\text{span}})^2} + \frac{1}{(x_{\text{strike}})^2} \right] \)

where,
- \( H \) = maximum cable strength, N
- \( H = 184,000 \, N \) for 1.27 cm (1/2 inch) cable
- \( H = 258,000 \, N \) for 1.6 cm (5/8 inch) cable
- \( x_{\text{span}} \) = length of span at sill mat, m
- \( x_{\text{strike}} \) = strike length of sill mat, m.

The 2656 sill mat had approximate dimensions of 2.4 meters span and a strike length of 20 meters and used 1.6 cm (5/8 inch) cable. The cable spacing suggested by the above equation is 7.5 meters:

\[ \text{C.S.} = 1.66E-4 \times 258,000 \times \left[ \frac{1}{(2.4)^2} + \frac{1}{(20)^2} \right] \]
\[ = 7.5 \, \text{meters}. \]

The spacing installed for the 2656 sill mat is 1.5 meters. Clearly, the 2656 sill mat will be quite stable for sill pillar removal if proper drainage is utilized and blast damage does not occur.
The above equation has been developed for the Snip Mine site and is based on characteristics of that mine site. Furthermore, it assumes fully drained backfill which will have a moisture content of 10 - 15 % moisture. Since the characteristics of insitu cemented backfill are difficult to determine, the cable spacing formula is derived from one stope segment ( n = 1 ) of backfill. Also, blasting is assumed to have no damage to the cable sling support capabilities. The above equation assumes a constant footwall slope of 52 - 53 degrees which gives the lowest coefficient of lateral earth pressure ( i.e. highest vertical stress ). The cable spacing equation should be used when the undercut of a stope is finished and the span of a sill mat is known. However, once mining has been completed and stope dimensions have been defined by survey, the application of the design procedure outlined in section 7.0 should be completed to estimate the actual cable spacing that should have been used. In addition, sill pillar removal can be designed based on the Cable Spacing Formula.

For larger spans where smaller cable spacing is not cost effective, the Cable Spacing formula can be used to calculate the largest panel that should be mined during sill pillar removal. The 3764 sill mat ( Figure 3.0.A ) provides an excellent example. The sill mat failed when the sill pillar beneath it was mined. Blasting was one major cause of the sill mat failure. Another factor was the cable spacing. Two meter cable spacing with 1.6 centimeter diameter ( 5/8 inch) cable was installed over most of the 3764 sill mat. Strike lengths greater than 20 meters existed and spans ranged from 10 to 15 meters. Using 20 meters strike length and 15 meters span the cable spacing equation suggests 0.30 meters spacing:

\[
\text{C.S.} = \frac{1.66E-4 \cdot 258,000}{1/(15^2) + 1/(20^2)} = 0.30 \text{ meters.}
\]

According to this calculation, not enough support has been installed. This problem can be handled by modifying the mining method of the sill pillar. A 2 meter cable spacing and 20 meter strike length for a 1.6 cm ( 5/8 inch ) cable is entered in the Cable Spacing formula. From the Cable Spacing formula, the largest span that should expose the sill mat cable slings is 4.8 meters:

\[
\frac{1}{\text{panel}^2} = \frac{2}{1/(1.66E-4)(258,000)} - (1/20^2) - (1/20^2) = 4.8 \text{ meters.}
\]
Through discussions with production, geology, and engineering a mining method for sill pillar removal in panels under larger sill mats at Snip mine can be determined based upon the Cable Spacing formula.

7.3.5 Weld Wire Mesh TYCLIPS™

Current installation of weld wire mesh in sill mat design utilizes TYCLIPS (Tannant, 1995) to attach weld wire mesh to the cable slings. The previous method used #9 gauge wire (3.7 mm diameter) to attach weld wire mesh to the cable slings. The wire was wrapped around the cable slings and welded wire. Twisting the wire by hand locked the weld wire mesh on to the cable slings. This method proved to be a weak point in sill mat design. It is evident from the 3764 sill mat (Figure 3.0.A) failure that attaching the weld wire mesh (WWM) to the cable slings with #9 gauge wire is not an adequate method of anchoring the WWM to the cable slings. After blasting a portion of the sill pillar beneath a sill mat, failure of the sill mat occurred. Evidence from the failure pointed to deficiency in the strength of the #9 gauge wire and its inability to maintain the weld wire mesh attached to the cable slings.

Once the unconsolidated backfill failed - loss of cohesion - the backfill flowed down onto the ore pile. The force from the blast caused the #9 gauge wire to yield and prevented the WWM from being attached to the cable slings. Also, the backfill may have flowed through the WWM. The failure of the WWM was a result of the method of attaching the WWM to the support cables and not a factor of the WWM strength. The initial purpose of the #9 gauge wire was to provide temporary support while pouring the cemented backfill. Once the cemented backfill is set, the cemented backfill acts like reinforced concrete with the WWM within it. This design methodology did not work since the WWM did not remain within the cemented backfill. Other explanations may be that the blast vibrated the cemented backfill sufficiently enough to reduce it to a granular state or there was not enough cohesion (high moisture content) to maintain the cemented backfill in place once the screen was free to move. As a result, the mining personnel at Snip suggested using wire attachments called TYCLIPS (Tannant, 1995).
Other industries contacted for possible methods of attaching weld wire mesh to support cables are:

- metal strapping
- concrete reinforcing
- steel fabricators and manufacturers.

