# APPENDIX A

# *CROWN PILLAR ASSESSMENT*

Report to:

Tantalum Mining Corporation of Canada Ltd. a Cabot Corporation Company

# Crown Pillar Assessment and Remediation Plan for the Tanco Mine, Bernic Lake, Manitoba

Document No. 1301660300-REP-R0001-04

Report to:

# TANTALUM MINING CORPORATION OF CANADA LTD.
# A CABOT CORPORATION COMPANY

# CROWN PILLAR ASSESSMENT AND REMEDIATION PLAN FOR THE TANCO MINE, BERNIC LAKE, MANITOBA

JUNE 2013


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- Mr. Will Brits, General Manager
- Mr. Dave Owens, Exploration Geologist
- Mr. Claude Deveau, Technical Service Superintendent
- Dr. Wen Wu, Rock Mechanics Engineer
- Mr. Jining Zhong, Mine Engineer
- Mr. Scott Rankmore, Mine Technologist
- Mr. Dan Boswick, Geologist
- Mr. Rocky Aitkenhead, Mine Foreman
- Ms. Sharon Inkster, Safety, Health and Environment Manager

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# GLOSSARY

## *UNITS OF MEASURE*

| | |
|---|---|
| cubic foot | $ft^3$ |
| cubic metre | $m^3$ |
| day | d |
| decibel | dB |
| degree | ° |
| dollar (American) | US$ |
| dollar (Canadian) | Cdn$ |
| foot | ft |
| gallons per minute (US) | gpm |
| gigapascal | GPa |
| inch | in |
| kilo (thousand) | k |
| kilogram | kg |
| kilometre | km |
| kilotonne | kt |
| megapascal | MPa |
| million | M |
| million tonnes | Mt |
| minute (time) | min |
| percent | % |
| second (time) | s |
| short ton (2,000 lb) | st |
| three-dimensional | 3D |
| tonne (1,000 kg) (metric ton) | t |
| tonnes per day | t/d |

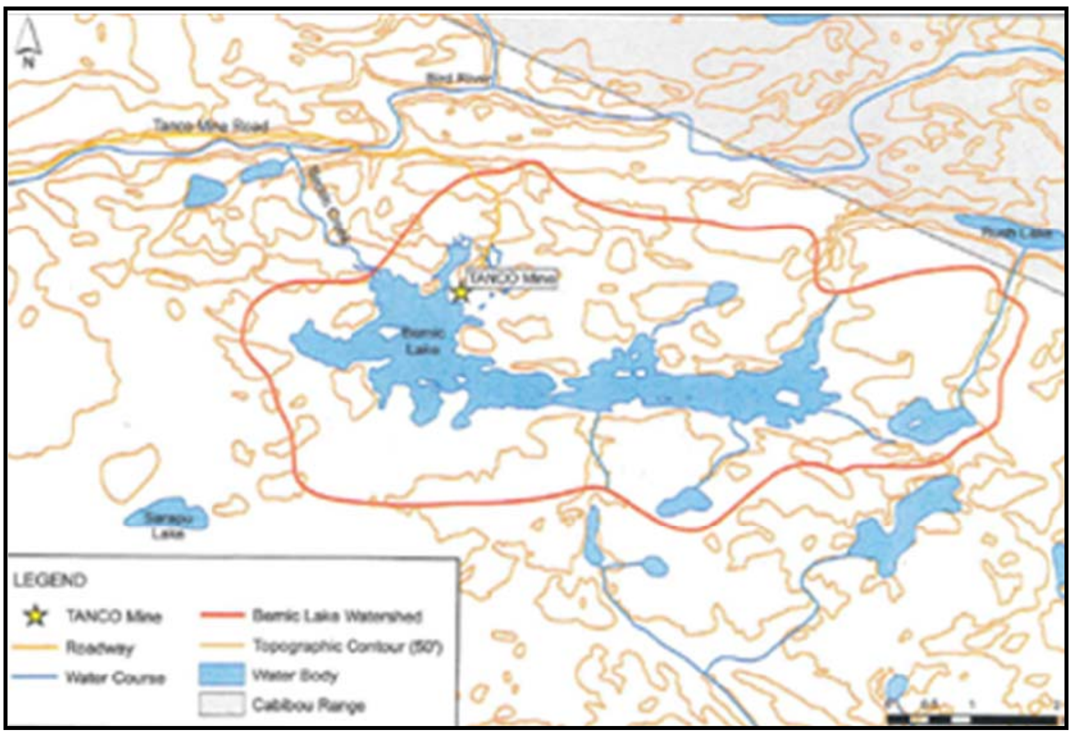
## ABBREVIATIONS AND ACRONYMS

| | |
|---|---|
| Tantalum Mining Corporation of Canada Ltd | Tanco |
| fall of ground | FOG |
| Cavity Monitoring System | CMS |
| US Bureau of Mines | USBM |
| Norwegian Geotechnical Institute | NGI |
| Manitoba Hydro | MBH |
| Notice of Alteration | NOA |
| Environmental Act Proposal | EAP |
| Fisheries and Oceans Canada | DFO |
| total suspended solids | TSS |
| Metal Mining Effluent Regulations | MMER |
| environmental effect monitoring | EEM |

# 1.0 INTRODUCTION AND SUMMARY

At the request of the Tantalum Mining Corporation of Canada Ltd. (Tanco), Jacques Ouellet and Andrew MacKenzie of Tetra Tech attended a site visit at the Tanco Mine on April 23 and 24, 2013. The Tanco Mine is located approximately 190 km north of the city of Winnipeg. Surface facilities for the mine are situated near the northwestern shore of Bernic Lake (see Figure 1.1). Tanco Mine's resource orientation can be described as an anticlinal reef lying 35 m (100 ft) to 200 m (600 ft) below surface. Half of the defined resource is located below Bernic Lake.

**Figure 1.1 Tanco Mine Location Relative to the Bernic Lake Watershed**




The initial scope of work assigned to Tetra Tech included the independent empirical assessment of the crown pillar where a recent fall of ground (FOG) had raised concern. Continuing scope items include assistance and recommendations to Cabot Corp. regarding concerns of the mines stability

During the site visit Tetra Tech representatives met with Tanco Mine staff including Will Brits, General Manager; Dave Owens, Exploration Geologist; Claude Deveau, Technical Service Superintendent; Wen Wu, Rock Mechanics Engineer; Jining Zhong, Mine Engineer; Scott Rankmore, Mine Technologist; Dan Boswick, Geologist; Rocky Aitkenhead, Mine Foreman; and Sharon Inkster, Safety, Health and Environment

Manager. Although most of the staff has less than two years of experience at the Tanco site, they demonstrated a high level of competence and were well informed. The Tanco team was helpful in providing Tetra Tech with operational information and technical evidence. The Tanco Mine technical staff shared resource, geotechnical and maintenance information in order for the Tetra Tech team to complete their assessment and recommendations as embodied in this report.

An underground tour allowed the Tetra Tech team to observe the FOG area from at least four different viewpoints. Discussions held during the underground tour confirmed the circumstances already described in the Golder Associates report entitled "Project # 13-1193-004, Site Visit and Evaluation of fall of Ground (March 2013)", by Dennis O'Donnell and Sandra Smith.

Tetra Tech observed that the current mining method included ongoing work to recover remnants of ore located in pillars and sills. Pillars and sills did not appear to be failing or compromised outside the area of the FOG.

In summary, this report focusses on the likely scenario of progressive failure of the crown pillar over time and also discusses the unlikely worst case scenario of catastrophic failure.

We report on mitigating activities by the mine's management including installation of instrumentation (microseismic systems), and a new "soft blast" mining approach anticipated to reduce the rate at which damage occurs in the host rock.

The report acknowledges the mines appropriate installation of monitoring systems and instrumentation intended to give early warning to personnel. The hierarchy of which is summarized as:

1. Microseismic System – Acts as first indicator of energy focused in the crown pillar. Typical warnings of falls of ground are between 0.5 to 2 days. This system is being monitored constantly by ESG Solutions – experts in the science of micro-seismic.
2. Stress cells – Immediate warnings if the crown begins to relax and load supporting pillars.
3. Extensometers – Progressive warning of yield or closure between pillars.
4. Water conductivity meter – expected to deliver trend warnings as failure progressively occurs.
5. Optimus Noise Monitor – expected to give trend warning as the crown pillar unravels over time.
6. Cavity Monitoring System (CMS) – Used to assess the progressive unravelling of the crown. 3D images are superimposed against previous CMS measurements to quantify the amount of material change.

Data trends from these types of systems, when properly interpreted, have demonstrated a high degree of certainty and early warning of changing ground conditions at many underground mines in North America. Our expectation is that these systems will provide the same type of early warning of progressive failure at Tanco Mine.

Tetra Tech has confidence that Tanco management is reacting appropriately to isolate and monitor the crown pillar as well as it has developed new blasting and evacuation procedures and provided training to their mine personnel.

Based on the observations of the authors, the information made available, and a strict adherence to the recommendations provided herein we see no reason to preclude the continuation of the underground mining operation.

This report also discusses mitigation options which we strongly believe should be considered in order to continue long term mining at the Tanco Mine. These options are focused on constructing dikes so that the area above the mine can be dewatered. Removal of the water will result in reduced load on the crown and will eliminate the risk of flooding.

# 2.0 CROWN PILLAR ASSESSMENT

## 2.1 SUCCINCT REVIEW OF AVAILABLE DATA

To assist with the assessment of the crown pillar, the following set of reports and documents were provided by Tanco as a complement to the site visit observations and staff interviews:

- Tanco Mine Ground Stability Study, by Golder Associates, April 12, 2012
- Technical memorandum of site visit by Golder Associates, dated March 2013
- Internal Tanco study conducted by mine staff over the area
- Mining study over pillar reduction and sill mining by J. D. Smith (1996)
- Microsoft PowerPoint® format files providing underground layout maps, sections (CMS), sound level tracking data, and pictures.

These documents provided the basic information to aid in the understanding of the current situation in relation to the crown pillar stability. For the purpose of this brief review, only the information relevant to the crown pillar stability evaluation including pillar width to height ratio and sill pillar mining is included.

The properties of the rock mass are essential in conducting any assessment of an underground cavity. For the purpose of this study no mapping, sampling or testing work was done. During the site visit a basic visual inspection of the various rock units present in the mine area was conducted. This visual inspection permitted the Tetra Tech team to appreciate the general conditions in the mine and develop confidence with the data presented in the reports and documents cited above. The following is a summary of the basic data considered.

### 2.1.1 GENERAL GEOTECHNICAL DATA

In situ stresses were measured in 1996 by Atomic Energy of Canada Ltd. using the US Bureau of Mines (USBM) borehole deformation gage system. The following results were produced:

- Principal Stress (horizontal, azimuth 220): 34 MPa
- Minor principal Stress (vertical): 11 MPa
- Intermediate principal stress (horizontal, azimuth 130): 21.7 MPa.

A fact that must be noted from these results is that the ratio of the horizontal to vertical stress is very high (3:1). Although it is common to see higher horizontal stress to vertical stress at shallow depth in the Canadian Shield, this ratio is usually of about 2:1.

### 2.1.2 Site Observations

Ground water was observed in the area of concern. Water seepage is apparent through fractures within the fall of ground area. Close to this area the Tetra Tech team observed a few bolts in the roof that were showing water leakage. There is no information that would allow the postulation of a causal effect with the fall of ground. Nevertheless, these fractures point to the existence of some water flow through the rock structures in that location.

**Figure 2.1 Underground Picture of the Affected Roof**

Source: courtesy of W. Brits

### 2.1.3 Rock Properties in the 14-P Area

- The host rock in the back is formed of amphibolite and gabbro. This gabbro contains shear planes and the in situ observations showed that these planes allow some water seepage. Moreover, a clay infilling material is present and is very sensitive to moisture, which indicates that an increase of water flow through these shear planes would impact significantly on the shear strengths along these structures. The rock in the back is blocky with at least three joints sets. The combination of clay and water in these structures has a detrimental effect on the rock mass strength for this rock unit. There is a major joint set with a dip angle of 30° to 45° that creates layers in the rock mass. Data, provided by the mine personnel, shows that these rock layer thicknesses can vary widely from one location to another. In the area of the fall of ground this thickness ranges from 0.5 m to 7 m. Considering the lack of detailed data it is difficult to assess the conditions of the rock mass. But, based on the information available and visual inspection, it is estimated that this rock unit would give a $RMR_{76}$ rating ranging from 40 to 50, or what could be called "Fair" rock conditions according to the classic Bieniawski rock classification system (1976). According to the data provided in the Golder Associates report, compressive strength for the rock in this unit is approximately 170 MPa, Poisson Coefficient of 0.23 and a Young's modulus of 77 GPa.

- The mining horizon is constituted primarily of pegmatite. This rock is neither faulted nor fractured and shows a rock mass rating of "very good" quality ($RMR_{76}$ 80+) as a preliminary assessment. The intact rock properties according to Golder Associated have a compressive strength of 135 MPa, Poisson coefficient of 0.18 and a Young's modulus of 50 GPa.
- There is a well-defined plane of weakness between the pegmatite and the rock unit in the back. This contact plane was visible in the fall of ground area. Contrary to other areas observed underground during the site visit, the roof has opened all the way to the back rock formation.

## 2.2 Geometrical Observations

### 2.2.1 CMS

The following diagram illustrates the footprint of the underground mine with the fall of ground area (14-P) indicated in grey. It is important to note that the location of this area is under Bernic Lake and is in the vicinity where the lake reaches its' deepest point of approximately 7 m.

**Figure 2.2 Underground Mine Footprint with Fall of Ground Area Identified**

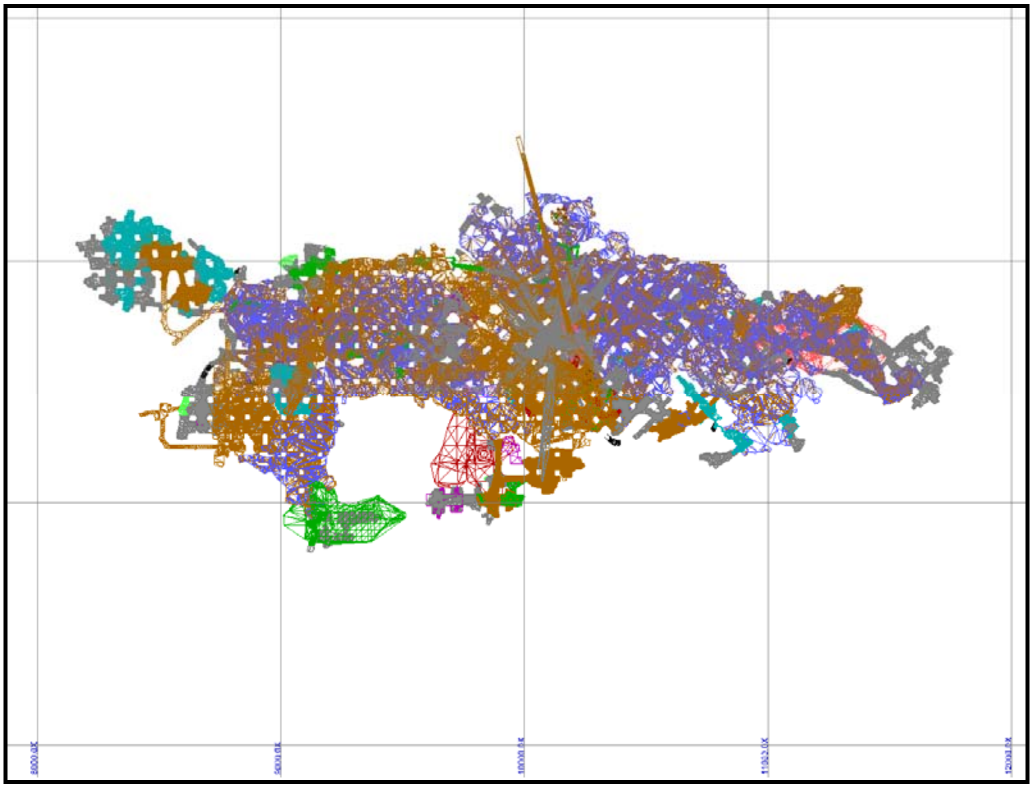
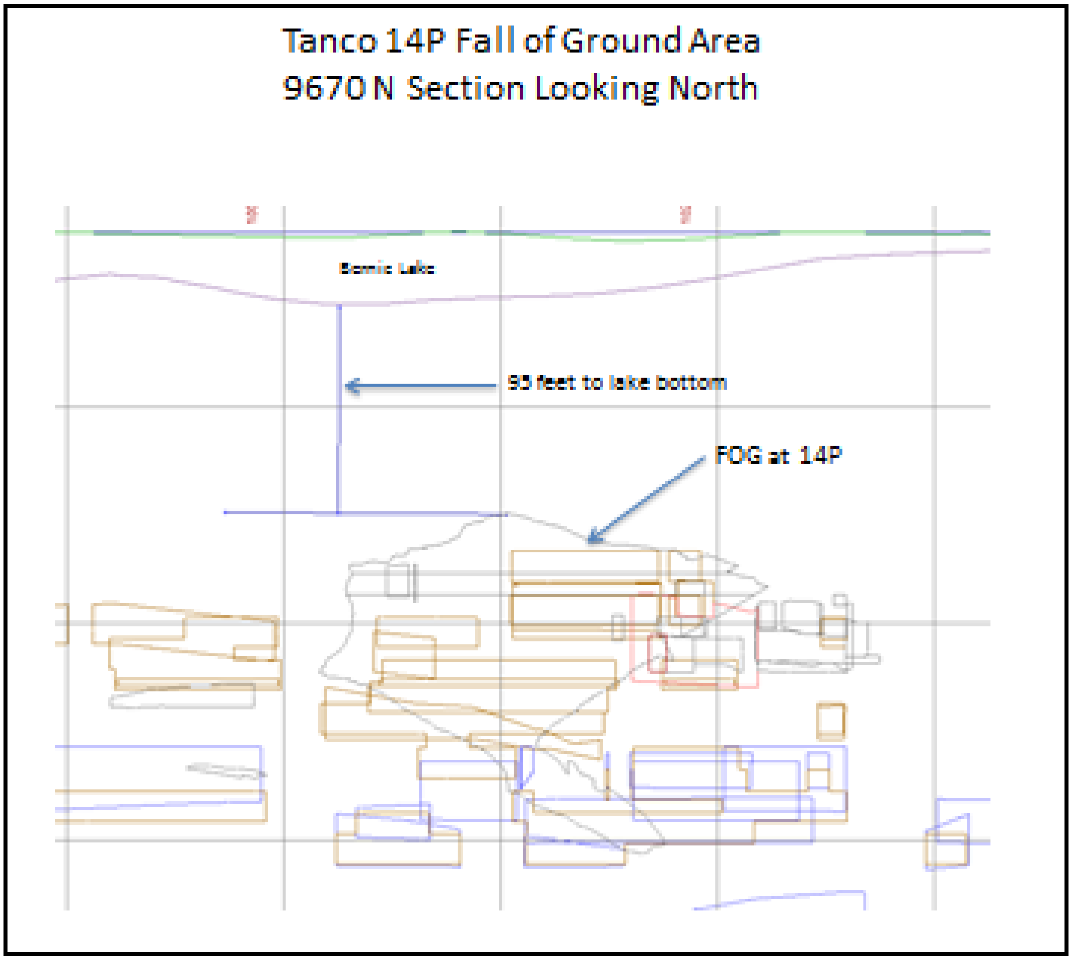


Figure 2.3 illustrates an elevation view of the most recent survey (CMS) provided by the mine staff. In terms of the underground openings, the maximum current span is 50 m (165 ft). The remaining thickness of the crown pillar is 29 m (95 ft).

**Figure 2.3 CMS Reconstructed Section Showing Excavation Geometry**




## 2.2.2 Ground Spalls and Pillar Conditions

A view of the roof conditions in the area of concern is shown in Figure 2.1. On the left hand-side of Figure 2.1 the remaining rock bolts can be observed to be still partially in the roof (9 m (30 ft) in three sections). The sections sticking out would represent the first 3.7 m (12 ft) of these bolts from the face plate when originally installed. In some instances the bolt plate is still in place while for some it has been apparently sheared off. The screen installed at the time of this first rehabilitation work has failed and portions of it are still hanging from the roof. The apparent conditions of the roof and bolts indicate that the rock has failed through an unraveling mechanism. The 3.7 m (12 ft) long bolts had been installed as well in that area. These are most likely the bolts that could be observed in the pile of broken material on the floor.

**Figure 2.4 Close-up Picture Showing Condition of the Failure Zone**

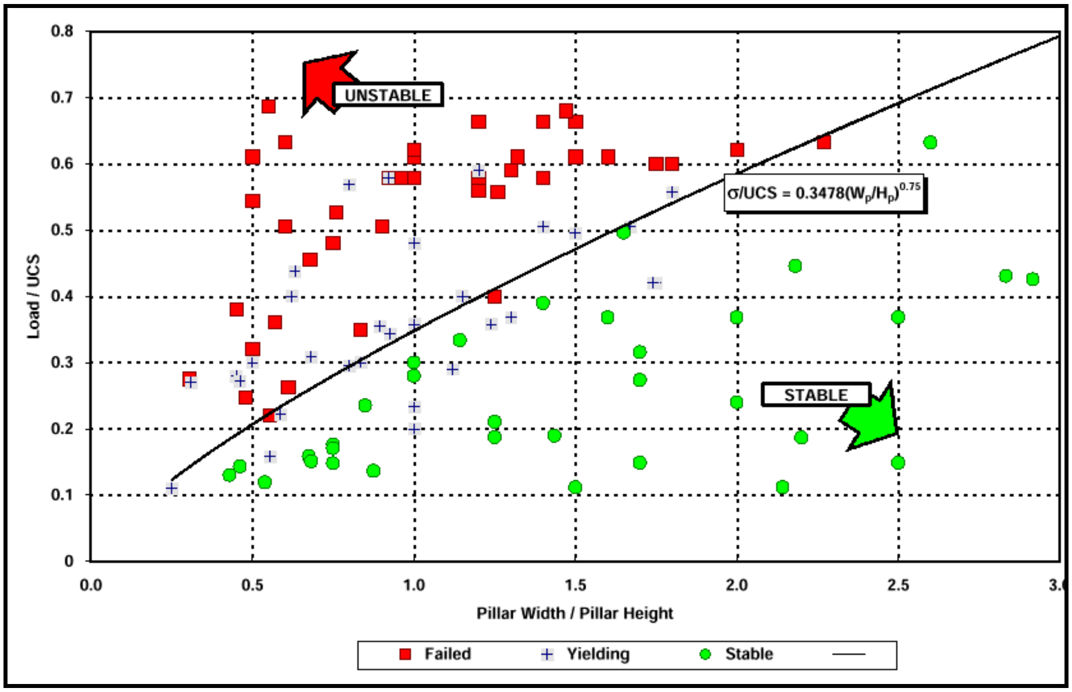
Source: courtesy of W. Brits

To the right hand-side of Figure 2.4 the typical rock unit forming the back (amphibolite and gabbro) can be observed. A closer look at this image shows that the shear faults creates a layering effect in the rock mass. These planes form an angle relative to the horizontal. The drillhole data analysis performed by the mine staff shows that these layers have a thickness that range from 0.5 m to 7 m (1.5 ft to 2 ft). In this picture we can confirm, at least for the first apparent layer, the thickness is approximately 0.5 m (1.5 ft) thick. A dense pattern of rock bolts (reported to be 3.7 m (12 ft) long by the mine personnel) can be observed in Figure 2.4. This in essence means that within that area the first 3.7 m (12 ft) could be considered to be one layer. As will be discussed later, this could be one of the factors contributing to the surprising stability exhibited by the roof so far. At this stage this portion of the roof is still showing good condition and the bolts in place are still looking intact.

Following the J. D. Smith recommendation (1996), the mine reduced the pillar width from 15 m (50 ft) down to 7.5 m (25 ft). This, in essence, reduces the strength of these pillars, increases the effective span of the crown pillars and reduces pillar stiffness. This results in significant stress redistribution in the crown pillars of the mine. This phenomenon translates in displacements in the roof abutments and pillars. These in turn can bring fractures, slippage or relaxation along discontinuities in the crown pillars that will impact their stability. Further to this pillar reduction, the mining began extracting the sill pillar as well. This resulted, in some areas, in creating extremely thin pillars. In the 14-P area (rock mechanic study performed by the mine personnel) the width to height ratio measured ranges from 0.07 to 0.29 with most pillars showing a ratio of less than 0.2. Based on experience and literature on the subject, a width to height ratio that falls below a value of 0.5 is considered potentially unstable. Consequently, this practice, without providing pillar confinement such as backfilling, may have contributed to the present ground issues.

The following figure illustrates a compilation of pillar case studies and the proposed empirical design criteria. As can be seen, all the pillars in the 14-P area are falling well outside of the standard practice. Trying to plot these pillars on this graph, all the points would fall in the yielding/unstable zone. Our field observations show these pillars to be in a remarkable shape. As indicated earlier, the empirical methods provide good guidelines but are not exact. Nevertheless, considering these pillars are well outside usual mining practice, their safety factor can only be marginal at best. In our experience we did encounter cases at other sites where the empirical method was indicating stable conditions but a massive failure occurred as well as the reverse. When a given case falls on the boundary line or well outside of the usual data points, caution is advised before deriving definitive conclusions. Definitely, in such cases the engineer should use more sophisticated means of analysis.

**Figure 2.5 Empirical Method to Assess Pillar Strength after Carter (2002)**




Source: after Carter (2002)

### 2.2.3 Water Inflow

Water infiltrates, seeps, into the mine at particularly conductive geological structures such as slip planes and gouge filled faults and/or mining induced openings such as cable or diamond drill holes in or close to the roof. In particular water was observed in the area of crown pillar instability, area 14-P. Reviews of pumping records from June 1 to 6, 2013 show a mean inflow of 1,100 $m^3$/day (200 gpm). No mining occurred during this time so it is reasonable to conclude that the water is representative of groundwater seepage only. This inflow does not appear to be changing over the period of observation. Mine staff continues to monitor inflows through pumping records and use of an appropriately located conductivity meter. In the event that inflows were to increase, the conductivity meter is triggered to alarm/notify personnel.

### 2.2.4 AUDIO RECORDS

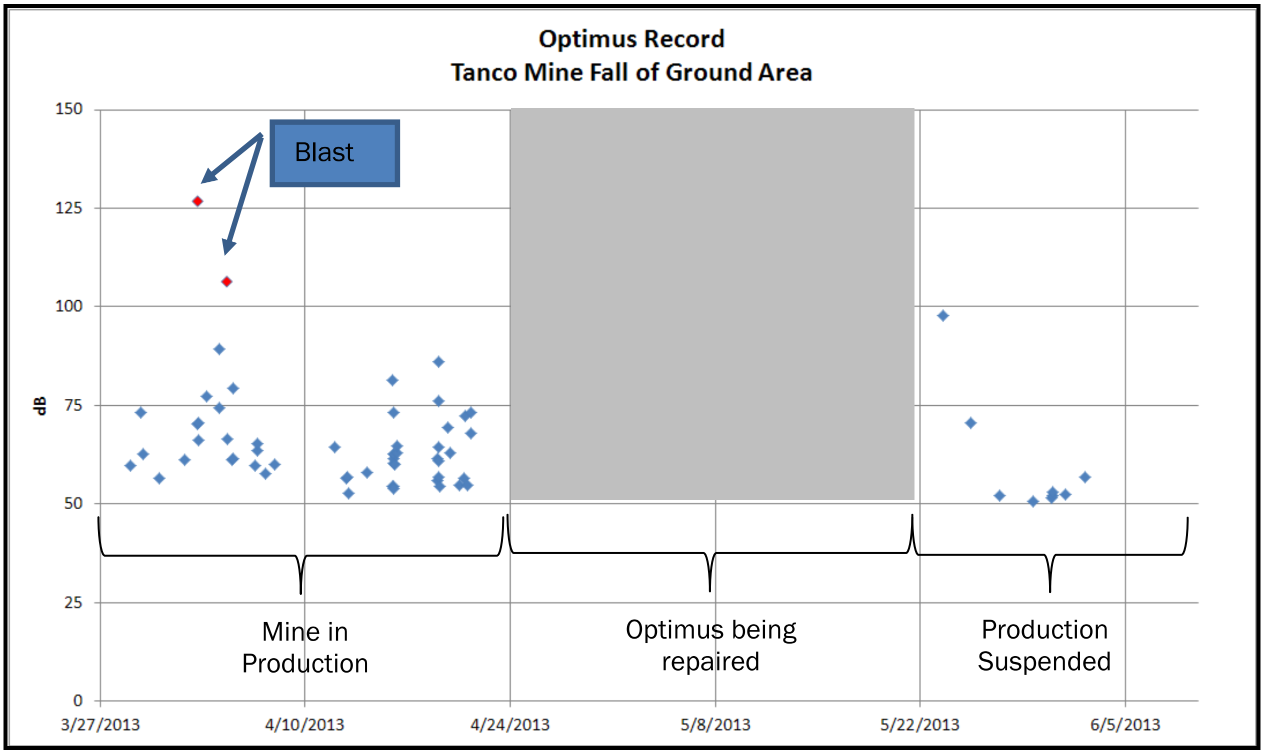
Mine staff installed an Optimus Noise Meter underground to monitor the fall of ground area around March 27, 2013. Since daytime mining activity is loud, the noise meter is set-up to operate only at night.

Noise needs to exceed a threshold of 50 dB before being considered "of interest" and being recorded by the Optimus.

Observations as shown in Figure 2.6, span a period of active mining, a period of no reading, when the Optimus was being repaired, and finally a period of 10 days when there was no mining activity.

While the data indicates no increased activity trend, it does show that the roof is still active.

**Figure 2.6 Nighttime Optimus Decibel Record – March 27 to June 5th, 2013**



### 2.2.5 MICRO-SEISMIC RECORDS

As recommended by Tetra Tech, Tanco has commissioned a micro-seismic monitoring system through ESG. Since its installation, the mine staff communicated to Tetra Tech the data for one seismic event recorded on June 4, 2013 at 13:14:40. The following data was provided by the Tanco staff:

- M = -2.49
- $M_0$ = 181 000 N-m
- Es/Ep = 57.41
- Rs = 4,15 m
- Ra = 6.17 m
- σs = 1.28 KPa
- σapp = 0.89 KPa
- σdyn = 5 KPa
- Coordinates:
  - 9570 N
  - 10058 E
  - 978 D

The intensity of this event is at the lower threshold of the system measuring range. Postulating that no other events were observed since then, we observe that this is a unique event. Its' spatial coordinates places it about 12 m (40 ft) above the existing back elevation within the FOG area. It is a location well within the crown pillar. Considering the low intensity we would likely attribute it to a minor movement along an existing structure rather than a fracture or crack propagation event. Two natural conclusions come out of this data; the monitoring system is functional and provides a good coverage of the target area. The pillar is still active, audio data indicates unraveling at the surface of the back while the micro-seismic system provides indication of some activity deep within the crown pillar. Considering that all mining activities have been stopped, the pillar still shows some activity. Although in limited quantity, these field measurements indicate that this crown pillar long term stability is compromised and that the failure process has a structural component.

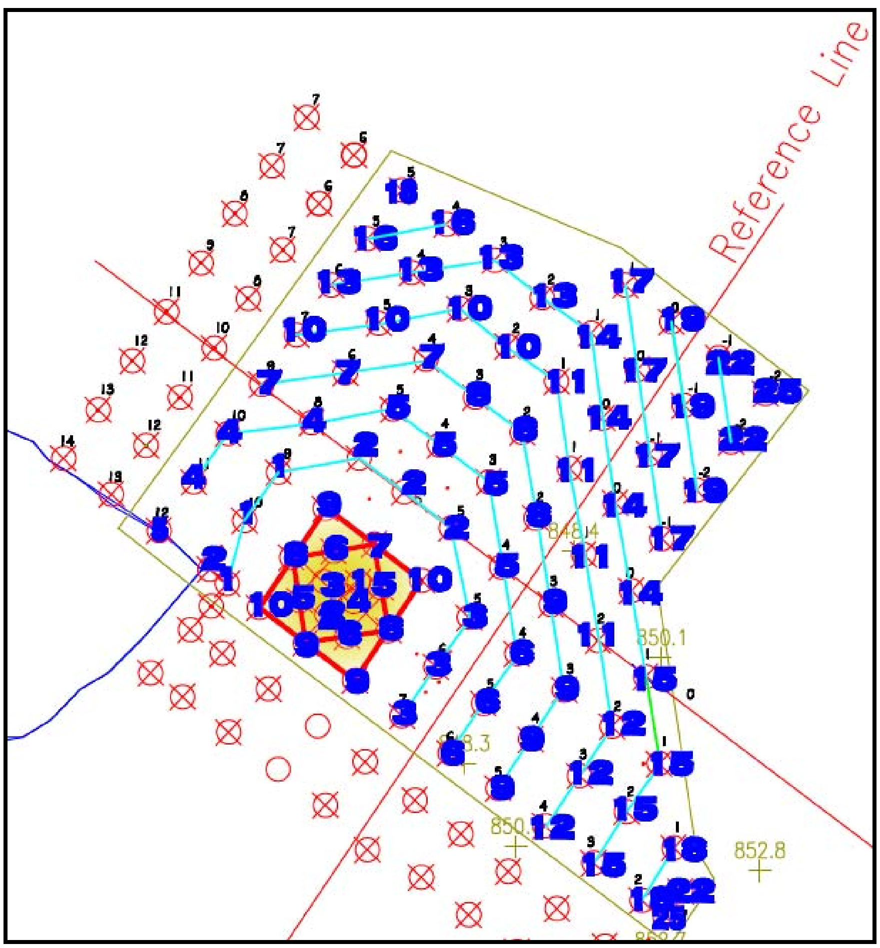
Extensometers as well as stress meters have been installed to complement the monitoring program. Although the information required to analyze this data has not been provided, initial indications from these instruments do indicate some stress redistribution occurring within the area of concern even while mining activities are stopped for the time being. Again, this information supports the conclusion that the crown pillar long term stability is compromised.

### 2.2.6 PROXIMITY OF ONGOING MINING EFFORTS

Mining activity in the vicinity of the unstable zone has an impact. Blasting places force on the rock mass which needs to be redistributed by the pillars, sills, and crowns. In mining it is common to expect the ground to absorb and redistribute impact stress. Over time, competence of pillars and sills is expected to degrade. Engineers and operators know, and expect, that repeated blast impacts will accelerate degradation.

In light of the pending concerns, the mine personnel submitted a blasting plan, 19N-1 longhole blast for our review. This plan, to be trialed in June 2013, is to develop a slot or “drop raise” over three small and successive blasts. Each slot blast will be detonated and followed up with inspection of site and instrumentation before proceeding to the next blast. Once the slot is complete, the main body of ore, 3,700 tons, will be blasted. This main blast will be relived into the slot. The main blast involves a total of 87 blastholes but the holes will be delayed so that no more than 4 blastholes will be fired simultaneously (Figure 2.7). The powder factor for this particular blast method is about 0.86 pounds of explosive per ton of ore. Tetra Tech has been informed that this approach represents a reduction in powder factor and explosive content per delay by a factor approaching 25%.

**Figure 2.7 Plan View - Delay Plan for 19N-1 Longhole Blast**



Tetra Tech believes that this is an appropriate adjustment to minimize risk of overloading the host rock with blast stress. The data collected from the monitoring equipment will significantly accelerate our learning curve for the behaviour, and timing and nature of failure, of the crown pillar.

## 2.3 PROBABILITY OF CROWN PILLAR FAILURE

In the context of this report, ultimate failure refers to the uncontrollable breakthrough to the surface. Since the location of that breakthrough would be underwater, the risk to mine and personnel is of the greatest magnitude. Our observations and the data available suggest that the roof stability in this fall of ground area is structurally controlled. In a blocky mass, such as the rock unit in the back, failure can occur where intersections of several adversely oriented discontinuities occur or where a particular suite of major joints provides a release mechanism for gravity collapse. This is the most likely mechanism involved in a portion of the crown pillar (14-P area) when we see the unraveling of the rock mass. The problem with this type of failure mechanism is that it is very difficult to predict its rate of evolution over time. Through the recently installed micro-siesmic system and the reactivation of mining practises we anticipate a learning curve of weeks or months will enable the prediction to ultimate failure.