Metal strapping is geared towards shipping materials. Flat stainless steel strapping has a high tensile strength (up to 725 MPa). However, the clips crimped on to the metal strapping to secure it are made up of softer metal and are susceptible to corrosion in underground applications. The strapping is easy to place around the weld wire mesh (WWM) and cables, but placing the clips on the strapping requires a reasonably flat surface so crimping the clip on to the strapping may be difficult for sill mat construction.

The concrete reinforcing industry uses #16 gauge (3 mm diameter) tie wire to hold rebar in place before concrete is poured. Once surrounded by cured concrete, the #16 gauge tie wire provides no support to design. In the case of a sill mat where the tie wire will be exposed from sill pillar removal, failure of the tie wire would occur due to lack of strength unless adequate amounts of #16 gauge wire were used - this would unproductive in a mining environment. The concrete reinforcing industry does not supply a material that would provide sufficient support within a sill mat employing the above criteria.

Steel fabricators and manufacturers may be able to produce a #4 - #6 gauge clip (6 mm diameter). However, the current TYCLIP (6 mm diameter) works reasonably well and any similarities may infringe upon copyrights. An increase in the size of the current TYCLIP (larger internal length and width) should improve ease of installation.

Gabions used in civil engineering designs to support retaining walls use a metal lacing wire to attach wire mesh to one another. The metal lacing wire is not a heavy enough gauge wire for sill mat construction.
After reviewing various methods for attaching WWM to each other, the TYCLIPS offer the most cost effective method of support. The wire clips must attach WWM to the support cables. Also, clipping the WWM sheets to each other should enhance the capability of the WWM to support backfill. Clearly, the next step is to determine the clip spacing (i.e. the number of clips) that must be used to ensure the WWM strength is being utilized. The following formula determines the load capability of weld wire mesh based on uniform loading (Tannant, 1995):

\[
L_p = \frac{(2NT)}{[1 + \left(\frac{s}{4d_p}\right)^2]^{0.5}}
\]

where,

- \(L_p\) = peak load capability of weld wire mesh (N)
- \(N\) = number of individual wires carrying the load
- \(T\) = tensile load capability of each individual wire (N)
- \(s\) = bolt spacing (m)
- \(d_p\) = peak mesh deflection (m).

Testing the weld wire mesh with rock bolts placed in a diamond pattern (Figure 7.3.5.A) failed the WWM by tensile failure (Tannant, 1995). Four wires carried most of the load for the diamond pattern (Tannant, 1995).

![Figure 7.3.5.A: Rock bolts used to anchor weld wire mesh are arranged in a diamond pattern. In this arrangement, tensile failure of the weld wire mesh is most likely to occur (plan view).](image)
By assuming the TYCLIPS are rock bolts the above equation can be used to calculate the TYCLIP spacing that should be used for sill mat installation. Tensile failure of the weld wire mesh is most likely since the TYCLIPS are installed perpendicular to individual wire on the WWM (Figure 3.1.E, section 3.1). Consequently, if the TYCLIPS are assumed to be rock bolts, the peak load \( L_p \) must be reduced by a factor of 2 since the installation configuration (Figure 3.1.E, section 3.1) has two of the WWM wires carrying the load and not four wires carrying the load. Although the TYCLIPS are installed on one of the WWM wires, as the wire bends, it will behave as two wires due to geometry (Figure 3.2.2.2.A).

Knowing some of the terms in Tannant’s peak load equations allows it to be reduce to terms characterizing the Snip Mine Site. WWM peak strength occurs between 0.1 and 0.25 meters deflection; therefore, a peak mesh deflection of 0.25 meters can be assumed since more movement in the cable sling system than in rock bolt usage is expected. The maximum tensile strength of the #9 gauge WWM used at the Snip Mine Site as supplied by the manufacturer is 7100 N. Also, the peak load \( L_p \) is equal to the vertical stress \( \sigma_{\text{MATSS}} \) * the square of the Cable Spacing \( \text{C.S.}^2 \). The number of wires carrying the load \( N \) can be defined in terms of TYCLIP Spacing \( s \). The illustration in Figure 7.3.5.B helps to visualize the derivation of \( N \) in terms of spacing \( s \).
It is evident from Figure 7.3.5.B that the number TYCLIPS attaching the WWM to the cable slings is:

\[
\text{# TYCLIPS} = 4 \times \frac{\text{Cable Spacing}}{s},
\]

where,

\[
s = \text{TYCLIP spacing}.
\]

Since the number of wires supporting the mesh at each TYCLIP as explained previously is two, then,

\[
N = 2 \times \text{# TYCLIPS}
\]

therefore,

\[
N = 8 \times \frac{\text{Cable Spacing}}{s}.
\]

Substituting \(d_p, T, L_p,\) and \(N\) into Tannant's equation:

\[
\sigma_{\text{MATSS}} \times \text{C.S.}^2 = \left( 2 \left( \frac{8 \text{ C.S.}}{s} \right) (7100) \right) / \left[ 1 + \left\{ s / (4 \times 0.25) \right\}^2 \right]^{0.5}
\]
rearranging,

Equation 7.3.5.A
\[ s \left( 1 + s^2 \right)^{0.5} = \frac{113,600}{\sigma_{\text{MATSS}} \ast \text{C.S.}}. \]

recalling \( \sigma_{\text{MATSS}} \) and C.S. from section 7.0,

Equation 7.3.5.B
\[ s \left( 1 + s^2 \right)^{0.5} = \frac{188,740}{\left( x_{\text{span}} \ast H \left[ 1 / (x_{\text{span}})^2 + 1 / (x_{\text{strike}})^2 \right] \right)} \]

where,
- \( s \) = TYCLIP spacing (m)
- \( x_{\text{span}} \) = span (m)
- \( x_{\text{strike}} \) = strike length (m)
- \( H \) = cable strength (N).

Solving the above equation (Equation 7.3.5.B) for the 2656 sill mat where a 60 cm (24 inches) TYCLIP spacing was used, a value of 1.1 meters (43 inches). Clearly, the TYCLIP spacing on the 2656 sill mat should be strong enough to prevent the WWM from separating from the cables. For larger sill mats where Cable Spacing is predetermined, Equation 7.3.5.A can be used. From a practical installation point of view, a TYCLIP spacing of less than 30 cm (12 inches) is unproductive. For ease of installation, it is not necessary to place clips at this interval on the support cables; where overlapping WWM makes it difficult to attach the WWM to the support cables, the clips can be used to attach the WWM to each other.

The TYCLIPS are made of #4 gauge (6 mm diameter) steel. Failure of the weld wire mesh and not the TYCLIPS which are a thicker gauge steel will occur if the proper cable spacing is used. As the WWM wires bend, failure may occur in the welds of the WWM. The welds failing increase the length of wire subjected to deformation loads; thus, the strain is transferred over a larger length of wire.
8.0 Production Considerations

The stability of the sill pillars and stopes based on the geometry of current mining is determined from the Modified Stability Method (Potvin et al, 1992) and the Detour Mine Lake Stability Method (CANMET, 1992). Sill mat design may be affected if current mining methods at Snip cause instability to backfill, sill pillars, or stope walls.

8.1 Mining Method

From the literature search and personal contact with various mines (section 1.2), it is evident that there are five mining methods currently employing sill mats for sill pillar recovery or as part of the mining method. The mining methods that use sill mats are (Figure 1.2.2.A):

- Vertical Crater Retreat
- Longhole
- Mechanized Cut and Fill
- Conventional Cut and Fill
- Underhand Cut and Fill

Vertical Crater Retreat is used at the Lockerby Mine in Sudbury and requires a long vertical distance in order to be cost effective and is not applicable to the Snip Mine since undulations in the ore body make it difficult to predict ore outlines. The underhand cut and fill mining method is feasible at the Snip Mine; however, it is not as efficient as the current cut and fill mining and requires high quality cemented backfill or concrete. In addition, underhand cut and fill mining is best applied in high stress environments. Since rockbursting has not been seen at the Snip Mine, stress is not problem to date. Furthermore, the ore zone at Snip is weak and would yield under high stress. If the ore zone yields, the chances of rockbursting occurring in a sill pillar are reduced.

Longhole mining has been used beneath sill mats at a few mines. At Stall Lake (Davies, 1993), only 30% ore recovery (Figure 1.2.5.A) due to high stress was achieved for a sill pillar recovery. Detour Lake (Placer Dome) uses cut and fill and longhole mining; however, cut and fill mining is conducted below a sill pillar. The intervening sill pillar (9 - 12 meters) is removed by longhole with a 2 to 3 meter remnant sill pillar (for spans greater than 20 meters) left in
place. The remnant sill pillar prevents backfill from flowing into the mining area from the stope above the sill pillar. The remnant pillar will be cable reinforced before mining commences. Muck is remove with remote equipment. Longhole blasting under the 3764 sill mat at Snip Mine damaged the sill mat and was a contributing factor to a failure that occurred in that sill mat. Improved blast design may enhance the recovery of sill pillars when mined by longhole methods.

The grade at Detour Lake (7g/ton gold) is much lower than the grade at Snip (22g/ton gold). The lower grade at Detour Lake allows for a 2-3 meter remnant sill pillar to be left insitu to hold backfill in place; conversely, 100% ore recovery is expected with the high grade values at Snip.

Considering the training of personnel, new and modified equipment that may be needed, and other costs that are associated with implementing addition mining methods for sill pillar recovery, it becomes evident that the same mechanized and conventional cut and fill mining methods currently used at Snip should be utilized for mining under a sill mat.

In conventional stopes (spans less than 3 meters), the stability of sill mats and good back support has enabled mining of sill pillars by cut and fill with a final pillar thickness of less than 3 meters. Conversely, the width of the mechanized stopes do not allow good back support for wedges in a 3-4 meter thick sill pillar. Rock bolting into a sill mat will not provide proper support for wedges that may form along joints parallel to the ore body. For safety considerations, the longhole method may be the best method for a stope having a wide span, especially if the quality of the cemented backfill is doubtful. Longhole prevents miners from having to enter the stope during the muck cycle. However, as mentioned in section 7.3.4, panel mining of sill pillars with cut and fill methods is feasible.

The current mining sequence at Snip has been used to successfully approach a sill mat. Hangingwall and footwall control was not found to be a problem. Consequently, the mining method for conventional and mechanized stopes could remain as cut and fill and longhole, respectively. In larger mechanized stopes, the sill pillars should be removed in panels. Then the
panel should be backfilled to the sill mat with cemented backfill. Drainage will be the most important factor for ensuring minimal dilution between adjacent panels.