The right hand-side of Figure 2.4 indicates that the rock conditions are markedly different. The layering observed and general conditions would indicate a buckling failure mechanism potential in that portion of the roof. As the volume of rock (left side of Figure 2.4) keeps unraveling, ultimately the exposed length of the layered rock will increase. A likely failure scenario would be the unraveling of the blocky volume of rock followed by the buckling failure of the layered gabbro. Since two very different failure mechanisms are involved, the application of empirical methods only must be treated with caution.

These empirical methods are based on a multitude of case studies and are very useful tools to assess crown pillar conditions. It must be kept in mind that these methods only provide general guidelines as discussed previously.

### 2.3.1 Some General Comments

From a general perspective, crown pillar stability could be described as a function of various parameters such as (according to Carter (2000)):

$$\text{Crown Stability} = f\left(\frac{t\sigma h\theta}{SL\gamma u}\right)$$

Where:

$t$ is the crown pillar thickness

$\sigma h$ is the in situ horizontal stress

$\theta$ is the dip of the rock foliation

$S$ is the span

$L$ is the strike length of the stope

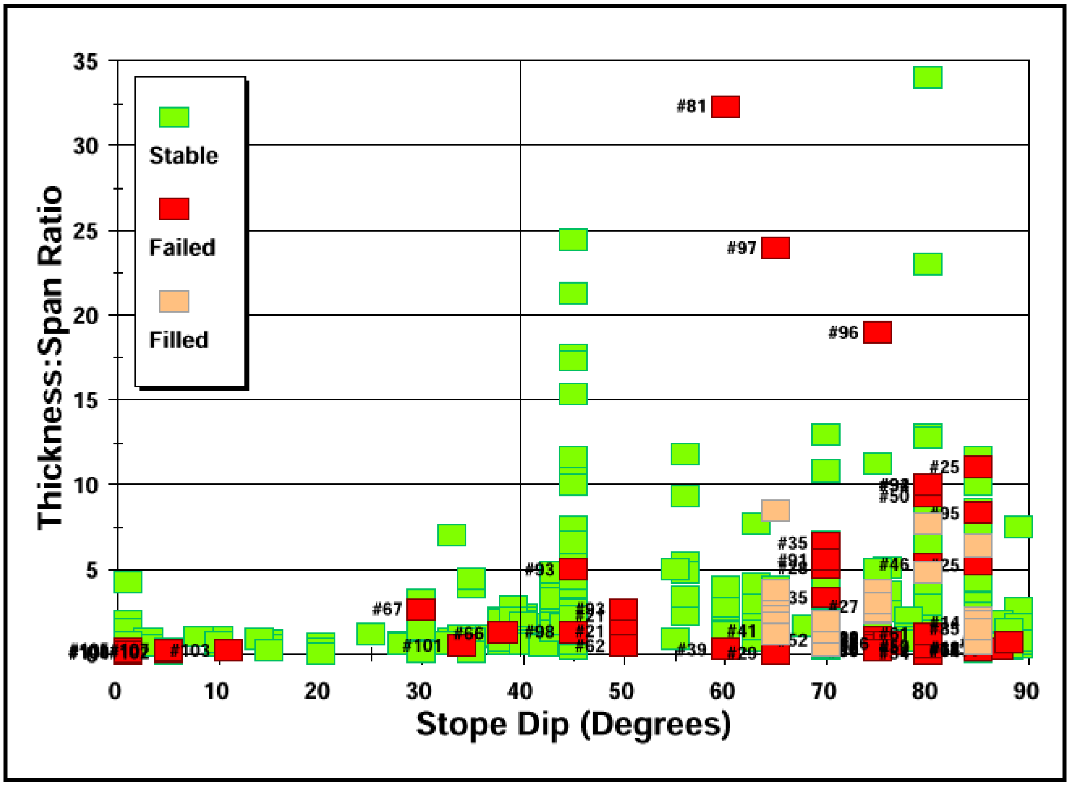
$\gamma$ is the rock density

$u$ is the ground water pressure.

From this general function we can deduce that the crown pillar thickness, in situ horizontal stress, and dip angle of the foliations are factors that will contribute to increased stability. On the other hand, the span, rock density, and ground water pressure will tend to decrease stability. We can see that except for stress, density, and water pressure, the controlling parameters are geometric.

In the case under consideration we have a high horizontal in situ stress (3.1 times the vertical stress), which is, as a general rule, a favorable condition to improve crown pillar stability. For the layered rock, the inclination would also have a favorable impact. In the case of the thickness and span the ratio is very unfavorable. Considering a thickness of 29 m (95 ft) for a span of 50 m (165 ft), we obtain a thickness to span ratio of 0.5. The general rule of thumb would call for a ratio of at least 1. Figure 2.8 presents a set of case studies compiled by various authors on crown pillar stability. As can be observed, the graphic presents no data for ratios less than 1. We could not find any compiled data for pillar geometries approaching the case at Tanco Mine. How the effective span is determined in a complex geometry as this one is not obvious. Using the maximum lengths may appear conservative. But all in situ observations indicate a condition of instability. So the conservative approach seems appropriate at this stage.

**Figure 2.8 Compilation of Crown Pillars Case Studies Illustrating Stable vs. Unstable Conditions after Carter (2002)**




## 2.3.2 STABILITY CURVE

Using a range of credible values for rock mass quality, various stability graphs were used to attempt to quantify safety level and probable stand up time. Using Bieniawski's rating system based on RMR, for the current span, stand up time predicted is less than a month.

Mathew's Potvin empirical stope design method shows the crown pillar already exceeds the maximum unsupported stable span. But this method is not strictly applicable to the current situation and it must be noted as well that there is some support (rock bolts) that have been put in.

Using the Norwegian Geotechnical Institute (NGI) system, Q rating of Barton, the current span puts the stope on the marginal stability line separating the unstable zone from the stable with support.

Using the scaled span approach proposed by Carter, the current geometry with the range of values used for the rock mass rating falls in the unstable zone.

Again, these empirical assessments must be looked at with caution. In our opinion they all point to a very low safety factor at best and some point to rapid failure. We must take into account that these assessments are based on a non-supported roof, which is not entirely the case here. In view of the evidence of ongoing progressive failure we obviously have a safety factor that is close to or less than 1.0. In mining, a safety factor of 1.2 on a crown pillar is considered to provide a stand up time that would be about two years at most.

Based on this exercise, a conservative estimate of stand-up time for the crown pillar is one to two years. We expect the probability of progressive failure within the next 12 months is 55%. Current monitoring (sonic, CMS surveys as well as initial micro-seismic data) shows that the crown pillar is in an active state, minor rock falls are registered although no obvious increasing trend has been observed.

A positive sign is that water seepage has not increased in a measurable way based on information received from the mine. This indicates that, for the time being, there is no relaxation of the rock mass structures deep in the crown pillar. The recent micro-seismic data give indications that contradict this observation. At this time the information is too limited to make definitive conclusions. As mining activities eventually resume, close monitoring and analysis of the micro-seismic data will be crucial to verify if indeed deep seated activities are triggered within the crown pillar.

We judge the probability of near term progressive failure as low but not negligible. Even with the recent micro-seismic data the empirical assessment methods are concerning since the crown pillar condition is well imbedded in the unstable zone. In order to have a means of quantifying failure probability over time, we attributed a 1% probability that progressive failure would be substantive within the next few weeks. As we gain more data from the micro-seismic system, we will develop a more refined prediction of time to failure.

Postulating a simplified linear relationship over time and assuming that mining activities resume, we expect a 25% probability of progressive failure to be approximately 6 months from the time of this report.

### 2.3.3 Paradox of Why the Crown Pillar Remains Intact – Clamping Force

The high horizontal in situ stress creates a confining force situation (beam with fixed ends). The ultimate strength of such a beam is greater than its elastic strength. But to consider the crown pillar stable, the thickness should be from 2 to 3 times the span. The likely reason why the pillar has maintained some stability for so long is due to; previously installed ground support, coupled with the fact that there is minimal water seepage. Furthermore the rock unit, although layered, exhibits a high compressive strength. When considering stability of the crown pillar in foliated/layered rock, the thicker the layers, the more stable the crown pillar will be. The concern is that as failure progresses and more relaxation happens (joints opening) hydraulic conductivity of the rock mass will increase, allowing water seepage to increase. Increased water seepage along with the clay infilling of the joints will cause the shear strength along the joints to be reduced, thus weakening the rock mass strength. An increase of seepage will be a clear indication that the crown pillar progressive failure process is accelerating.

Using the empirical design method proposed by Beer & Meek (1982), and assuming some of the parameters based on our best experience and judgment of stability for a comparable span, average thickness of the layers or strata should be of about 3.7 m (12 ft). Although we observed that the thickness in this area is about 0.5 m (1.5 ft), the 3.7 m (12 ft) long rock bolts installed in the roof create a solid layer approaching 3.7 m (12 ft). We do not know what the joint spacing is higher in the roof, so prudence must be used when interpreting this result.

The crown pillar empirical methods all considered unsupported roof conditions. In this instance, some support was installed in 2010 and does affect the progressive failure process. Due to the lack of accurate data, most methods required assuming values for some of the parameters. Although some methods predict immediate collapse, we expect that the current conditions of this pillar are at the threshold between failure and stable with support. In our opinion, the roof support installed after the first fall of ground in 2010 had a major impact in stabilizing and slowing down the failure process. The events since then demonstrate that this support was not sufficient to permanently stabilize the crown pillar.

### 2.3.4 Spall Trend and Probability of Forming a Stable Arch

The main failure mechanism anticipated is the Voussoir case, buckling. From coal mining subsidence studies, the extent of failure toward the surface for a crown pillar is 2 to 3 times the thickness of the mined seam. In the current circumstances, the height of the excavation below the unstable crown pillar is comparable to the total thickness of the remaining pillar. It is highly unlikely that there would be a sudden collapse of the crown pillar but, ultimately, progressive unravelling will occur until the bottom of the lake is reached.

In our opinion, based on the information we have available, the progressive failure of the crown pillar is unavoidable and eventual breach over a one year period of time is likely.

The combination of potentially two different mechanisms in the failure process of this pillar makes it very difficult to properly quantify evolution over time. Much more detailed information would be required and numerical analysis using discrete elements techniques and stochastic methods would be needed. Under the current circumstances, time does not allow for this level of investigation and the mine has appropriately selected mitigation measures such as soften blasting techniques and intensified monitoring, and the eventual elimination of the water above the mine via the use of dikes (see Section 3.0).

### 2.3.5 Inflow Rates Should a Breach Occur

When a breach occurs in the crown pillar it is useful to predict the severity of water inflow over different scenarios so that site management can prepare safety procedures and plans.

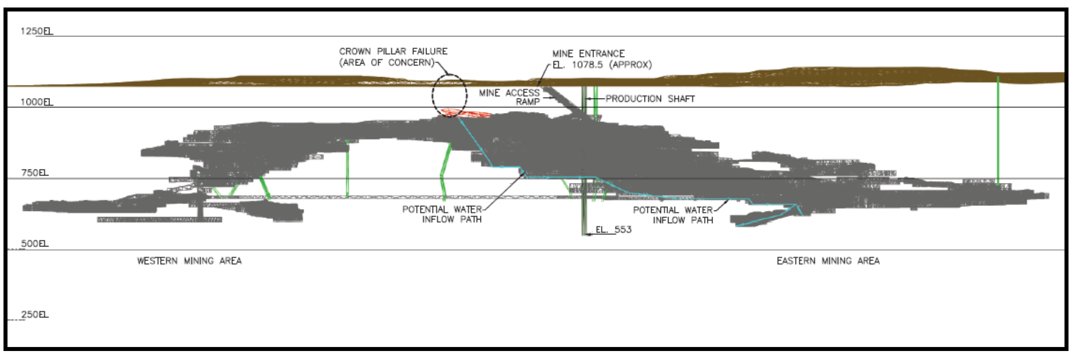
An expected progressive degradation of the crown pillar between pillars 19SGG and 25SGG on elevation 965 is already being evidenced by water inflow to the mine (see

Section 2.2.3). Cavity monitoring surveys indicate that the crown pillar continues to unravel or degrade. As the crown pillar progressively degrades the barrier between the lake bed and mining excavations will be reduced, increasing the conductivity and inflow of water from Bernic Lake. In a progressive failure, water inflow rates would be expected to be obvious to an observer over a matter of days or weeks as well as identifiable by the installed conductivity meter.

Water inflow from the degraded crown pillar will originate from the top central area of the mine and gradually flow outward towards the southeastern edge of the mine. Water will follow a path down through existing excavations to the lowest levels on the eastern side of the mine, as shown in Figure 2.9. Current mining activity is focused at the 511 elevation. This area will be one of the first areas of the mine to be affected if a water inflow were to occur.

Based on currently available information, inflow to the western side of the mine will not occur until the eastern side of the mine has been submerged to the 680 elevation. A ventilation drift at the 680 elevation, connecting the eastern and western side of the mine, will allow water to flow between both sides of the mine. At this time the mine will begin to fill evenly from the bottom up. In the event of a significant inflow miners will be able to safely egress from the lower eastern side of the mine to surface via the production shaft or the western side of the mine by vehicle through the main ramp.

**Figure 2.9 Long Section of Tanco Mine Showing Potential Path of Inflow**




Tetra Tech has no intention to create unreasonable concern in the readers mind; however for the sake of completeness we hereafter review the impact of a catastrophic failure of the crown pillar. Catastrophic failure is defined as a rapid deterioration of the structural elements of the crown pillar leading to complete collapse within a period of days.

We estimate the probability of catastrophic failure of Tanco mine's crown pillar to be less than 0.5% based on the historical reference to the number of North American hard rock mines that have suffered catastrophic failure of the crown pillar in the last 10 years.

Out of a total of over 200 operations the only known case is the Kidd Creek wedge failure. It was considered catastrophic due to the rapid and complete loss of a portion of the upper region of that mine. The event was anticipated through early warning systems and occurred over the period of a number of days.

In soft rock mines (potash, salt and coal) where host rock is typically comprised of shales, clays, and sandstones catastrophic failure of the crown pillar is slightly more common than hard rock mines. As common practice, soft rock mines employ instrumentation and appropriate monitoring/procedures to mitigate risk to workers and equipment. To the author's knowledge, no catastrophic failure of a crown pillar (hard rock or soft rock) occurred in North America resulting in injuries to personnel in the past decade.

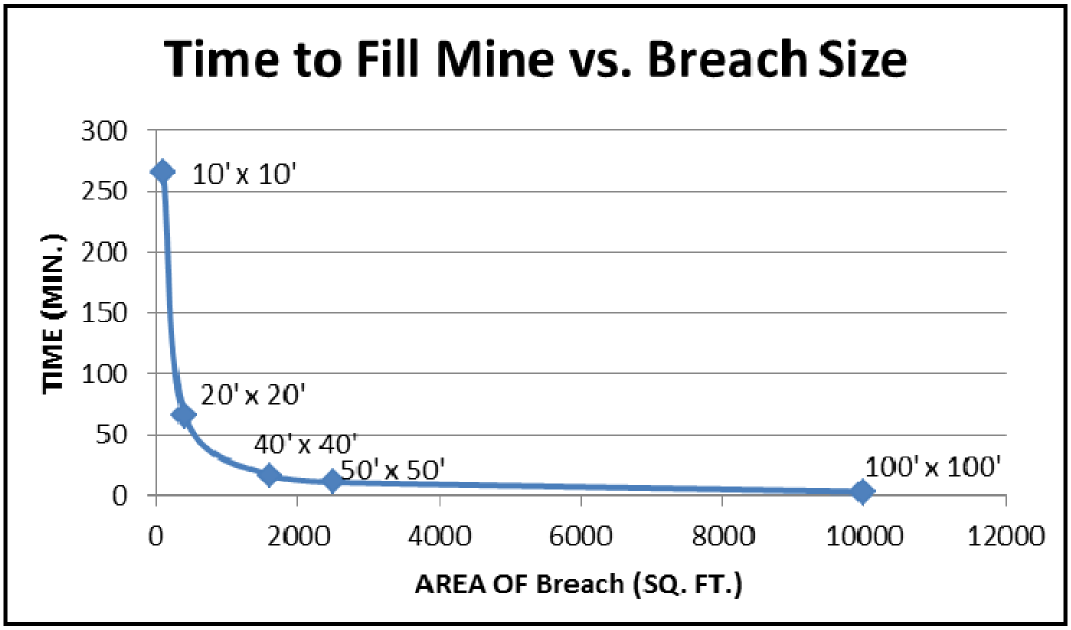
In the very unlikely event of a catastrophic failure of the crown pillar, water inflow rates could be significant. Tetra Tech has prepared a table based on assumed breach sizes (see Table 2.1). These rates have been calculated to a maximum of 254,000 $m^3$/min if a breach opens with dimensions of 15 m (50 ft) by 15 m (50 ft). The table shows the calculated time for 2,700,000 $m^3$ of water (the equivalent volume of existing underground openings) to pass through the orifice.

**Table 2.1 Inflow Rates in the Event of an Unlikely Catastrophic Failure**

| Breach Size | | Inflow Rate | | Time to Fill Underground Voids |
|---|---|---|---|---|
| ft x ft | m x m | $ft^3$/sec | $m^3$/min | min |
| 10 x 10 | 3 x 3 | 5,971 | 10,150 | 266 |
| 20 x 20 | 6 x 6 | 23,886 | 40,600 | 66 |
| 40 x 40 | 12 x 12 | 95,545 | 162,350 | 17 |
| 50 x 50 | 15 x 15 | 149,288 | 254,000 | 11 |

Tabled values are plotted in Figure 2.10 so the reader can appreciate how the inflow rate changes dramatically relative to the comparatively small change in breach size.

**Figure 2.10 Time to Fill Mine versus Breach Size**



The chart shows that if the crown pillar breached 6 m by 6 m (20 ft by 20 ft); the mine would flood within 60 minutes. Calculations show this would drop the lake level, currently averaging 7 m, to approximately 6 m within an hour. While the lake would not disappear, it could impact, any persons or infrastructure on the lake proximate to the breach.