8.2 Sill Pillar Design

The dimensions of the final sill pillar need to be addressed to ensure a safe working environment in cut and fill mining employing a man entry method. Stress analysis indicates that stress related problems are not a major concern for Snip sill mat design. Stress is not a problem since the ore body is close to surface (Figure 2.0.A). Structure has not hindered overhand cut and fill mining. Therefore, the rock mass rating (RMR) is considered in conjunction with Modified Mathew’s Method and Detour Lake Mine Method for optimum sill pillar dimensions.

8.2.1 Modified Mathew’s Method

Modified Mathew’s Method is an empirical stope design method - for information on applying this method see Potvin and Milne, (1992). Table 8.2.1.A lists the stope geometry that is assumed for conventional and mechanized stopes. The stability number for Mathew’s Modified Method is also given in Table 8.2.1.A.

| Table 8.2.1.A: Modified stability method (Potvin and Milne, 1992). |
|-----------------|-----------------|-----------------|-----------------|-----------------|
| MECHANIZED STOPE | CONVENTIONAL STOPE |
| Span | 12 | Span | 5 |
| Strike Length | 90 | Strike Length | 30 |
| Shape Factor = (12*90)/(24+180) | 5.3 | Shape Factor = (5*30)/(10+60) | 2.1 |
| Rock Stress Factor (A) | 0.4 | Rock Stress Factor (A) | 0.4 |
| Joint Orientation Factor (B) | 0.55 | Joint Orientation Factor (B) | 0.55 |
| Gravity Factor (C) | 2 | Gravity Factor (C) | 2 |
| RMR | 50 | RMR | 50 |
| Q' | 1.95 | Q' | 1.95 |
| Stability Number (N') | 0.9 | Stability Number (N') | 0.9 |
The RMR of the back is assumed at 50%; this is heavily influenced by the weaker ore. The strike and span for each stope are estimated from wider stopes seen on site. The following assumptions for a conservative approach are made for Factors A, B, C:

- Factor A - From previous stress analysis, the induced stress in a sill pillar is 15 MPa and the uniaxial compressive strength of the BSU unit is assumed at 75 MPa.
- Factor B - A joint parallel to the ore body is the most critical since this is the dominant structure for the Snip mine (Section 2.2).
- Factor C - The back is horizontal.

Figure 8.2.1.A: Modified Stability Graph (Potvin et al., 1992) illustrating the stability of conventional and mechanized stopes at Snip mine.

In this case, the Modified Stablility Method is applied to the back only to determine the stability of a sill pillar as mining approaches the final lift under a sill mat. The shape factor (S) is calculated as follows:

\[
S(\text{back}) = \frac{\text{Span} \times \text{Strike Length}}{2 \times (\text{Span} + \text{Strike Length})}
\]
The Modified Stability Number (N) is a product of the factors Q' (NGI Rock Mass Rating), A (Rock Stress Factor), B (Rock Defect Orientation Factor), and C (Orientation of Design Surface):

\[ N = Q' \times A \times B \times C \]

On Figure 8.2.1.A, the mechanized stope plot of S versus N falls into the support required/potentially unstable area. A point plotted for the conventional stope lies in the stable region. This agrees with back support in the field - mechanized stopes require support and conventional stopes require support where needed.

### 8.2.2 Detour Lake Mine Method

Detour Lake Mine collected data for cut and fill stopes relating RMR to unsupported spans in the back. Span is defined as the largest circle in the back and unsupported span refers to spans with no support or support which includes pattern rockbolting - 1.8 m long bolts on 1.2 m x 1.2 m pattern (CANMET, 1992). With a back RMR of 50%, Figure 8.2.2.A predicts an unsupported span of 5 meters.

![Rock Mass Rating Chart](image)

**Figure 8.2.2.A:** Detour Lake Mine Stability Graph (CANMET, 1992).

With the use of back support, mechanized stopes can increase mining spans to greater than 5 meters. All methods used to predict spans for pillar design indicate current sill pillar designs...
allow the current mining methods to be used for sill pillar recovery; furthermore, man entry
below a properly designed sill mat will be safe. However, in the case of large mechanized
stopes approaching 10 to 15 meters in span, sill pillars may have to be mined in panels as
suggested in section 7.0 since the critical span has been exceeded. The critical span is the span
in which the backfill load exceeds the support strength of the sill mat.
9.0 Conclusion

After reviewing various sill mat support designs (section 1.0), cable slings are the best method of support for sill mats at the Snip Mine. Figure 9.0.A illustrates the final sill mat design. The installation of the sill mat is described in section 3.0. The current design should continue to be used at the Snip mine. In order to aid in drainage and decanting of the cemented backfill,

"Big O" pipes should be installed on the sill below the cable slings. Above the cable slings, weld wire mesh should be attached to the cables with TYCLIPS. A TYCLIP spacing of 30 centimeters (12 inches) will be strong enough to maintain support for most sill mats. However, the procedure outlined in section 7.3.5 will help increase the TYCLIP spacing for sill mats with smaller spans (less than 5 meters) while utilizing the full strength of the weld wire mesh and the TYCLIPS.