The mine has recently completed the installation of micro-seismics, extensometers, and stress cells in the underground mine within the instability zone. These in situ instruments will provide valuable information on the ground response as the mining proceeds and a potentially better understanding of the evolution of the crown pillar over time as well as the impact of the mining activities on its progressive failure.

The mine's recent installation of a micro-seismic system provides improved monitoring of the crown pillar area in real time and offers a warning of progressive or imminent failure. Literature on the subject of micro-siesmics describes various applications of the method in underground mines. We have appended two papers describing applications of micro-seismic monitoring for prediction of fall of ground underground. These papers were selected based on the similarity between the cases presented and the room and pillar method employed at Tanco. The first paper is from Iannacchione et al. (2005). From this example of field application they show that the system would allow the anticipation of massive fall of ground with a lead time of about 12 to 24 hours. The second paper from the same lead author presents further examples of applications of the system.

With the numerous instrumentation systems now installed at Tanco, Tetra Tech submits that the learning curve to predict crown pillar behaviour/failure will be relatively short (weeks or months). It is noted that if mining activity does not continue then very little change may occur in the stress regime of the crown and it may take an extended period of time before we can learn enough to predict the ultimate failure or breach.

### 2.3.6 Evacuation Plan

Tanco has prepared a "Mine Flood Event safety Plan", issued June 2013. This plan is recognized as ever evolving however, it is summarized in its current state as follows.

- All employees are to be trained on the plan.
- Mine technical staff will, prior to shift start and more frequently if deemed appropriate, assess conditions via instrumentation, inform, and permit/deny personnel access to the mine based on their assessment.
- In the event of serious water inflows, stench gas will be used to alert personnel.
- Personnel will immediately seek refuge in refuge stations.
- Technical representatives will advise personnel if they should leave the refuge, and if so, by what route.

Tetra Tech has reviewed the Tanco mines evacuation plan for underground personal. While our review does not constitute acceptance of liability, we are generally satisfied that the plan appears appropriate and reasonable.

# 3.0 MITIGATION OPTIONS

The paramount risk to the Tanco Mine is near term failure of the crown pillar and the ensuing flood from Bernic Lake into the mine.

Given the potential for a progressive breach of the crown pillar, it would be prudent to plan for isolation of the mine from the lake. This can be achieved by constructing dikes allowing parts of the lake to be isolated and dewatered.

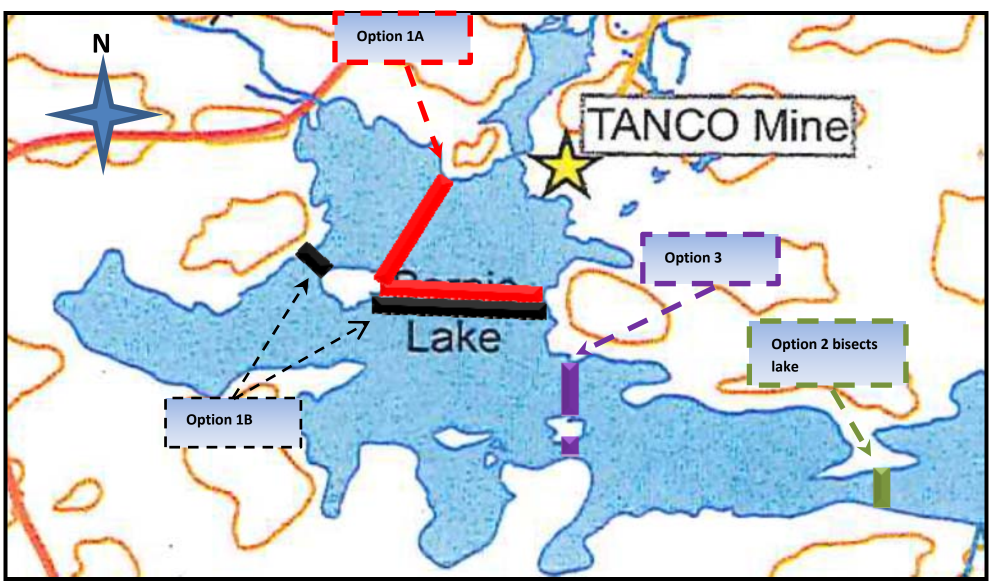
Once the water weight is removed from the crown pillar it is possible but not probable that the crown deterioration would stop. If deterioration did stop, then mining could continue underground until the minable reserves have been completely extracted. Underground mining reserves exist for another 3 to 5 years with the potential to reclassify more resource as minable within the near future.

The removal of water load above the crown, would, more likely, buckle the stressed crown causing a breach. This breach would relieve some of the horizontal stress in the remaining crown and improve probability of access for potential surface mining.

## 3.1 OPTIONS TO ISOLATE MINE FROM LAKE

Four Dike options have been identified and are presented in Figure 3.1.

**Figure 3.1 Dike Option Location Map**



- Option 1A, red line, resides just outside the footprint of the underground resource.
- Option 1B, black line, parallels the first leg of Option 1A but then continues from the island across the short channel to the west side of the lake.
- Option 2, green line, cuts across a narrow channel to the southeast and effectively bisects the lake.
- Option 3, purple line, straddles the second narrowest channel making intermediate use of a small rock islet.

The four options were analyzed for cost and schedule; the results of which are shown in Table 3.1 and Table 3.2. The dike sizes are based on an assumed average height of 12 m (40 ft) which accommodates freeboard, water to sediment, and lake bed sediment to bedrock. Cost estimates are approximate.

**Table 3.1 Capital and Schedule Estimates for Double Walled Dike Options**

| Double Walled Dike Construction | Option | | | |
|---|---|---|---|---|
| | 1A | 1B | 2 | 3 |
| **Capital Costs** | | | | |
| Average Crest (m) | 10 | 10 | 10 | 10 |
| Average Height (m) | 12 | 12 | 12 | 12 |
| Length (m) | 1,050 | 700 | 110 | 400 |
| **Cost Breakdown ($000)** | | | | |
| Key dike footprint to shoreline | 100 | 100 | 50 | 100 |
| Place Rockfill Across Inlet | 4,198 | 2,799 | 452 | 1,599 |
| Install Cement/Clay Core | 2,048 | 1,366 | 220 | 780 |
| Install Sheet-Piles | 5,195 | 3,480 | 605 | 1,979 |
| Dewater | 395 | 1,062 | 2,711 | 2,169 |
| Engineering @ 10% | 1,189 | 876 | 404 | 663 |
| Contingency @ 15% | 1,961 | 1,445 | 666 | 1,094 |
| **Total** | **15,087** | **11,128** | **5,108** | **8,384** |
| **Construction Timeline** | | | | |
| Dike Engineering, Mobilization and Construction (months) | 23.4 | 17.2 | 5.5 | 9.5 |
| Dewatering Time (months) | 1.7 | 3.3 | 6.0 | 4.6 |
| Total Schedule (months) | 25.1 | 20.5 | 11.5 | 14.1 |

**Table 3.2 Capital and Schedule Estimates for Single Walled Dike Options**

| Single Walled Dike Construction | Option | | | |
|---|---|---|---|---|
| | 1A | 1B | 2* | 3 |
| **Capital Costs** | | | | |
| Crest (m) | 10 | 10 | 10 | 10 |
| Height (m) | 12 | 12 | 12 | 12 |
| Length (m) | 1,050 | 700 | 113 | 400 |
| **Cost Breakdown ($000)** | | | | |
| Key Dike Footprint to Shoreline | 100 | 100 | 50 | 100 |
| Place Rockfill Across Inlet | 2,699 | 1,799 | 291 | 1,028 |
| Install Cement/Clay Core | 2,048 | 1,366 | 220 | 780 |
| Install Sheet-Piles | 2,623 | 1,766 | 328 | 999 |
| Dewater | 395 | 1,062 | 2,711 | 2,169 |
| Engineering @ 10% | 787 | 609 | 360 | 508 |
| Contingency @ 15% | 1,298 | 1,005 | 594 | 838 |
| **TOTAL** | **9,949** | **7,707** | **4,554** | **6,422** |
| **Construction Timeline** | | | | |
| Dike Engineering, Mobilization, and Construction (months) | 14.9 | 11.5 | 4.6 | 6.6 |
| Dewatering Time (months) | 1.7 | 3.3 | 6.0 | 4.6 |
| Total Schedule (months) | 16.7 | 14.9 | 10.6 | 11.2 |

Note: *Recommended for further study - Option 2, single walled

Dikes will be constructed using sheet piles, either single wall or double walls, supported with rocks on both sides. The crest, 10 m wide, is deemed sufficiently wide to allow traffic passage. In the case of a double wall dike, designed to reduce seepage and offset operating costs, the space between the walls will be grouted or filled with cemented fine rock. Figure 3.2 shows typical a cross-section view for a double walled sheet-pile dike and Figure 3.3 shows the conceptual construction of a single walled sheet-pile dike structure.

Figure 3.2 Section - Double Walled Sheet-Pile Dike

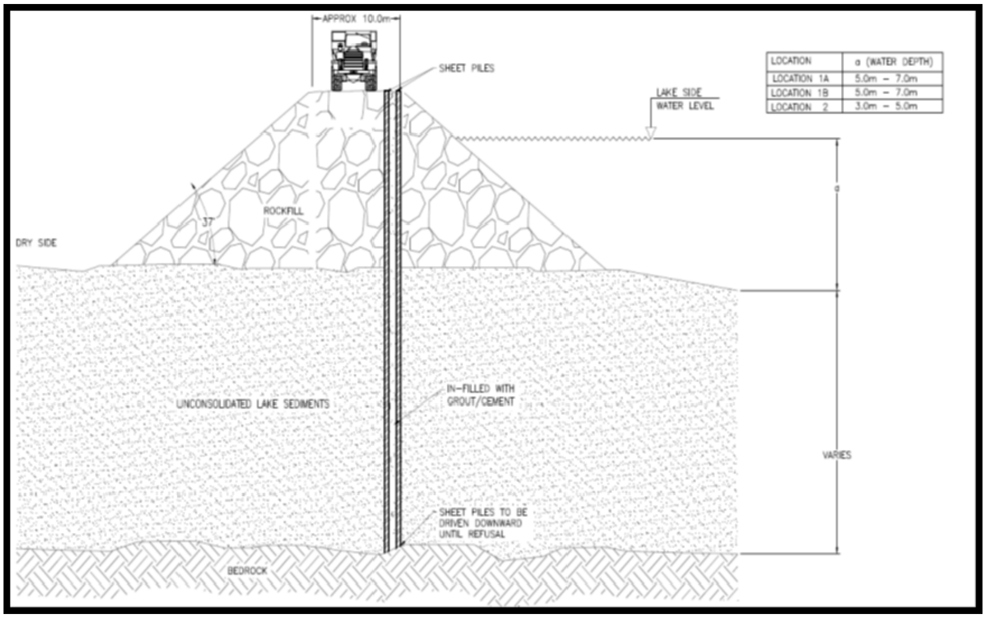
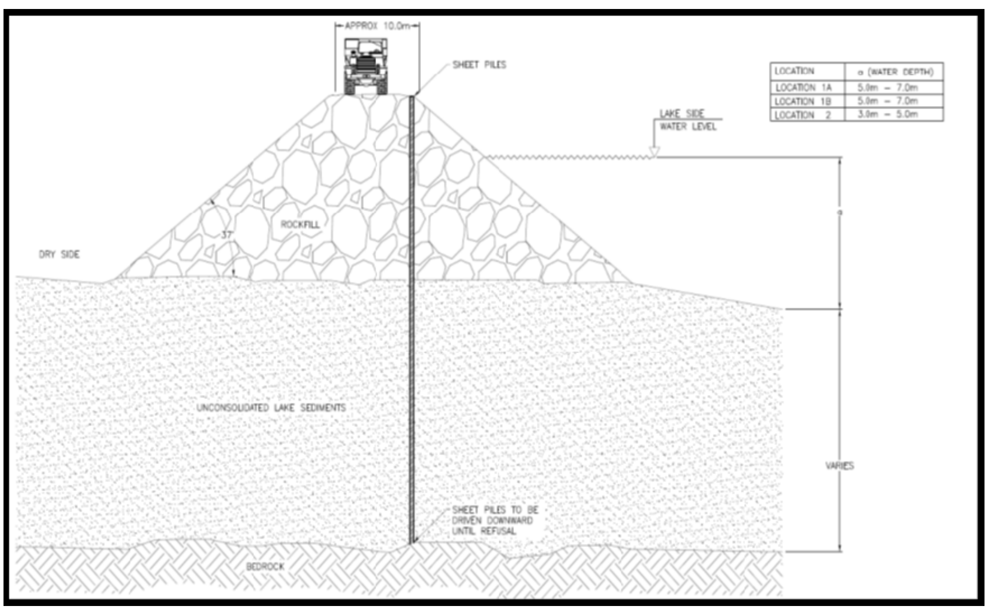



Figure 3.3 Section - Single Wall Sheet-Pile Dike




The following construction performances were used in calculation of schedules.

Table 3.3 Performance rate for Engineering, Dike Construction and Dewatering

| Task | Rate | |
|---|---|---|
| Engineering | 10% | of Capital Schedule |
| Mobilization of Contractor | 3 | weeks |
| Keying of Dike to Shore | 2 | weeks per key location |
| Place Rockfill Across Channel | 5,000 | $m^3$/day |
| Install Sheet-piles (1 m sections) | 12 | linear meters per day |
| Install Cement/Clay Core | 168 | $m^3$/day |
| Install Dewatering Pipeline | 250 | m/day |
| Demob Contractor | 1 | week |
| Dewatering | 5,000 | gpm/500 mm (20 in) pipeline |

The tasks for engineering, construction and dewatering were scheduled in sequence in order to yield a very conservative schedule. Tasks such as engineering, contractor mobilization/demobilization, and simultaneous twin-Dike construction fronts may be performed in parallel allowing significant, as much as 30%, reduction in the schedule.

### 3.1.1 Option 1A ($15 million - double walled, $10 million - single walled)

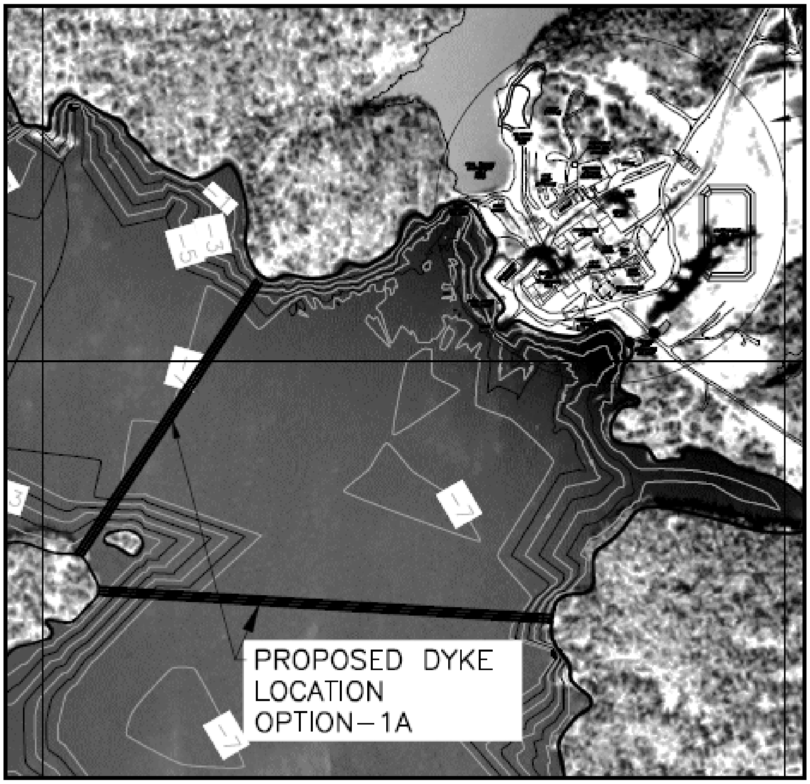
This option consists of two dikes converging on the Bernic Island, located as shown in Figure 3.4.

Construction of the 1,050 m (3,450 ft) dike can start from two points–from the shore on the north end and from the shore on the east end. Note that the schedule, 25 months double walled, 17 months single walled has been calculated as if only one side of the dike is constructed at a time.

Once the dikes are constructed, 2.9 $Mm^3$ of water will be pumped over the dike into Bernic Lake where it will gradually make its way to Bernic Creek. Water outside the dikes will continue to flow to the creek with no disruption or diversion to local hydrology. This option has the least environmental impact but is the most expensive and longest schedule time to construct.

Our largest concern with this layout is the probability of getting the dike constructed, prior to the progressive failure of the crown pillar.

**Figure 3.4 Option 1A, Dike Location Plan View**




### 3.1.2 OPTION 1B ($11 MILLION - DOUBLE WALLED, $7.7 MILLION - SINGLE WALLED)

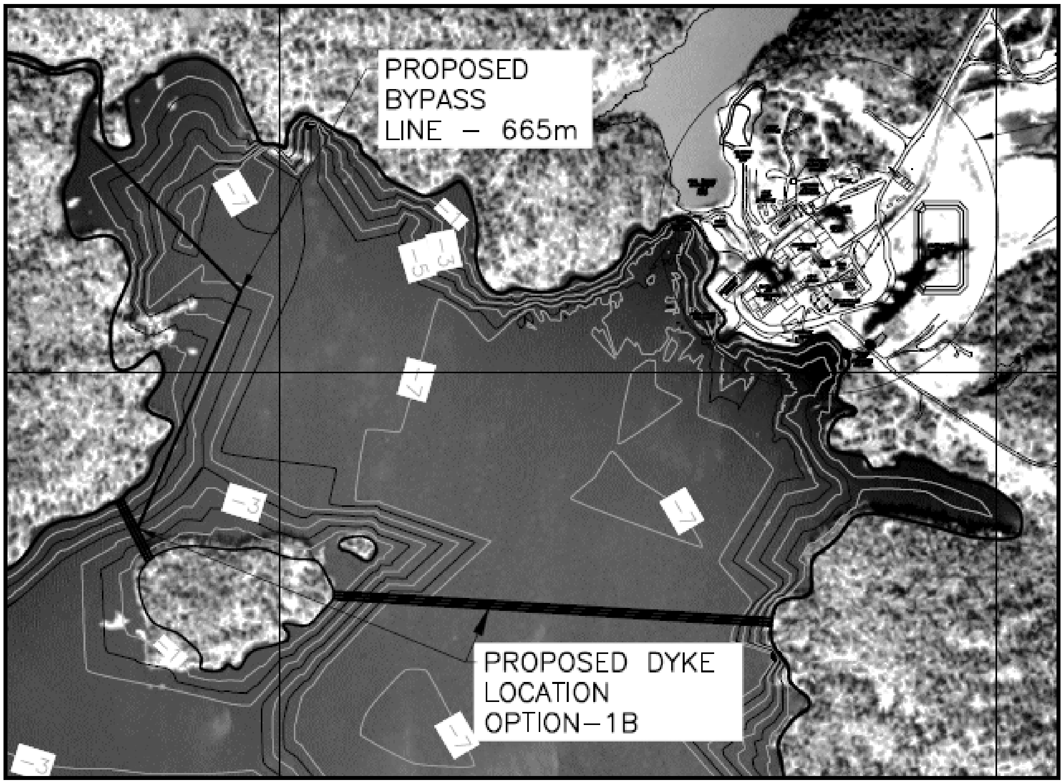
This option is similar to Option 1A except that the north portion in Option 1A is replaced with a shorter dike west of Bernic Island, see Figure 3.5. The total dike length is estimated at 700 m (2,300 ft) and also has the potential to be constructed from both shores simultaneously. Difficulties in starting the dike from two points include getting a road built around the northern part of the lake and over Bernic Creek.