Above the weld wire mesh, geotextile material should be placed to maintain cemented backfill and uncemented backfill above the cables. Evidence also suggests that the geotextile provides
an excellent barrier to cement and prevents excess cement bleed from the geotextile (section 4.0). The current geotextile (Model TS 700) that is used by the Snip Mine is manufactured by Polyfelt and distributed by Layfield Plastics. Mullen Burst (ASTM) tests indicate geotextile strength capable of withstanding 2750 kPa (400 psi) of pressure, well above the pressure exerted on a sill mat. However, installation procedures underground are generally rugged; as a result, the same geotextile should be used. Degradation may occur from installation since ripping and tearing is likely to occur for a weaker material. The TS 700 geotextile currently used proved to be a tough and resilient material upon installation.

Once the cemented backfill is poured, “BIG-O” pipes should be laid on top of the cemented backfill. This will aid in drainage of backfill since water pressure increases the vertical load on the sill mat and reduces the arching effect of the backfill (Section 4.4).

The Cable Spacing formula (Section 7.4) provides an initial starting point for sill mat design and it calculates a mining width under current sill mats that utilize cable slings. However, if any characteristics of a stope that make it unique (span increases) and some question as to whether the Cable Spacing formula may be inadequate, a complete design analysis as outlined in the following steps.

**STEP 1 - CABLE SPACING**

\[
\text{Cable Spacing} = \left(1.66 \times 10^4\right) H \left[ \frac{1}{(x_{\text{span}})^2} + \frac{1}{(x_{\text{strike}})^2} \right]
\]

This formula assumes no cemented backfill is used since the performance and quality of insitu cemented backfill is unknown. The formula is also based on a stope dip of 52-53°. Backfill properties for this formula are described in section 4.0.

**STEP 2 - TYCLIP SPACING**

\[
s \left(1 + s^2\right)^{0.5} = \frac{188,740}{x_{\text{span}} \times H \left[ \frac{1}{(x_{\text{span}})^2} + \frac{1}{(x_{\text{strike}})^2} \right]}
\]

0.30 meters (12 inches) TYCLIP spacing used be used on all sill mats in stopes with spans greater than 5 meters. For smaller spans, the TYCLIP spacing can be increased using the above formula.
STEP 3 STOPE MINED OUT - KNOW STOPE DIMENSIONS

\[
\frac{1}{(\text{panel width})^2} = \{ \text{Cable Spacing} / [ (\#A) \ H ] \} - \frac{1}{(\text{panel length})^2}
\]

#A is based on new vertical stress determined from \( \sigma_{\text{MATSS}} \) as outlined in Section 7.0. This number replaces 1.66E-4.

A sample sill mat design problem based on the steps above is worked out in Appendix II.

Further research on sill mats needs to be completed in order to increase the understanding of support systems that are required to support backfill. The cable sling system works well in low stress environments such as the Snip mine. The field performance of the cable slings should be monitored; specifically, the differences between the 1.27 centimeter diameter cable slings and the 1.6 centimeter diameter cable slings should be noted. If possible, when the sill pillar is removed from beneath a sill mat, data should be collected to determine the performance of a sill mat and the vertical pressures exerted on it by backfill.

The effects of blasting a sill pillar below a sill mat need to be addressed. The insitu properties of cemented backfill will change drastically if exposed to blast forces greater than the cemented backfill strength.

Instrumentation on other sill mats should concentrate on piezometers, pressure cells, and tense meters. The piezometers can be used to monitor positive or negative water pressure. Tense meters installed on cable slings will indicate the tension the cable is under. Pressure cells should be installed to gather information about vertical, and horizontal pressure and the relationship between the two; in addition, pressure exerted on the footwall from the backfill should be monitored. Understanding the relationship between the three stresses aids in determining the stress distribution in backfill. The stress distribution relates to the coefficient of lateral earth pressure which influences the condition of the backfill - passive, active, or \( K_o \). Data collected from this project and others, including written works in soil mechanics, indicates the importance of the coefficient of lateral earth pressure in determining vertical and horizontal stress in soil and backfill.
The use of cable slings appears to be an improvement over past sill mat designs. However, other methods of backfill support utilizing other construction materials such as wooden beams can be used if engineering design accounts the properties of those materials. Whatever engineering design is constructed to support backfill, Marcinysyn's Arching Theory Summation Series (MATSS) provides an accurate method of determining the backfill load on a sill mat. MATSS is based upon Terzaghi's arching theory and correlates well to field data. MATSS is a flexible method which accounts for changes in stope geometry and material properties. If cable slings are used for backfill support and the material properties are changed accordingly, then the method outlined in this thesis can be used at other mine sites to determine the cable spacing of cable slings and a mining width of sill pillars beneath a sill mat.

This thesis contributes to sill mat design technology in underground mining. The MATSS approach considers the unit weight of backfill (γ), friction angle (φ), the coefficient of lateral earth pressure (K), and water pressure (positive and negative). In addition, the geometry of a stope is divided into segments to account for dip and width (w) changes which previous researchers considered as a single segment. It must be emphasized that the method of sill mat design that is presented must be augmented by engineering judgment, analysis, observation, and subsequent review of all input parameters to ensure a workable and safe solution to sill mat design.
References


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Wiles, T. (1994) *Map3d*. Version 35. Mine Modelling Limited. 16 Park St. P.O. Box 386, Copper Cliff, Ontario, Canada, P0M 1N0.
APPENDIX I

MATSS
Figure 7.2.2.A: 2656 stope segment geometry for October, 1995 backfill level.
Figure 7.2.2.B: 2656 stope segment geometry for November, 1995 backfill level.
Figure 7.2.2.C: 2656 stope segment geometry for February, 1996 backfill level. Note: 0.79 meters water above cemented backfill.
Table 7.2.4.A: 2656 sill mat stope segments z, x, and Kp values.