As in Option 1A, water, 5.5 $Mm^3$, from within the impoundment area will be pumped out to Bernic Creek. The dikes will prevent the natural outflow of water to Bernic Creek; therefore a bypass pipeline will be required to transfer water from the lake basin to Bernic Creek.

The schedule, 21 months double walled, 15 months single walled, is an improvement of 20% over Option 1A when calculated as if only one side of the dike is constructed at a time.

The biggest concern remains accomplishing the construction effort before the progressive failure of the crown pillar occurs.

Figure 3.5 Option 1B, Dike Location Plan View




### 3.1.3 OPTION 2 ($5.1 MILLION - DOUBLE WALLED, $4.6 MILLION - SINGLE WALLED)

The authors recommend this 110 m dike option for further study due shortest schedule. Our calculations, based on a level one estimate, show the dike could be engineered and contractors could be mobilized in 1.5 months with construction then being accomplished in about 3 months based on construction from a single shore. Following the same 1.5 month engineering and contractor mobilization phase, construction from both shores could see the dike completed in less than 2 months. Dewatering of the 20 $Mm^3$ to Bernic Creek would take an additional 5 months (see Figure 3.6)

The location of the dike in this option is about 10 m east of the identified right of way of the Manitoba Hydro (MBH) power line (see Figure 3.6). This location shows the lowest cost and shortest schedule. However, a 2.5 km bypass line to Bernic Creek is required to pump water from the east basin to Bernic Creek. The cost of the 500 mm (20 in) double bypass pipelines is included at $167/m ($51.00/ft) for pipe and $328/m ($100/ft) for installation.

**Figure 3.6 Option 2, Dike Location Plan View**

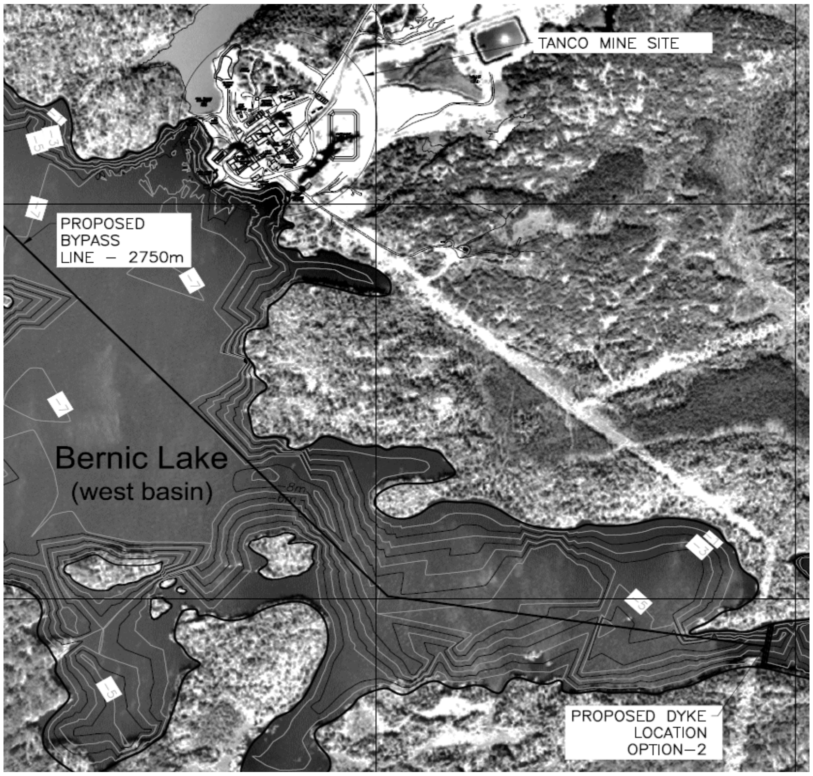
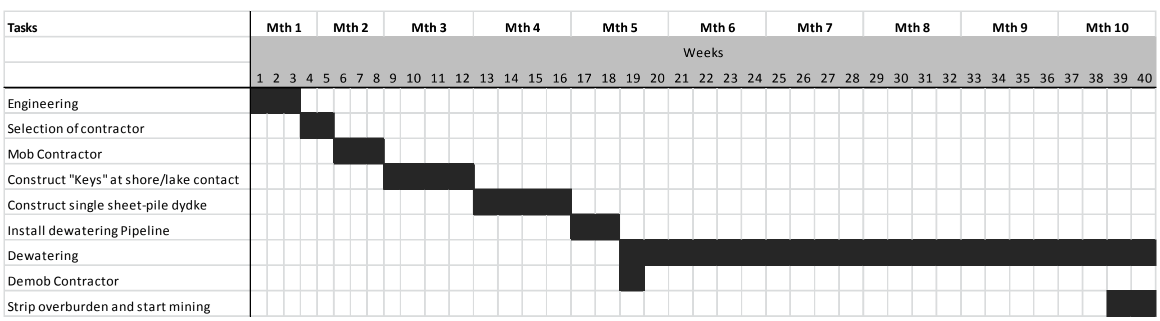



**Figure 3.7 Option 2, Schedule to Engineer, Construct and Dewater**

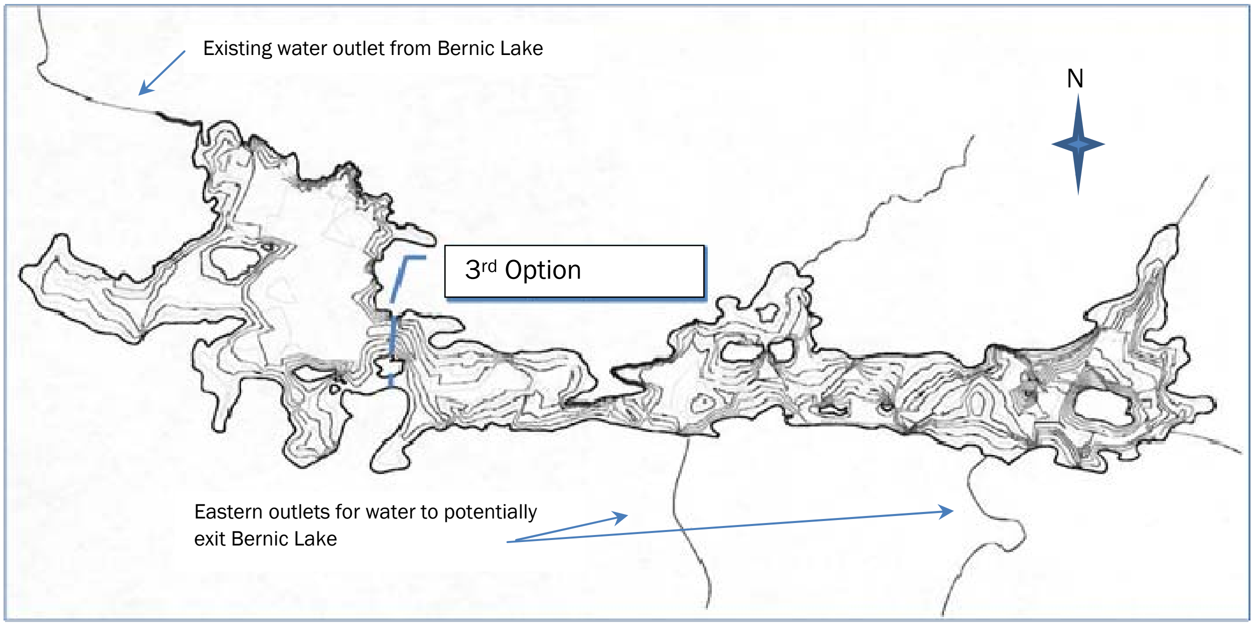



### 3.1.4 Option 3 ($5.8 million - double walled, $3.8 million - single walled)

If the Option 2 location encroaches on the MBH right of way, this location closer to the mine site presents an alternative that is faster to construct then Options 1A or 1B but not as fast as Option 2. The proposed location as shown in Figure 3.8 requires 400 m (1,310 ft) of dike spanning the second narrowest choking point in the lake. Since the dikes are bridged by a small island, a total of four keys would be required to secure the dike to bedrock.

Dewatering would be similar in methodology to Option 2 however, a 2 km bypass line, 500 m shorter than Option 2 would be required to get water to Bernic Creek.

**Figure 3.8 Option 3, Dike Location Plan View**




## 3.2 ASSESSMENT OF UNDERGROUND MINING METHOD

Tanco Mine has been in operation since 1967 and over that period has extracted approximately 2,700,000 m$^3$ (6,500,000 t) of reserves. Extraction methodology has always been mechanized post pillar without backfill. Following extraction around planned pillars and on all horizons, mine operators started mining pillars and sills as sanctioned by J.D. Smith, Mining Consultant, Kingston, Ontario (1996) in a pillar recovery engineered study.

Remaining pillars, originally designed at 15 m (50 ft) by 15 m (50 ft) by 18 m (60 ft) high, have been slimmed down to half or a quarter of their original cross-section and with the removal of horizon separating sills, some pillars now stand twice their original height, 7.5 m (25 ft) by 7.5 m (25 ft) by 36 m (120 ft) high.

Sufficient pillar and sill pillar material has been removed so that instability is now being observed in the crown pillar earlier than expected.

The first impulse is to bring in backfill and surround or confine the pillars to maintain whatever rigidity remains while going on top of that fill to install overhead ground support. This course of action was frustrated since:

- There is insufficient blasted rock on surface that could be brought underground. The central area where stability is the greatest concern would easily require 125,000 m$^3$ or assuming voids of 50%, 205 kt of rockfill.
- Over 200 days would be required to move sufficient fill underground assuming the mine could find and transport 1,000 t/d.

- The cost to move that many tonnes would be prohibitive, even at optimistic material transport costs, approximately $1.6 million at $8/t placed.
- The crown pillar is still "working" suggesting that unraveling will continue. Degradation is likely to accelerate as the crown pillar thins resulting in a breach to surface in the near future.
- Application of paste fill technology would require as much as two years to implement starting with recipe design, plant design, and distributions system design followed by the actual infrastructure construction then backfilling effort. Since paste costs between $7 and $15/t placed the effort would take between two and three years, cost over $5 million in capital and run another $1.2 million in paste fill placement costs.
- If we fail to get the fill underground and the roof supported before a breach was to occur then the investment in backfill is lost.

### 3.2.1 STABILITY OF CROWN BELOW EXISTING INFRASTRUCTURE

The FOG on March 7, 2013 removed approximately 7.5 m (25 ft) of the crown pillar in that area leaving about 41 m (135 ft) to the lake bottom. Since then, it has been reported that sporadic peeling of the crown pillar has occurred such that the estimated thickness has now been reduced to 29 m (95 ft) in one place. Obviously the stability of the overall crown pillar including that material which forms the foundations below existing infrastructure will diminish overtime.

The progressive deterioration of the crown will affect surface facilities. The facilities closest to the FOG are the propane tank farm, approximately 100 m (328 ft) projected in plain view, the Tantalum & Spodumene Mill 150 m (490 ft), and the Transformer Station also 150 m (490 ft).

Given the precariousness of the crown over time, it would be prudent to consider the eventual backfilling of voids below mine surface facilities.

# 4.0 REGULATORY APPROACH

The Tanco mine is currently operated under *Environment Act* License No. 943 issued in 1983 by the Province of Manitoba; however, construction of a dam and dewatering of Bernic Lake will constitute a change to the current operation of the mine and result in environmental effects different from those previously identified. As a result, a Notice of Alteration (NOA) must be submitted to Manitoba Conservation and Water Stewardship to permit the proposed undertaking. Dam construction and dewatering will most likely be considered a Major Alteration, thus requiring a new *Environment Act* Proposal (EAP) and Closure Plan to be filed.

Under the Canadian *Environmental Assessment Act* (2012) this undertaking is not considered a designated activity and therefore a federal environmental assessment will likely not be required; however, it is recommended by the Canadian Environmental Assessment Agency that a project description be prepared and submitted to determine the level of required federal involvement, if any. Upon acceptance of a complete project description, the Agency has 45 days to review and decide if federal involvement is required. Regardless of whether or not a federal environmental assessment is required, it is expected that federal authorizations from Fisheries and Oceans Canada (DFO; Fisheries Act s.35(2) Authorization) and Transport Canada (*Navigable Waters Protection Act* Approval) will be necessary to enable dam construction and dewatering.

The short window of opportunity available to mitigate risks associated with potential crown pillar failure does not allow for the standard regulatory review process and timeline to be followed. Therefore, Tetra Tech recommends an alternative approach and timeline that addresses the urgency of the situation. This approach was recently and successfully used to permit another Tetra Tech mining project under similar timeline constraints.

Development of the EAP will be based substantially on previous environmental studies of Bernic Lake conducted by Tetra Tech and others between 1969 and 2012. The findings of these studies will be summarized and the anticipated impacts associated with dam construction and lake dewatering will be assessed for inclusion in an initial EAP submission. The initial EAP submission will also provide a biological work plan that will detail the additional baseline environmental data collection, if necessary. It is anticipated that this initial submission will provide Manitoba Conservation with enough information to provide Conditional Approval for dam construction to be initiated.

Additional data requirements will be determined during initial meetings with the review committee and may include fish and fish habitat considerations, water/sediment quality modeling, hydrologic modeling, and community engagement. The work plan will set out the types of surveys Tanco is planning to carry out, the survey methods and timeline, and reporting schedule.

## 4.1 ENVIRONMENTAL CONSIDERATIONS

The standard permitting process for a project of this nature requires extensive upfront environmental studies to support the permit application. Proponents are expected to demonstrate an understanding of the natural environment, assess potential effects, provide mitigation measures, and provide plans for managing and monitoring potential environmental effects prior to construction and operation. The urgency of the project precludes following the standard linear permitting process as the process cannot be completed within the acceptable risk window. It is anticipated that Tanco may have to make some commitments to the regulators to conduct environmental studies and develop management plans in order to expedite the process. Described below are key items where we anticipate the regulators may require commitments.

## 4.2 FISH AND FISH HABITAT

### 4.2.1 LAKE FISH-OUT

DFO typically requires that fish-bearing water bodies be fished-out prior to dewatering and the fish made available to local communities. The purpose of the fish-out is to ensure a) the fish are utilized to the extent possible by local stakeholders (typically aboriginal communities), and b) to take advantage of an opportunity to collect extensive data on a fish community. There are published protocols to aid in the design and implementation of a fish-out program (Tyson et al. 2011). A more detailed habitat inventory of the west basin may be required prior to dewatering. The fish-out would begin as soon as the west basin has been isolated and, based on the size and productivity of the west basin, would take six to eight weeks to complete. A report containing the data and analyses would be required to complete the task.

### 4.2.2 FISH HABITAT COMPENSATION PLAN

DFO requires physical fish habitat compensation for any fish habitat lost, even temporarily, due to development. The fish habitat compensation plan details the assessment of habitat losses, outlines the proposed physical compensation, and provides an estimate of costs to implement the plan. DFO requires a letter of credit for the costs to implement the plan. It is anticipated that the west basin will be returned to fish habitat at closure while the area around the crown pillar will provide additional habitat. DFO has recently accepted plans that provide partial compensation during operations with full compensation at closure through restoration of dewatered water bodies.

## 4.3 SURFACE HYDROLOGY

The isolation of the west basin will alter the surface hydrology of Bernic Lake. A water management plans will be required.

## 4.4 LAKE DEWATERING

Lake dewatering will require the discharge of a large volume of water at a rate higher than natural flows and the water will be required to meet provincial guidelines before discharge to the environment. Total suspended solids (TSS) are expected to increase as the lake level drops below the armored shoreline and wave action re-suspends fine sediment. Options being considered for managing TSS include active water treatment methods as well as passive methods such as a staged dewatering. In a staged dewatering plan, surface ice is allowed to form before the final dewatering stage. The surface ice will prevent the formation of waves and the re-suspension of sediment.

## 4.5 WATER MANAGEMENT

The west basin will likely collect surface run-off during operation. This water will have to be periodically discharged to prevent back flooding of the mine workings. Surface run-off is likely to re-suspend and transport sediment. Surface water will be required to meet total suspended solids guidelines before discharge to the environment. Potential solutions include active and passive treatment. Passive treatment would involve holding the water through the winter, allowing the sediment to settle out, before discharge to the environment.

The isolation of the west basin will also disconnect the east basin from the Bernic Lake outflow (Bernic Creek). Preliminary estimates indicate that surface water will accumulate in the east basin and require and alternative discharge location. Possible solutions under consideration include construction of a channel connecting the east basin with Bernic Creek, periodic pumping to Bernic Creek or an adjacent watershed, and passive discharge channels to adjacent watersheds.