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Table 7.2.4.B: 2656 sill mat MATSS values for October stope segments.

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**Table 7.2.4.B:** 2656 sill mat MATSS values for November stope segments.

- **vertical stress (measured):** 0 Pa
- **apparent cohesive strength (c):** 1517 Pa
- **stope width (x1):** 4 m
- **unit weight B/F:** 15000 N/m³
- **friction angle:** 34 degrees
- **z1:** 4.55 m
- **surcharge (q):** 0 Pa
- **Kp:** 2.38

**vertical stress (calculated):** 17283 Pa

- **apparent cohesive strength (c):** 1517 Pa
- **stope width (x2):** 2.37 m
- **unit weight B/F:** 15000 N/m³
- **friction angle:** 34 degrees
- **z2:** 2.28 m
- **surcharge (q):** 17283 Pa
- **Kp:** 2.43

**vertical stress (calculated):** 10234 Pa

- **apparent cohesive strength (c):** 1517 Pa
- **stope width (x3):** 1.69 m
- **unit weight B/F:** 15000 N/m³
- **friction angle:** 34 degrees
- **z3:** 1.73 m
- **surcharge (q):** 10234 Pa
- **Kp:** 2.43

**vertical stress (calculated):** 6927 Pa

---

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Table 7.2.4.B: 2656 sill mat MATSS values for November stope segments, continued

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Table 7.2.4.B: 2656 sill mat MATSS values for February stope segments.

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Feb-96 19.6 meters vertical fill above cable slings
*** Read with RS Tech Readout unit
### Table 7.2.4.B: 2656 sill mat MATSS values for February stope segments, continued

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<td></td>
</tr>
</tbody>
</table>
Table 7.2.4.B: 2656 sill mat MATSS values for February stope segments, continued

<table>
<thead>
<tr>
<th>7</th>
<th>apparent cohesive strength (c)</th>
<th>1517 Pa</th>
<th>*** negative pore pressure</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>stope width (x7)</td>
<td>1.91 m</td>
<td></td>
</tr>
<tr>
<td></td>
<td>unit weight CB/F</td>
<td>15000 N/m³</td>
<td></td>
</tr>
<tr>
<td></td>
<td>friction angle</td>
<td>34 degrees</td>
<td></td>
</tr>
<tr>
<td></td>
<td>z7</td>
<td>0.4 m</td>
<td></td>
</tr>
<tr>
<td></td>
<td>surcharge (q)</td>
<td>9468 Pa</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Kp</td>
<td>2.43</td>
<td></td>
</tr>
<tr>
<td></td>
<td>vertical stress (calculated)</td>
<td>8647 Pa</td>
<td></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>8</th>
<th>apparent cohesive strength (c)</th>
<th>1517 Pa</th>
<th>*** negative pore pressure</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>stope width (x7)</td>
<td>1.78 m</td>
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<td>unit weight CB/F</td>
<td>15000 N/m³</td>
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</tr>
<tr>
<td></td>
<td>friction angle</td>
<td>34 degrees</td>
<td></td>
</tr>
<tr>
<td></td>
<td>z8</td>
<td>0.33 m</td>
<td></td>
</tr>
<tr>
<td></td>
<td>surcharge (q)</td>
<td>8647 Pa</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Kp</td>
<td>2.43</td>
<td></td>
</tr>
<tr>
<td></td>
<td>vertical stress (calculated)</td>
<td>7997 Pa</td>
<td></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>9</th>
<th>apparent cohesive strength (c)</th>
<th>1517 Pa</th>
<th>*** negative pore pressure</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>stope width (x7)</td>
<td>1.64 m</td>
<td></td>
</tr>
<tr>
<td></td>
<td>unit weight CB/F</td>
<td>15000 N/m³</td>
<td></td>
</tr>
<tr>
<td></td>
<td>friction angle</td>
<td>34 degrees</td>
<td></td>
</tr>
<tr>
<td></td>
<td>z9</td>
<td>0.33 m</td>
<td></td>
</tr>
<tr>
<td></td>
<td>surcharge (q)</td>
<td>7997 Pa</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Kp</td>
<td>2.43</td>
<td></td>
</tr>
<tr>
<td></td>
<td>vertical stress (calculated)</td>
<td>7312 Pa</td>
<td></td>
</tr>
</tbody>
</table>
### Table 7.2.4.B: 2656 sill mat MATSS values for February stope segments. continued