## 4.6 DUST/SEDIMENT STABILIZATION

Fine sediments will be exposed when the west basin is dewatered. The sediment will dry out and potentially provide a source of dust. In addition, the fine sediment is likely to be re-suspended and transported by surface water collecting in the basin. A number of control methods are being considered including the application of dust control products and promoting the growth of grasses and sedges on the exposed sediments.

## 4.7 ENVIRONMENTAL EFFECTS MONITORING

The mine is subject to the Metal Mine Effluent Regulations (MMER) under s.36 of the federal *Fisheries Act*. The MMER govern how the mine discharges effluent to the environment and requires annual environmental effects monitoring (EEM) program and reporting to Environment Canada. The existing EEM program is centred on monitoring Bernic Lake as the receiving water body for the mine effluent. As the area Bernic Lake currently receiving mine effluent will be dewatered, a new final discharge point will have to be identified and a new EEM program will need to be developed.  It is not anticipated that these changes will affect the current annual EEM costs.

# 5.0 SWOT ANALYSIS

A strengths, weaknesses, opportunities, threats (SWOT) analysis is included in Table 5.1.

**Table 5.1 SWOT Analysis**

| | Option 1A | Option 1B | Option 2 | Option 3 |
|---|---|---|---|---|
| Strength | • Close to mine, easy to monitor | • Close to mine, easy to monitor | • Shortest dike 110 m | • Relatively short dike |
| | • Does not require flow diversion | • Intermediate footprint on lake | • Capital expense approximately $5.1 million | • Relatively low cost |
| | • Minimum footprint on lake | | • Shortest schedule | |
| Weakness | • Dike length 1,050 m | • Dike length 700 m | • Maximum footprint on lake | • Large footprint on lake |
| | • Capital expense estimated at $15.1 million | • Capital expense estimated at $10.3 million | • Requires long bypass pipeline if natural discharge upstream not available | • Capital expense estimated at $8.4 million |
| | • Construction time approximately 24 months | • Construction timeline approximately 20 months | | • Requires long bypass pipeline if natural discharge upstream not available |
| | • | • Restricts exploration outside of dike perimeter | • Impacts natural drainage of lake | • Impacts natural drainage of lake |
| | | • Requires diversion of flows | | |
| Opportunities | • Plenty of rockfill available | • Plenty of rockfill available | • Easy to remediate | • Easy to remediate |
| | • | • Could install culvert to reconnect to lake discharge | • | • |
| Threats | • Environmental permitting | • Environmental permitting | • Environmental permitting | • Environmental permitting |
| | • Longest schedule has greatest impact of early crown failure | • Long schedule has great impact of early crown failure | • Breach on bypass line | • Breach on bypass line |
| | • High cost has greatest impact on cash flow | • High cost has great impact on cash flow | | |

# 6.0 RECOMMENDATIONS

Considering the gravity of the consequences resulting from the unlikely worst case scenario, the authors have used a very conservative approach in their analysis. In this instance, the worst case scenario was anticipated in deriving our recommendations.

- Meet with federal and provincial authorities to warn of the potential impact to the lake.
- Bernic Lake is completely situated on mine property. Therefore we suggest the mine place warning signs at or near any entranceways to the lake to ward off potential trespassers.
- Create a procedure to real-time monitor, micro-seismic, and preventively forewarn underground and surface staff if the crown is showing signs of progressive failure.
- The accessible parts of the mine should be scanned with a CMS unit as soon as practical. It is recommended that several units be deployed since a number of crews will accomplish this huge task in a shorter time. The results should be used to create a 3D model of the mine so that certainty exists about the relative location of underground voids from surface if open pit mining becomes an option.
- Continue to monitor the crown pillar using noise monitors, micro-seismic and CMS units so that trends can be identified. Trends may be the best indicator of when progressive failure is advanced.
- Meet with regulators and gain approval to start construction of a dike.
- Pre select a contractor that will build the dikes.
- Pre-select an appropriately disciplined engineering provider, such as Tetra Tech, to partner with for studies and design of environmental issues, dikes, open pit, or infrastructure relocation projects. A typical suite of technical staff that could compliment the mines technical resources should include the following basic skill sets:
  - civil
  - environmental
  - geotechnical and geomechanical
  - mining
  - processing and metallurgy
  - estimating and procurement
  - project management and project control.

- We recommend Option #2 – 110m Dike as the top mitigation measure and Option #3 – 400 m Dike as the alternate or backup option to mitigate the risk of flooding the mine.

# 7.0 CONCLUSIONS

A site visit was conducted in mid April 2013 by Andrew MacKenzie and Jacques Ouellet to conduct an empirical assessment of crown pillar stability. The crown pillar continues to “work” or redistribute stress within and around the FOG area as is evidenced by the recent falls of ground and the frequent spall of minor material as tracked by the noise monitor.

The crown’s thickness has decreased from 41 m (135 ft) to 29 m (95 ft) over the last few years as tracked by the mine site CMS unit. Considering the gravity of the consequences resulting from a failure, the authors have used a very conservative approach in their analysis. In this instance, the worst case scenario was anticipated in deriving our recommendations.

Following a thorough review of peer consultant’s reports as well as having plotted/estimated the crown stability index according to various empirical methods we are quite certain that the crown pillar has a 55% probability of substantive, progressive failure within the next year and a 25% chance of substantive, progressive failure within the next 6 months.

Based on the observations of the authors, the information made available, and a strict adherence to the recommendations provided herein we see no reason to preclude the continuation of the underground mining operation.

The first mitigation measures that come to mind in normal circumstances would be preventing further instability by installing support to the roof. This being a large span, 50 m (165 ft), of unsafe ground, no personnel should be sent within the exclusion zone. Moreover, the roof is out of reach and the likelihood of bringing sufficient backfill underground to later stand upon and install support would take about one year and cost several million dollars. The probability of failure before fill is installed is high so this approach has been discarded.

In order to protect the lake environment as well as workers at the mine, the risk of flooding must be eliminated as soon as practical. Even if mining activities were to be discontinued permanently at site the risk to the environment remains. Since water flooding into the mine is the greatest risk the mitigating options with the greatest chance of success involve removal of the water from the mine area.

Our report proposes eight (8) various configurations of sheet pile dikes which could be erected to hold back some portion of the lake water. We recommend “Option 2” (110 m (360 ft), single sheet pile walled dike) as the preferred choice since it has the quickest schedule (four months to engineer, construct and another five months to dewater for a total of nine months) and the lowest overall cost at $4.5 million.

Our authors have touched on the subject of diking parts of the lake with provincial and federal regulators. They allude that permitting would be granted, in short order, following proper notification and application for construction especially since the lake is at risk. We believe that demonstrating that environmental work was proceeding in earnest then construction of a dike solution could start within weeks–long before an environmental impact statement is finalized.

The report describes alternatives that could occur if the lake water was diverted from overhead of the mine. Other alternatives could be studied to provide a backup plan to recover the mineral resource should the now dry crown pillar continue to degrade and make underground work unsafe. We reason that tall slim pillars left over from mining and currently under the mine offices and plants will fail if an open pit mine starts stripping the crown pillar. Therefore we suggest a detailed study be conducted to identify the methodology to backfill below the surface infrastructure items.

# 8.0 REFERENCES

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Iannachione, A. T.; Esterhuizen, G.S.; Bajpayee, T.S.; Swanson, P. L.; Chapman, M. C. . 2005. Characteristics of Mining-Induced Seismicity Associated with Roof Falls and Roof Caving Events. . NIOSH Publications, 10p.

# APPENDIX A

ROO FALLS SEISMICITY

# Characteristics of Mining-Induced Seismicity Associated with Roof Falls and Roof Caving Events

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**ABSTRACT:** The National Institute for Occupational Safety and Health (NIOSH) evaluated microseismic activity from three field sites to compare and contrast the characteristics of microseismic emissions from very different geologic, stress, and mining environments. Recently, NIOSH has embarked on a research program to evaluate the use of microseismic monitoring information to identify roof fall failure processes and to assess its potential to warn of unstable roof conditions. Large roof instabilities, such as roof falls and certain roof caving events, have proven difficult to anticipate representing an increased risk to miners working in these inherently hazardous areas. When local failure processes are better understood, appropriate control measures can be engineered to mitigate these hazards. This study used microseismic emissions to help identify three local rock failure processes. It was also shown that analysis of microseismic emissions can aid in assessing the degree of instability associated with these local rock failure processes.

## 1. INTRODUCTION

Roof falls and roof caving events represent a serious hazard to miners working underground. The development of roof fall hazard recognition, monitoring, and control techniques is highly dependent on a sound understanding of the failure processes responsible for roof falls and roof caving events. Recent NIOSH studies in a limestone mine [1] have found that tracking the signatures emanating from strata transitioning from stable to unstable states aids in identifying roof failure mechanisms and monitoring for roof stability conditions. In the earliest stages of this transition, the rock strata fail by fracturing in both shear and tension. Generally, one or more significant large events and dozens of smaller events are observed on a daily basis during unstable roof states. As more fracturing occurs and existing fractures coalesce, the ground softens, increasing the potential for roof collapse. Slip can also occur along fracture planes or pre-existing bedding planes as strata move toward the developing roof fall cavity. Finally, the impact of the falling rocks onto the mine floor produces a wide range of seismic signatures that help characterize the rock fall mass and the condition of the surface upon which the rock has impacted.

### *1.1. Microseismic Monitoring and Analysis*

This study analyzed microseismic emissions from three mine sites: Springfield Pike Quarry, Moonee Colliery, and Willow Creek Mine. Springfield Pike and Willow Creek used a microseismic system developed by NIOSH [2] and were analyzed using techniques discussed by Iannacchione et al. [3]. The Moonee field site employed a microseismic system developed in South Africa by ISS International. Emissions from the Moonee Colliery were collected by the operating mining company, Coal Operations Australia Limited, and provided to NIOSH for this study.

In all three study sites, care was taken to determine accurate event locations. Typical rock fracture events have distinct first arrivals which allow for easy P wave identification, an essential characteristic for accurate event location. The size of rock failure events is established by calculating the moment magnitude (M). The moment magnitude scale, developed by Hanks and Kanamori [4], is consistent with the more familiar Richter magnitude scale and is currently used by seismologists as a measure of seismic source size. Moment magnitude is based on the static seismic moment, $M_o$, by the relationship:

$$M = \frac{2}{3}\log(M_o) - 6.0, \qquad (1)$$

where $M_o$ is expressed in SI units (N-m).

The static seismic moment [5] is determined from the observed Fourier displacement amplitude spectrum of body waves, the rock density at the source, the body wave velocity, the body wave radiation pattern, and the distance between the seismic source and the receiver. The seismic moment is also important because it is a measure of the dynamic inelastic deformation, or non-recoverable deformation, associated with a seismic source.

### *1.2. Rock Failure Process*

In this study, three general classes of rock failure mechanisms are identified as progressive, episodic, and continuous. The roof in room-and-pillar mines is generally stable until a localized increase in stress or a loss in strength disturbs the local equilibrium condition and initiates failure. After the onset of rock failure, additional failures are possible as the adjacent rock layers adjust to the changing conditions. In this way, a progression of rock failure is established until the surrounding rock mass reaches a new state of equilibrium or until it results in a roof fall. This process is termed progressive rock failure.

When mining creates expansive extraction zones where stresses and deformations are concentrated at panel edges, such as those found in longwall coal mining, the roof rock fails in a continuous incremental fashion. As support is removed from beneath the overlying strata, the strata break apart and eventually cave into the void made by mining. In this case, the failure process is highly dependent on the extraction effort. If the extraction effort is slow, the failure process is correspondingly slow. Rock failure progresses more rapidly as the mining rate increases. In this way, the failure process assumes a continuous incremental pattern that closely matches the daily extraction rate.

An intermediate condition exists between the progressive and continuous failure process. When mining takes place in panels of limited size or under thick competent strata, the stresses and deformation gradient at the face are reduced in comparison to continuous incremental failure. In this case, strata failure and gob formation do not immediately occur after an increment of face advance, and the expanding area of supported overlying strata becomes larger. As a result, when failure does finally occur, it does so over a wide area. This type of failure can be thought of as an end member of the continuous failure process where the increments, or episodes, are very large and not tied very closely to the short-term mining rate. With additional mining the failure progresses, much like the room-and-pillar rock failure process, until the overhanging rock mass becomes unstable and falls as a large mass. Hence, this type of rock failure is episodic in nature. These three rock failure processes provide an opportunity to examine microseismic information as it relates to the stability of underground structures.

## 2. FIELD DATA

NIOSH evaluated microseismic activity from three field sites: Springfield Pike Quarry, located in southwestern Pennsylvania; Willow Creek Coal Mine, located in eastern Utah's Wasatch Plateau; and Moonee Colliery Coal Mine, located in New South Wales 100 km north of Sydney, Australia. Each of these sites exhibits a rock failure process that resulted in different microseismic emission characteristics. Of the three sites, only the microseismic system at Moonee Colliery was designed and utilized to provide real-time roof instability warning information.

### *2.1. Springfield Pike Quarry: A Case of Progressive Failure*

The Springfield Pike Quarry is an underground room-and-pillar mining operation in the Loyalhanna Limestone. Overburden in the study area is approximately 100 m. The Loyalhanna Limestone makes up the mining horizon and approximately 2 m of the immediate roof. Above the Loyalhanna is the Mauch Chunk Formation containing alternating layers of weak claystones, shales, siltstones, and thin sandstones. The Loyalhanna Limestone has an unconfined compressive strength ranging from 130 to 200 MPa and a high horizontal stress field ranging from 14 to 55 MPa [6].

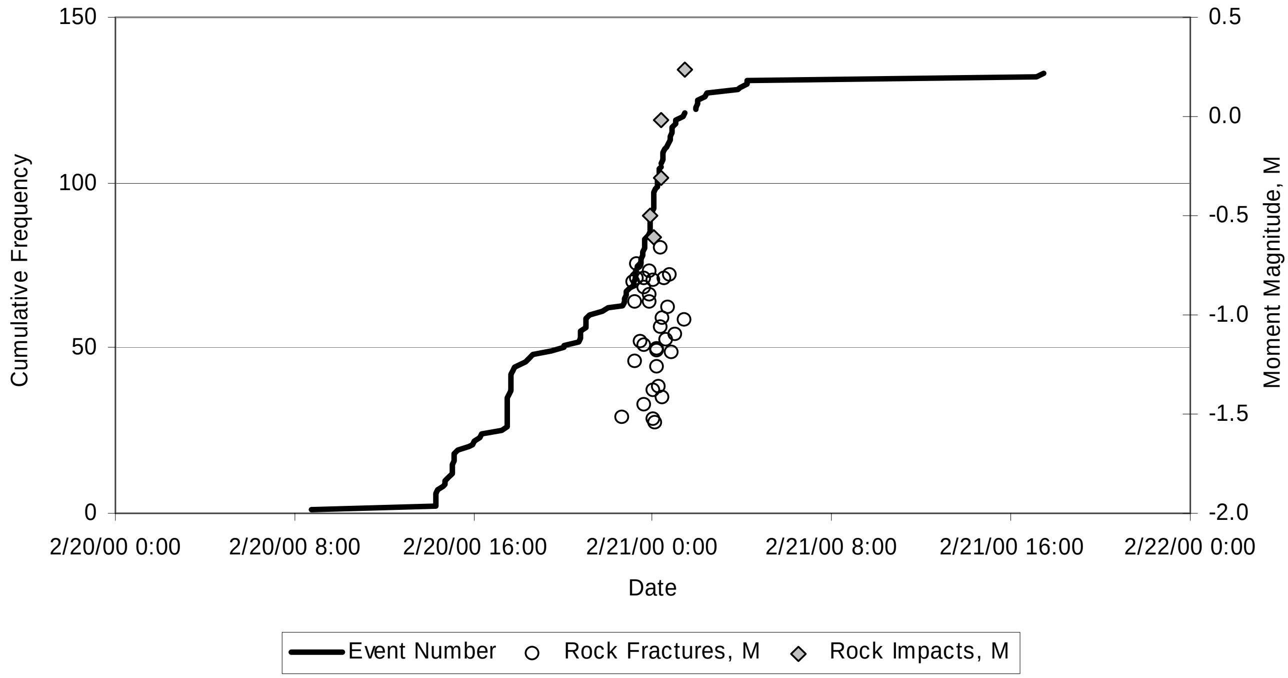


Fig. 1. A plot of the cumulative frequency of the 136 rock fracture events and five rock impact events associated with the February 21, 2000, roof fall at the Springfield Pike Quarry. Moment magnitudes were also calculated and displayed for 32 of the 136 rock fracture and for all five rock impact events.

At the site, 4.5-Hz three-component geophones are used in a single-component configuration. The geophones are mounted on the roof some 8 m above the mine floor. During the course of seismic monitoring at the study site, several thousand seismic events, including several roof falls, were recorded and located [6]. Events involve medium size rock impacts and relatively small rock fracture events. In general, the smallest rock fracture events are typically recorded only by geophones in the immediate vicinity.

On February 20-21, 2000, a series of rock impact events were associated with a major roof fall. This fall extended over 50 m in length, 13 m in width, and as much as 9 m in height. The seismic characteristics of these impacts are similar to those previously discussed by Iannacchione, et al., [3] in that they contain higher seismic moments than rock fractures, are long in duration, and emergent in form. Although it cannot be definitively determined, the authors believe the majority of the energy contained in these events comes from the rocks impacting the mine floor as opposed to the release of the rock from the roof.

Five rock impact events and 136 rock fracture events were recorded at this site (Figure 1). Almost all of the rock fracture events occurred over a 14-hour period from 2:17 p.m. February 20 until 4:15 a.m. February 21. The five rock impact events occurred over a 94-minute period and ended with a final major event at 1:24 a.m. on February 21. The locations of the geophones, rock fractures, rock impacts, and roof falls are shown in Figure 2.

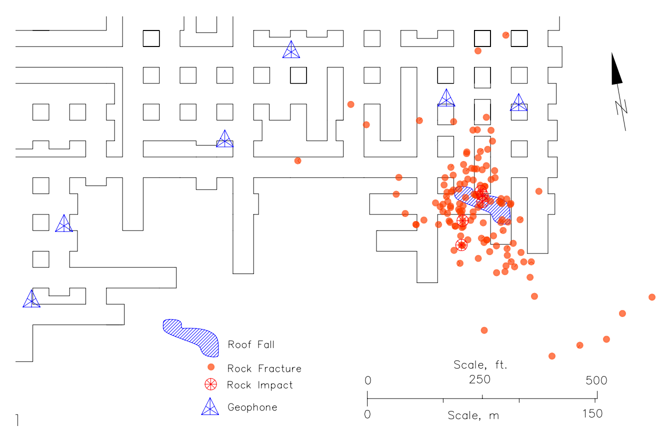


Fig. 2. Location of the 136 rock fracture events, five rock impact events, five geophones, and the February 21, 2000, roof fall at the Springfield Pike Quarry.

The moment magnitude (M) of 32 rock fracture and five rock impact events with the lowest location error measurements are shown in Figure 1. The moment magnitude of the rock fracture events ranged from -1.5 to -0.7, averaging -1.1. The moment magnitude of the rock impact events ranged from -0.6 to 0.2, averaging -0.2. The last rock impact event at 1:24 a.m. on Feb. 21 was the largest (M = 0.2) and most likely represented the biggest mass of falling rock. The magnitude of the event was probably lessened when debris buildup from the previous impacts partially absorbed the energy transfer [3].

The rock failure process in bedded formations with excessive levels of horizontal stress is initiated when the stiffest and thinnest beds in the roof strata begin to buckle from the horizontal loading [1]. When these layers buckle (Figure 3a), shear and tensile rupture between layers and low-angle shears through the intact rock layers can occur (Figure 3b). Eventually the beam begins to cantilever, initiating a tensile failure along the fixed contact area at the edge of the roof fall (Figure 3c). From this point forward, reductions in super- and sub-adjacent layers lower confinement, resulting in additional low-angle shear failures. One-by-one the individual roof beams are strained to failure, shedding their load to adjacent layers which are also strained to failure. The rock failure process is progressive in nature and reduces when the shape of the roof cavity assumes the more stable arch shape. The roof cavity shape is defined by the sub-vertical tensile failures (Figure 3d). Rock failure can be reinitiated at the axial ends of these roof fall cavities where elevated stress conditions sometimes occur.

### 2.2. *Moonee Colliery: A Case of Episodic Failure*

The Moonee Colliery is a longwall mining operation in the Great Northern Coalbed of the Newcastle Coal Measure. Overburden ranges from 90 m in the north to 170 m in the south of the mine [7]. The immediate roof comprises 1.6 m of coal and claystone. These layers are overlain by the Teralba Conglomerate with a thickness of 30 to 35 m. Unlike most longwall mines, the Teralba Conglomerate typically does not continuously cave as the longwall advances. Instead, it can hang in place until extensive unsupported spans exist. When it does cave, it can fall as a series of impacts well behind the longwall face or as one continuous mass [8]. The non-continuous caving of the roof is most likely influenced by low overburden, narrow panels, and strong abutment strength of the adjacent solid longwall panels. During mining of the first longwall panel, 41 distinct roof falls occurred with an average hanging span of 46 m. Many of these spans encompassed the entire width of the 100-m-wide panels.

Roof fall caving events began to occur at Moonee after the initial 200 m of longwall face advance of the first panel. Sometimes massive roof falls resulted in dangerous windblasts. For example, six miners were injured on Jan. 22, 1998, from a windblast event associated with the fifth longwall

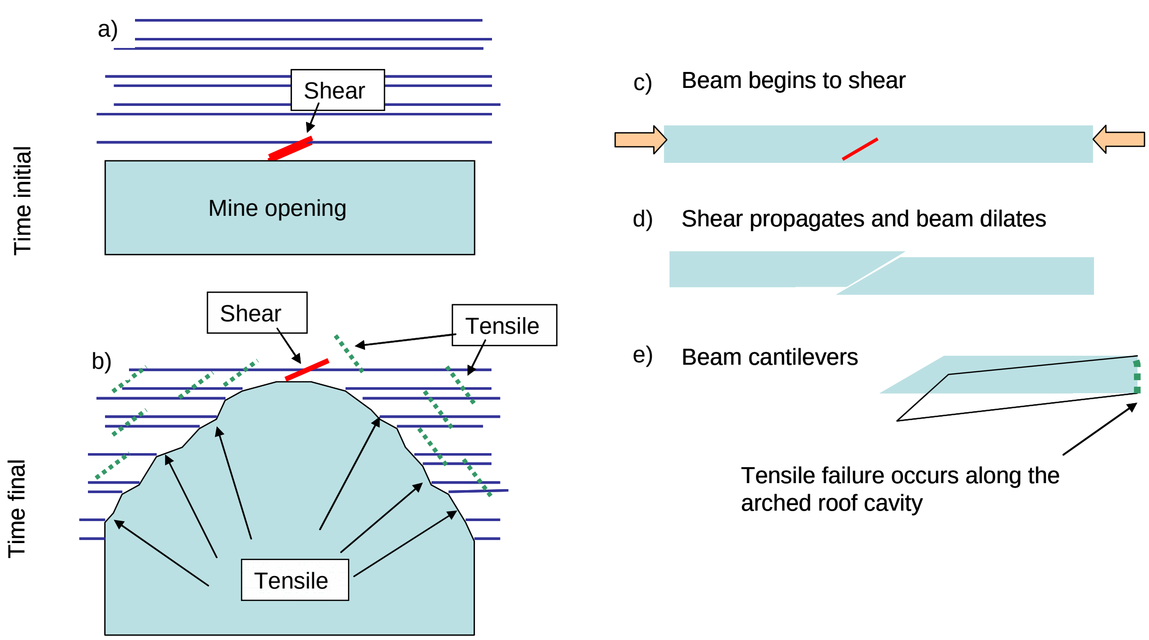


Fig. 3. Generalized sequence in which individual roof beams fail and develop into large roof falls under elevated horizontal stress conditions.

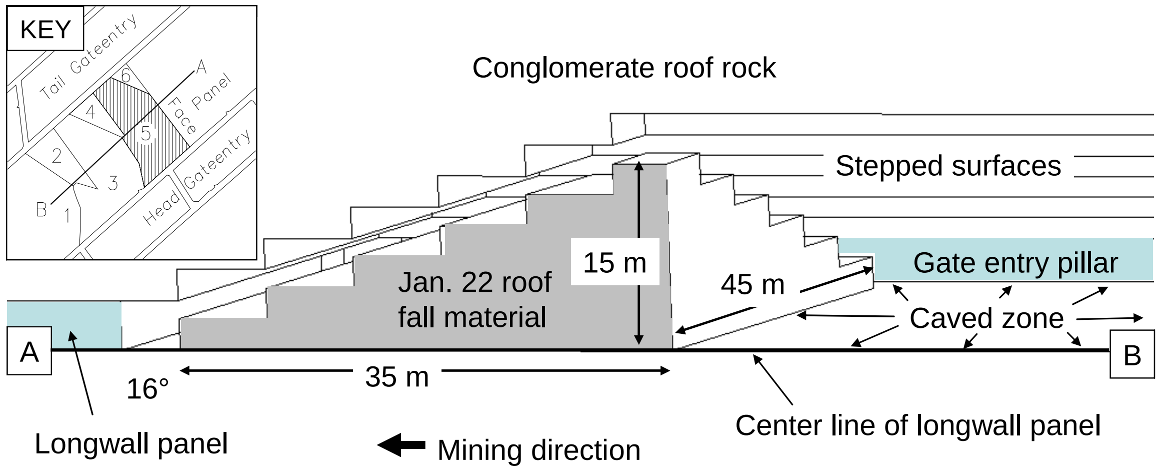


Fig. 4. Generalized sketch of the Jan. 22 roof fall (Fall No. 5), longwall panel No.1, Moonee Mine.

panel roof fall event [9]. This roof fall produced a fallen material geometry similar to half a cone with stepped surfaces (Figure 4). The top of the roof fall failure surface arched approximately 16º over the panel from the longwall face and the two gate entries, reaching a maximum thickness of 15 m in the center of the panel, 35 m from the longwall face (Figure 4). The top of the roof fall cavity was made up of both horizontal and vertical planes that formed a step-like surface. The horizontal planes were most likely associated with local bedding structures within the conglomerate, while the vertical planes were associated with the local jointing, spaced a few meters apart [10]. Above the fallen material, the conglomerate strata continued to bridge across the panel, leaving a 2- to 3-m high air gap. The back side of the fall, facing the previous caved rocks, was approximately parallel to the longwall face but arched toward the longwall face [10]. Edwards [8] and Mills and Jeffrey [10] have indicated that the general pattern described above was typical of longwall panel No.1's roof fall.

Microseismic monitoring was introduced to Moonee soon after the Jan. 22 roof fall as a way to predict the onset of caving with sufficient warning to enable miners on the face to take shelter in a safe location prior to the associated windblast [11]. This system used 14-Hz three-component geophones. Four geophones are mounted in 10-m roof boreholes around the longwall and were continuously moved to surround the longwall face. During the course of this multi-year seismic monitoring project, tens of thousands of seismic events, including numerous roof fall caving episodes, were recorded and located [7, 12]. For example, there were 118 rock fracture events (Figure 5) in the eight-day period that occurred after roof fall No. 20 and prior to roof fall No. 21. Approximately 70% of these seismic events occurred during the last three days of advance. In this case, there was only a weak correlation between advance rate and seismicity. It seems reasonable to ascribe this seismicity to the stepped fracture surface developing in the overlying conglomerate that would soon outline the fallen material for roof fall No. 21 (Figure 6).

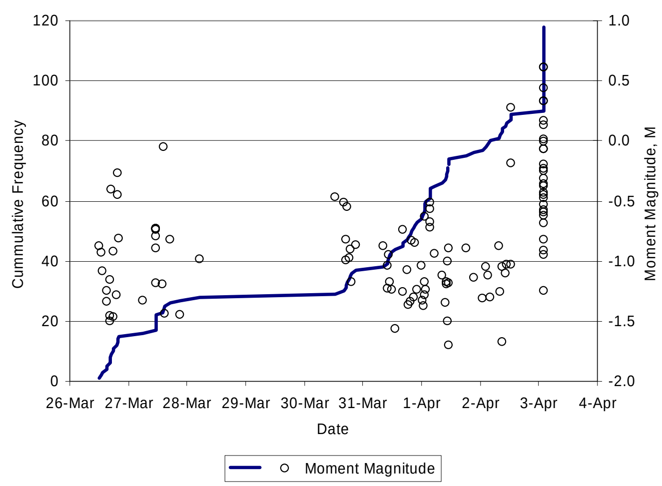


Fig. 5. A plot of the cumulative frequency of the 118 rock fracture events associated with the period between Roof Fall No. 20 and 21 at the Moonee Colliery. Moment magnitudes were also calculated and displayed for all 118 rock fracture events.

By April 3, the hanging roof extended 97 m encompassing a 9,830 $m^2$ area (Figure 6). The average rock fracture event occurred approximately 16 m above the extraction horizon. Also, a small cluster of rock fracture events occurred along a northwest-southeast trending dyke 150 m from the longwall face. These events may be associated with fracturing surrounding this stiff intact dyke. The moment magnitude of 118 rock fracture events, shown in Figure 5, ranged from -1.7 to 0.6 and averaged -0.9. Three rock fracture events with moment magnitudes between 0.3 and 0.6 occurred approximately three minutes prior to roof fall No. 21.

### 2.3. *Willow Creek Mine: A Case of Continuous Failure*

The Willow Creek Mine, abandoned since 2001, was a longwall coal mining operation in the Castlegate “D” Coalbed. Overburden in the study area ranges from 750 to 900 m. The immediate roof strata are made up of less than 3 m of thin claystones, shales, sandstones, and coalbeds. Above this is between 150 to 180 m of Blackhawk Formation composed of strong interbedded coalbeds, siltstones, and sandstones with unconfined compressive strengths ranging from 60 to 120 MPa. Capping the Blackhawk is 120 to 180 m of massive cliff forming Castlegate sandstone.

The Willow Creek longwall panels are over a thousand meters long and range in width from 162 to 244 m. The immediate roof strata cave tightly behind the longwall shields as the longwall advances. The continuous caving of the immediate roof is undoubtedly influenced by significant overburden, the wide panels, and the marginal abutment strength of the adjacent longwall gate entries.

A three-dimensional microseismic array used 14 horizontally oriented and 9 vertically oriented surface geophones (4.5 Hz). The underground geophones were mounted directly on the roof and the surface geophones were placed in shallow holes. During the course of seismic monitoring, five thousand high-quality seismic events were recorded and located [13]. The seismicity associated with a typical day of longwall advance is shown in Figure 7. During the early hours of the morning, when maintenance activities idle the longwall, seismicity is very low. Seismicity increases considerably a short time after mining commences.

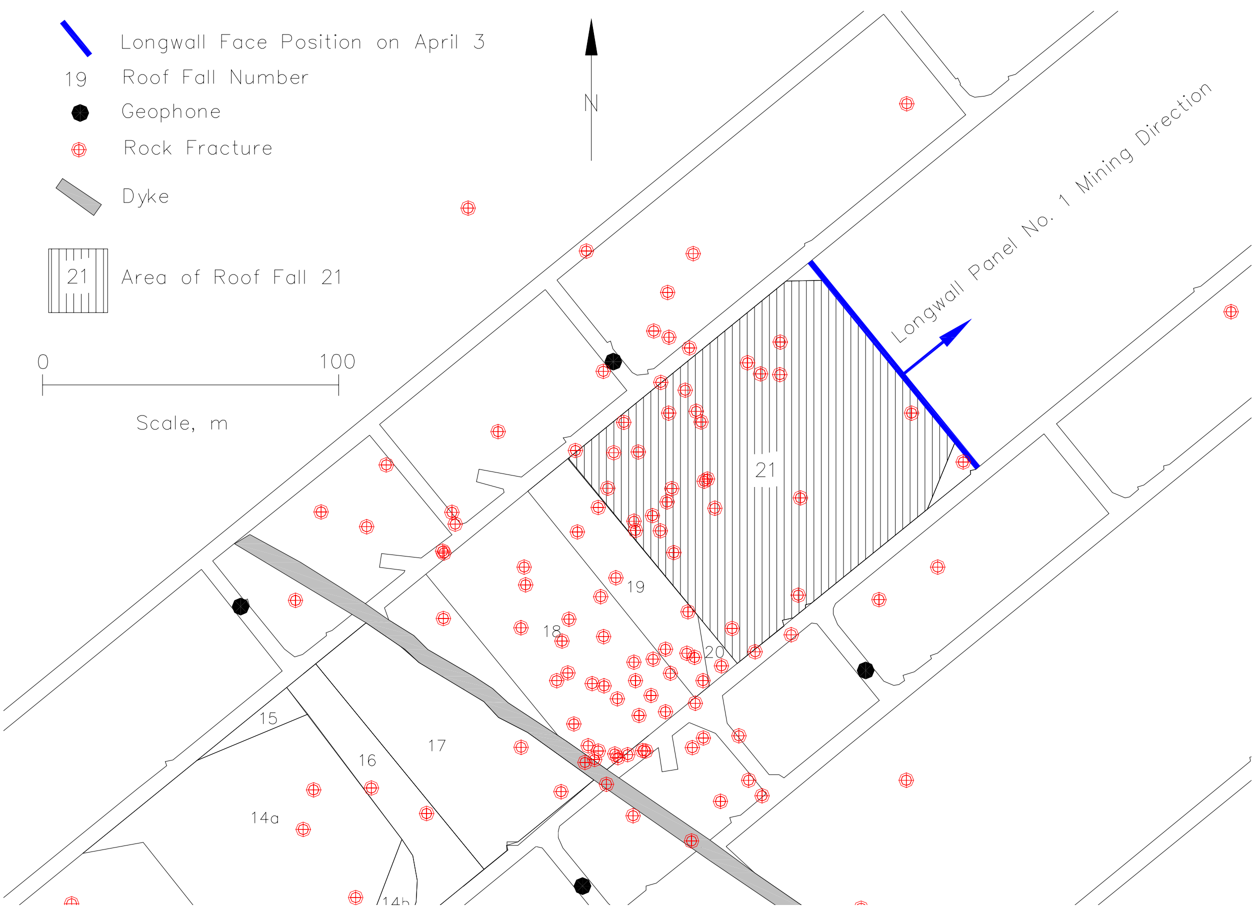


Fig. 6. Location of the 118 rock fracture events, geophones, roof falls, longwall face, and dyke at the Moonee Colliery.

During this period, events occur at a somewhat constant rate until production ceases and then relative quiet returns to the longwall panel. Correlations between advance rate and seismicity rate were documented by Heasley et al. [13] and Westman et al. [14].

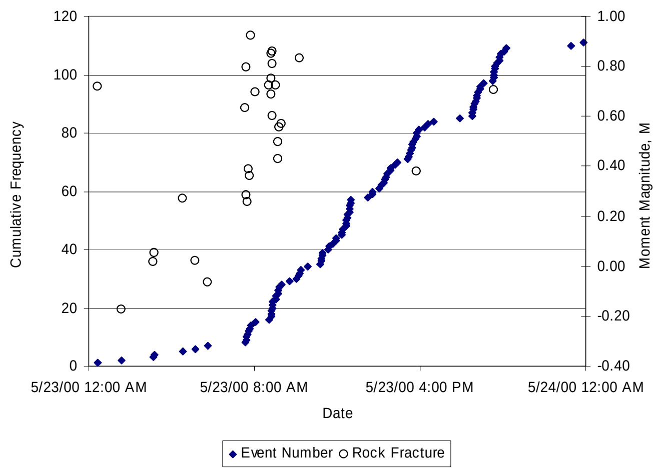


Fig. 7. A plot of the cumulative frequency of the 111 rock fracture events associated with longwall mining on May 23, 2000, at the Willow Creek Mine. Moment magnitudes were also calculated and displayed for 30 of the 111 rock fracture events.

The location of 111 rock fracturing events from May 23, 2000, cluster along the longwall face area (Figure 8). There is a noticeable concentration near the headgate entries. Additionally, the events occur both above and below the mining horizon. In examining the data for the entire longwall panel, Heasley et al. [13] found that the events generally occurred in advance of the longwall face, distributed both above and below the mining horizon.