<table>
<thead>
<tr>
<th>11</th>
<th>apparent cohesive strength (c)</th>
<th>1517 Pa</th>
<th>*** negative pore pressure</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>stope width (x7)</td>
<td>1.31 m</td>
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<td>unit weight CB/F</td>
<td>15000 N/m^3</td>
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<td></td>
<td>friction angle</td>
<td>34 degrees</td>
<td></td>
</tr>
<tr>
<td></td>
<td>z10</td>
<td>0.3 m</td>
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<tr>
<td></td>
<td>surcharge (q)</td>
<td>6633 Pa</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Kp</td>
<td>2.43</td>
<td></td>
</tr>
<tr>
<td></td>
<td>vertical stress (calculated)</td>
<td>5807 Pa</td>
<td></td>
</tr>
<tr>
<td>12</td>
<td>apparent cohesive strength (c)</td>
<td>0 Pa</td>
<td>***saturated zone</td>
</tr>
<tr>
<td></td>
<td>stope width (x8)</td>
<td>2.6 m</td>
<td></td>
</tr>
<tr>
<td></td>
<td>unit weight CB/F</td>
<td>10200 N/m^3</td>
<td></td>
</tr>
<tr>
<td></td>
<td>friction angle</td>
<td>0.01 degrees</td>
<td></td>
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<tr>
<td></td>
<td>z11</td>
<td>0.79 m</td>
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</tr>
<tr>
<td></td>
<td>surcharge (q)</td>
<td>5807 Pa</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Kp</td>
<td>1</td>
<td></td>
</tr>
<tr>
<td></td>
<td>vertical stress (calculated)</td>
<td>13864 Pa ***calculated stress = effective stress</td>
<td></td>
</tr>
<tr>
<td></td>
<td>water pressure</td>
<td>7791</td>
<td></td>
</tr>
<tr>
<td></td>
<td>MATSS + water pressure</td>
<td>21655</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Fj#2 (measured)</td>
<td>18272 Pa</td>
<td></td>
</tr>
<tr>
<td></td>
<td>measured theory</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>Fj#1 Vertical Stress</td>
<td></td>
<td></td>
</tr>
<tr>
<td>13</td>
<td>cohesive strength (c)</td>
<td>0</td>
<td>0 ***saturated zone</td>
</tr>
<tr>
<td></td>
<td>stope width (x9)</td>
<td>2.4</td>
<td>2.4</td>
</tr>
<tr>
<td></td>
<td>unit weight CB/F</td>
<td>14190 14190 ***unit weight - bouyant force</td>
<td></td>
</tr>
<tr>
<td></td>
<td>friction angle</td>
<td>0.01 0.01 degrees</td>
<td></td>
</tr>
<tr>
<td></td>
<td>z12</td>
<td>1.2</td>
<td>1.2 m</td>
</tr>
<tr>
<td></td>
<td>surcharge (q)</td>
<td>18272 21655 Pa</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Kp</td>
<td>1</td>
<td>1 ***saturated zone</td>
</tr>
<tr>
<td></td>
<td>vertical stress (calculated)</td>
<td>35295</td>
<td>38678 Pa</td>
</tr>
</tbody>
</table>

***no measured value Fj#1
Table 7.2.5.B: 4867 sill mat MATSS values for 1 lift backfill.

<table>
<thead>
<tr>
<th>Oct./95</th>
<th>3.55 meters vertical fill above pressure cell</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>0.55 meters drained</td>
</tr>
<tr>
<td></td>
<td>3.0 meters saturated</td>
</tr>
</tbody>
</table>

\[ i=1 \]
- apparent cohesive strength (c): 1517 Pa
- stope width (x8): 10 m
- unit weight B/F: 15000 kN/m\(^3\)
- friction angle: 34 degrees
- \( z_1 \): 0.55 m
- surcharge (q): 0 Pa
- \( K_0 \): 0.44

vertical stress (calculated): 7953 Pa

***calculated stress = effective stress

measured/ theory

\[ i=2 \]
- cohesive strength (c): 0 Pa
- ***no neg. pore pressure
- stope width (x2): 10 m
- unit weight B/F: 10200 kN/m\(^3\)
- ***unit weight = bouyant weight
- friction angle: 0.001 degrees
- \( z_2 \): 3 m
- surcharge (q): 7953 Pa
- \( K \): 1

vertical stress (calculated): 38552 Pa

water pressure: 29430 Pa (3 meters H2O)

vertical stress (calculated): 67982

vertical stress (measured): 68261
APPENDIX II

Sample Sill Mat Design
Problem: At the Snip mine, an undercut of a cut and fill mining block is mined to a span of 10 meters. A sill mat is to be installed having a 10 meter span and a 50 meter strike. If 1.6 centimeter diameter cable (258,000 N maximum tensile strength) is used, what is the installation spacing of cable slings installed along the span and strike directions?

**STEP I (section 9.0)**

Cable Spacing = (1.66E-4) H (1/span² + 1/strike²)

= (1.66E-4) (258,000) (1/10² + 1/50²)

= 0.43 meters.

After discussions with production, engineering, and geology, it is determined to use a wider cable spacing to be more cost effective. According to exploration drill holes on the level above, the ore body narrows to spans less than 10 meters above the sill mat. As a result, it is decided to use a 1.5 meter cable spacing for the sill mat.

**STEP II (section 9.0)**

This is a large sill mat; therefore, the suggested TYCLIP spacing of 30 centimeters is used to attach the weld wire mesh to the cable slings.

**STEP III (section 9.0)**

After mining the stope to the next level, the dimensions of the stope are known (Figure IIA)

![Figure IIA: Stope dimensions after mining - example problem.](image-url)
and mining widths are wider than expected. The vertical load on the sill mat is expected to be higher than originally calculated; as a result, a second iteration of MATSS needs to be completed with the new known stope dimensions. The dimensions from Figure IIA are substituted into MATSS (n = 3) with the following backfill parameters and no cement segment:

- friction angle - 34 degrees
- negative pore water pressure - 1.5 kPa
- unit weight - 15,000 N/m³.

<table>
<thead>
<tr>
<th>Segment</th>
<th>apparent cohesive strength (c)</th>
<th>1500 Pa</th>
<th>*** negative pore pressure</th>
<th>stope width (x1)</th>
<th>20 m</th>
<th>unit weight CB/F</th>
<th>15000 N/m³</th>
<th>friction angle</th>
<th>34 degrees</th>
<th>z1</th>
<th>10 m</th>
<th>surcharge (q)</th>
<th>0 Pa</th>
<th>Kp (embankment)</th>
<th>2.9</th>
<th>***Figure 7.2.1.C, section 7.2.1</th>
</tr>
</thead>
<tbody>
<tr>
<td>vertical stress (calculated)</td>
<td>65182 Pa</td>
<td>***Equation IV, section 7.1</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
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<td></td>
<td></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Segment</th>
<th>apparent cohesive strength (c)</th>
<th>1500 Pa</th>
<th>*** negative pore pressure</th>
<th>stope width (x2)</th>
<th>15 m</th>
<th>unit weight CB/F</th>
<th>15000 N/m³</th>
<th>friction angle</th>
<th>34 degrees</th>
<th>z1</th>
<th>15 m</th>
<th>surcharge (q)</th>
<th>65182 Pa</th>
<th>Kp (embankment)</th>
<th>2.43</th>
<th>***Figure 7.2.1.C, section 7.2.1</th>
</tr>
</thead>
<tbody>
<tr>
<td>vertical stress (calculated)</td>
<td>67626 Pa</td>
<td>***Equation IV, section 7.1</td>
<td></td>
<td></td>
<td></td>
<td></td>
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<td></td>
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</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Segment</th>
<th>apparent cohesive strength (c)</th>
<th>1500 Pa</th>
<th>*** negative pore pressure</th>
<th>stope width (x3)</th>
<th>10 m</th>
<th>unit weight CB/F</th>
<th>15000 N/m³</th>
<th>friction angle</th>
<th>34 degrees</th>
<th>z1</th>
<th>3.8 m</th>
<th>surcharge (q)</th>
<th>67626 Pa</th>
<th>Kp (embankment)</th>
<th>2.54</th>
<th>***Figure 7.2.1.C, section 7.2.1</th>
</tr>
</thead>
<tbody>
<tr>
<td>vertical stress (calculated)</td>
<td>49625 Pa</td>
<td>***Equation IV, section 7.1</td>
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<td></td>
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<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
The known stope dimensions give a vertical stress of 49.6 kPa on the cable slings. This is higher than the predicted pressure (section 7.3.1) if a stope has a constant span of 10 meters:

\[ \text{Predicted pressure} = 3625.8 \times (\text{Span}) = 3625.8 \times 10 = 36.3 \text{ kPa}. \]

Since the pressures exerted on the sill mat are greater than originally predicted, a panel width will have to be determined for mining the sill pillar beneath the sill mat (STEP 3, section 9.0).

The linear support equation (section 7.3.3) must be adjusted for a 10 meter span:

\[
\omega_{\text{linear}} = 7.63 \times d_{\text{span}} \times H \left[ \frac{1}{(x_{\text{span}})^2} + \frac{1}{(x_{\text{strike}})^2} \right] \text{ N/m} \\
= 7.63 \times (0.0791 \times 10) \times H \left[ \frac{1}{(x_{\text{span}})^2} + \frac{1}{(x_{\text{strike}})^2} \right] \\
= 6.04 \times H \left[ \frac{1}{(x_{\text{span}})^2} + \frac{1}{(x_{\text{strike}})^2} \right] \text{ N/m.}
\]

An equation for calculating a mining width (panel) can be determined as outlined in section 7.3.4.

\[
\omega_{\text{linear}} = 6.04 \times H \left[ \frac{1}{(x_{\text{span}})^2} + \frac{1}{(x_{\text{strike}})^2} \right] \\
\sigma_{\text{vertical}} = 49,600 \text{ Pa}
\]

Cable Spacing = \frac{\omega_{\text{linear}}}{\sigma_{\text{vertical}}} \\
= 6.04 \times H \left[ \frac{1}{(x_{\text{span}})^2} + \frac{1}{(x_{\text{strike}})^2} \right] / 49,600 \\
= 1.22 \times 10^{-4} \times H \left[ \frac{1}{(x_{\text{span}})^2} + \frac{1}{(x_{\text{strike}})^2} \right]
\]

Know,

- cable spacing installed = 1.5 meters
- maximum cable tension = 258,000 N
- strike length = 50 meters.

Substituting,

\[
1.5 = 1.22 \times 10^{-4} \times (258,000) \left[ \frac{1}{(50)^2} + \frac{1}{(\text{panel})^2} \right] \\
\text{panel} = 4.6 \text{ meters.}
\]

The sill pillar (10 meter span) could be mined in 2 panels of 5 meters. Discussions between geology, engineering, and production should be able to determine the best method of mining the sill pillar in panels.