A majority of the events are associated with the forward abutment stress that fractures the rock in the longwall face area. The forward abutment stress acts along the longwall face area, producing fractures that eventually outline blocks of various size and shapes. These blocks are held together in this area because the longwall face and shields act to supply some level of confinement. As the longwall face advances and the confinement is removed, the blocks begin to separate and collapse forming the broken rock mass known as the longwall gob.

A noteworthy observation is that microseismic emissions are obvious in the front abutment stress area but not so apparent in the longwall gob zone. This is most likely an issue of event detection sensitivity. A minimum of eight stations with high-quality arrivals were required to satisfy the criterion for locating events plotted in Figure 8. Average source to receiver distance was 1.1 km. Thus, smaller events which did not satisfy this condition were not detected. Numerical modeling studies by Gale et al. [14] support the notion that events representing incremental fracture in the frontal abutment zone should be larger than events in the gob zone. In the two-dimensional models, fracturing behind the longwall shield was generally not associated with microseismic activity because of low stress drop conditions, a corresponding lack of shear fracturing, and poor transmission of elastic energy through the broken rock mass.

The moment magnitudes of 30 selected rock fracture events are also shown in Figure 7. This subset of the May 23 events was selected because of their low location value residuals. The moment magnitudes of the rock fracture events ranged from -0.2 to 0.9 with an average of 0.5.

## 3. MICROSEISMIC EMISSIONS AND STABILITY MONITORING

The three case studies demonstrated that the local rock failure process, active at each site, could be determined from a combination of observational information and microseismic monitoring. The microseismic information was able to detect and characterize the rock deformations in a way that was otherwise not possible. In each case a distinct pattern of seismicity occurred.

### *3.1. Stability Assessment at Springfield Pike*

At the Springfield Pike Quarry, a period of quiet was interrupted by the onset of microseismic activity lasting approximately 14 hours and culminating in a series of rock impacts followed by the return to quiet. Approximately 12.5 hours of microseismic activity preceded the rock impact events that occurred in the last 94 minutes of this time period. In this case, it appears that the microseismic information could be used to provide warning of impending major roof instabilities.

### *3.2. Stability Assessment at Moonee*

The episodic rock failure process at Moonee Colliery was in many ways similar to the Springfield Pike Quarry with the exception that it was rarely very quiet in the longwall panel area. The almost continuous advance of the longwall face

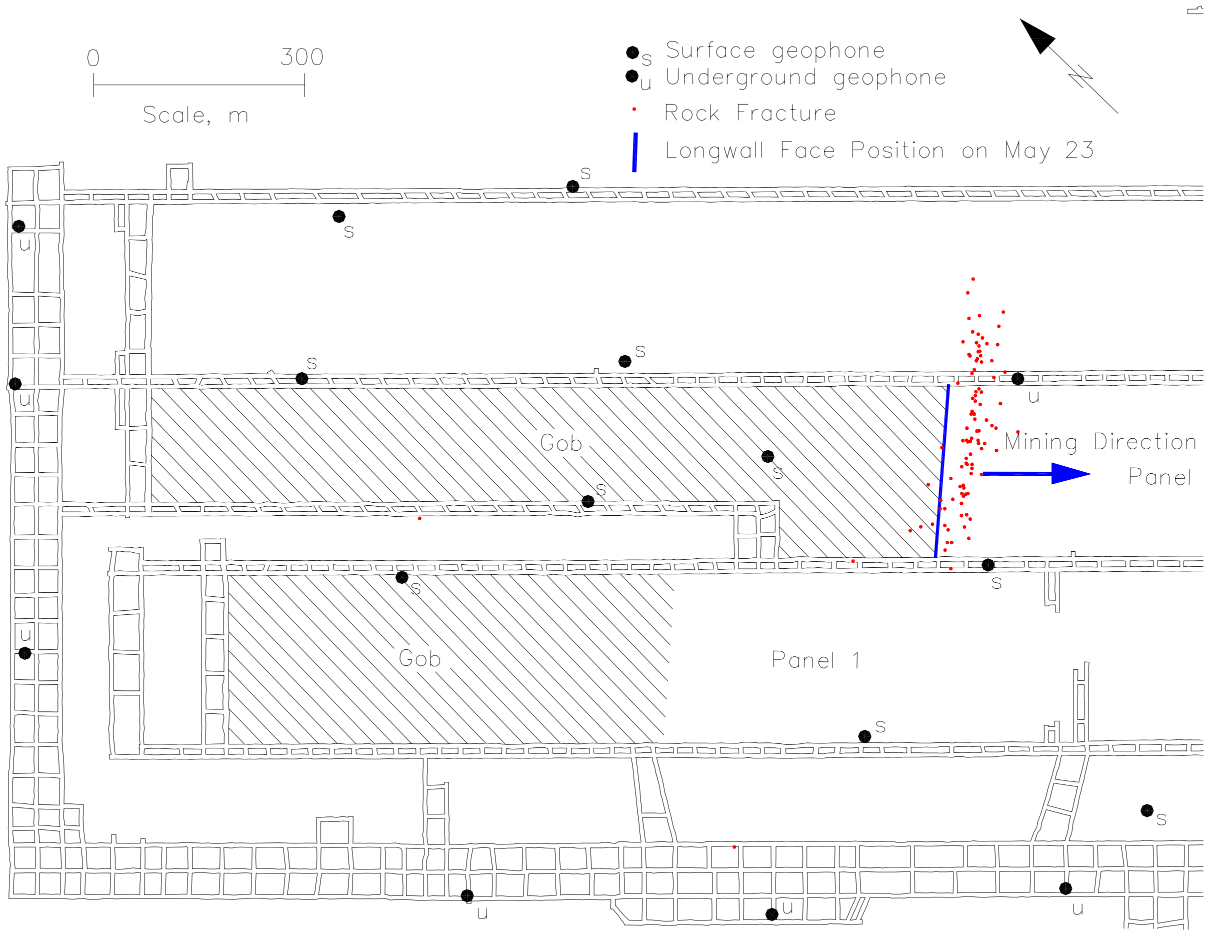


Figure 8. - Location of 111 rock fracture events, geophones, longwall caving zone (gob), and longwall face at the Willow Creek Mine.

acted to destabilize the overlying roof strata above the longwall panel. The majority of the microseismic activity did not cluster around the face. Instead, the microseismic activity apparently represents the initiation and development of the stepped failure surface and accommodation of stress adjustment over a wide area. This final activity just prior to the roof collapse represented a dramatic increase in the occurrence, rate, and, to a lesser degree, magnitude of microseismic events (Figure 5). In fact, Moonee Colliery developed a system of alarms that warned of impending roof falls from observed trends in microseismic activity. These alarms were:

1. Trend Alarms – based on interpreted trends of microseismic activity. Alarms were typically long-term warnings, usually a few hours, that provided sufficient time for the longwall crew to evacuate the face.
2. Frequency or Magnitude Alarms – based on a sudden flurry of microseismic activity. Alarms were typically short-term warnings, usually a few minutes, that provided sufficient time for the longwall crew to move to the safest area of the face.
3. Auto Alarms – based on the capture of six events or greater in a 10-second period. In this case, an alarm was automatically signaled.

Hayes [7] reports that the microseismic system was able to give sufficient warning of impending roof falls that could cause windblast in approximately 90% of the cases.

### *3.3. Stability Assessment at Willow Creek*

The continuous failure process at the Willow Creek Mine is very different from the progressive and episodic processes at Springfield Pike and Moonee. Microseismic activity was associated with the constant caving of the longwall panel. This constant activity was a sign of “normal” mining conditions. While the microseismic information was never used to issue warning of specific hazardous ground conditions, deviations from a continuous failure process may represent the onset of abnormal caving. Recognition of this deviation from normal provides the basis of a hazard warning methodology. Abnormal caving behavior is often associated with coal bumps, weighting of the longwall shields, large roof collapses in the gob, and, in some cases, damaging windblasts.

## 4. SUMMARY AND CONCLUSIONS

This paper reviews the behavior of two roof fall events and one case of continuous roof caving. Significant differences are observed in the collective seismic character from each site. These differences are related to the interaction of local geologic, mining, and stress conditions. It would be very difficult to develop an adequate understanding of the important failure processes operating at each of these sites without the microseismic information. Three rock failure processes were identified as progressive, episodic, and continuous.

Microseismic information collected from three field sites consisted of:

- At Springfield Pike, 136 rock fracture events occurred over a 14-hour period. The moment magnitudes of these rock fracture events ranged in size from -1.5 to -0.7. During the final 94 minutes of this failure event, five significant rock impact events occurred. The moment magnitudes of these rock impact events ranged in size from -0.6 to 0.2.
- At Moonee, 118 rock fracture events occurred over an eight-day period between Roof Fall No. 20 and No. 21. The moment magnitudes of these rock fracture events ranged in size from -1.7 to 0.6.
- At Willow Creek, 111 rock fracture events occurred during a normal production day. The moment magnitudes of 30 selected rock fracture events ranged in size from -0.2 to 0.9.

In each of the three case studies, the character of the microseismic information helped to identify the rock failure process:

- Progressive – Periods of quiet are interrupted by the onset and progression of microseismic activity which lasted in this case for approximately 14 hours and culminated in a series of rock impacts followed by the return to quiet.
- Episodic – The majority of the microseismic activity represents the initiation and development of the stepped failure surface that outlines the eventual roof fall material. The final surge in activity is associated with the completion of this surface and the collapse of the roof.
- Continuous – Microseismic activity in the forward abutment stress zone was associated with the constant caving of the longwall panel.

It was also demonstrated that the microseismic information provided a useful assessment of the stability of the roof rock. In the case of the progressive rock failure process, the onset of microseismic activity signaled the beginning of unstable conditions. It was also demonstrated that while activity continued, rock impacts were possible. In the case of the episodic rock failure process, trends in the microseismic activity were used to warn of roof falls with a high degree of success. Finally in the case of the continuous rock failure process, the constant uniform activity in the forward abutment stress area was a sign of "normal" conditions. Deviations from normal strata response can provide useful stability information.

Microseismic activity, occurring during and before roof fall and roof caving processes, can be measured and analyzed to characterize local failure processes. Enormous potential exists for application of this technology to assess the stability conditions of underground structures, so that safer mine layouts, monitoring systems, and support systems can be engineered.

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# APPENDIX B

## Characteristics of Mining Induced Seismicity Associated with Roof Falls and Roof Caving Events

# OPEN PIT MINING THROUGH UNDERGROUND WORKINGS

## GUIDELINE

Department of Industry and Resources

MOSHAB

Mines Occupational Safety and Health Advisory Board

## FOREWORD

This Mines Occupational Safety and Health Advisory Board (MOSHAB) Guideline offers advice on the issues that should be addressed when open pit mines are excavated through abandoned underground workings, or in close proximity to current underground workings.

Comments on, and suggestions for, improvements to the Guideline are encouraged. This Guideline will be revised as appropriate.

Comments should be sent to:

State Mining Engineer
Safety Health and Environment Division
Department of Industry and Resources
100 Plain Street
EAST PERTH WA 6004

**ISBN 0 7309 7814 1**



**Email: shed@doir.wa.gov.au**

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## 1.0 INTRODUCTION

A number of open pit mines in Western Australia (WA) are mining orebodies that have previously been mined by underground methods. There are hazards with high risk potential which develop where open pit mines approach and then progressively mine through underground workings.

These hazards in the open pit include:

- sudden and unexpected collapse of the open pit floor and/or open pit walls;
- the loss of people and/or equipment into unfilled or partially filled underground workings;
- loss of explosives from charged blast holes that have broken through into the underground workings;
- overcharging of blastholes where explosives have filled cavities connected to the blasthole;
- risk of ejecta (flyrock, etc) from cavities close to the pit floor and adjacent blast holes, particularly when explosives have entered the cavity from the blasthole during charging and the loss is not detected.

In general, the above hazards are significantly increased when the underground workings have not been backfilled with waste rock, sand fill, etc. As these hazards are not generally evident during normal open pit mining operations it is necessary to take additional measures to better define their nature and extent. Some of these measures are discussed in **section 5**. Once the relevant hazards have been adequately defined the mine operator should put in place a range of controls to mine safely through the underground workings. A number of these controls are discussed in **section 6**.

In addition to the above hazards, when open pit mines approach currently operating underground mines, the potential hazards may include:

- flooding of the underground workings;
- instability of the open pit walls and surrounding surface areas;
- adverse effects on the underground mine ventilation system.

This guideline primarily addresses hazards associated with open pit mining through abandoned underground mine workings. Some of the additional hazards associated with open pit mining through currently operating underground mine workings are summarized in **Appendix A**.

## 2.0 LEGISLATIVE REQUIREMENTS (WA)

The **Mines Safety and Inspection Regulations 1995** includes a provision (**Regulation 13.8**) relating to surface mining operations where mining is being conducted through or in proximity to underground mine workings.

| |
|---|
| Geotechnical considerations<br><br>13.8. (3) Each responsible person at a mine must ensure that appropriate precautions are taken and written safe working procedures are followed if open pits are excavated through abandoned underground workings, or in close proximity to current underground workings.<br><br>Penalty: See regulation 17.1. |

## 3.0 HAZARD IDENTIFICATION

Knowledge of the previous mining history of the area to be mined will be of primary importance in determining the likelihood of abandoned underground workings being present below the open pit. A thorough review of previous mine plans is essential prior to any open pit development. The validity of old underground mine plans should be checked diligently, particularly if they are abstracted or copied from originals.

A review of underground workings should be part of the design and planning of the open pit to ensure, as far as reasonably practicable, that:

- all known underground workings are marked clearly on all working mine plans and the plans rechecked;
- there is a recognition that the rock mass surrounding the underground workings may be highly variable in strength and potentially unstable;
- a three dimensional model of underground workings is developed and used in all mine design, planning and scheduling.

It is essential that all plans are updated following all phases of exploration to ensure that the revised outlines of the actual extent and shape of underground workings are recorded.

A further aspect requires a cautious approach. Tributors may have carried out further work in old gold mines, which may not have been recorded on the closure plans lodged by the mine before tribute mining took place.

Where it is likely that underground workings could be of large dimensions and extended in length and depth, or where no previous plans are available, it may be necessary to carry out specific investigations to confirm the location of the workings. Some of the methods that may be used for this purpose are briefly discussed in **section 5**.

## 4.0 MAKING THE HAZARD VISIBLE

All areas of a working bench or flitch that are likely to be underlain by underground workings must be clearly marked and access to this area must be controlled by a specific set of procedures. These procedures should specify the personnel responsible for monitoring and marking out the hazardous areas. Every bench or flitch should be clearly marked with the projected excavation outline as mining progresses downward through the underground workings.

The marking of areas potentially underlain by underground workings must involve a clear method of identifying the potential hazard. If coloured flagging tape is used, a specific colour – preferably visible in both day and night conditions - should be used solely for this purpose. Steps should be taken to ensure that hazardous areas are adequately marked at all times. Damaged or displaced flagging tape should be immediately replaced. All employees must be informed as to the purpose of the marking or flagging tape.

Care should be exercised in the location of the marked areas. Allowance should be made for the uncertainty in the precise position of the underground workings and any potentially unstable ground surrounding the underground workings. In short, an extra margin of safety should be allowed in the separation of permissible work areas from suspect zones.

# 5.0 RISK ANALYSIS

## 5.1 Determining the extent of underground workings

A number of detection methods are available which may be used to confirm the lateral extent and shape of underground workings prior to mining, including:

- probe drilling[1];
- geophysical techniques – including seismic, resistivity, conductivity, and gravity methods;
- ground probing radar;
- laser based electronic distance measurement (EDM) surveying methods;
- closed-circuit TV cameras lowered through probe holes;
- where practicable, actual physical inspection and survey.

Probe drilling is the most widely applied technique to delineate the detailed geometry of underground workings in WA. Remote sensing techniques (ie geophysical techniques and ground probing radar) have been used with varying degrees of success, with individual techniques having limitations depending on the nature of the local geological conditions. Ground probing radar has been used with limited success to detect voids below open pits in WA. Remote sensing techniques are not universally applicable and even when successful, require some level of confirmatory drilling.

## 5.2 Probe drilling procedures

Site specific written probe drilling procedures are essential. These procedures should specify the following:

- the type of drilling rig to be used and the provision and use of any special safety precautions, eg safety lines, remote controls, communications procedures, refuelling procedures, maintenance;
- the training requirements for persons operating drill rigs used for probe drilling purposes;

[1] Probe drilling should be carried out only on ground determined to be secure. Information obtained from exploration drilling, or grade control and blasthole drilling, may be of assistance when determining the shape and extent of underground workings.

- the procedures to be followed when working or drilling within a marked area that may be underlain by underground workings; the person responsible for approving entry to a marked area should be identified;
- the procedures to be followed when marking out the proposed probe drilling pattern;
- the sequence in which drilling should proceed – drilling operations should proceed from known safe ground towards the anticipated underground workings, see **Figure 1**;
- any equipment, eg tapes, inclinometers, etc required for the drilling activities;
- the capability to drill steeply dipping holes to determine a floor pillar thickness (measured vertically) is generally available; however, it may be necessary to drill shallow dipping holes to determine a rib pillar width (measured horizontally) in the walls of an open pit;
- the requirement for and method of completing any logging sheet to record the result of the drilling operations; eg driller, hole number, void depth, void size, descriptions of the ground conditions encountered, types of material encountered, drilling difficulties;
- the procedures to be followed when:
  - (a) difficulties occur in completing the hole to its planned depth;
  - (b) workings (either open or filled) are intersected during the drilling of a hole;
  - (c) workings are intersected at or less than the minimum stable floor pillar thickness or width (as determined in section 6);
  - (d) other potential hazards are intersected, eg gases, water, etc;
- the minimum and maximum probe hole drilling depth, angle and spacing; these should ensure that workings of hazardous magnitude do not remain undetected, eg ore passes, etc;
- the procedures to be followed when other personnel are working adjacent to any marked area when drilling is in progress, eg surveyors, samplers, maintenance crew, etc;
- special requirements or restrictions on carrying out maintenance work on the drilling equipment within a marked area; (ordinarily no maintenance work should be carried out within a marked area).

# 6.0 RISK CONTROL

## 6.1 Introduction

The control measures that are available to eliminate or minimise the risk of unexpected pit floor and/or wall collapse include:

- leaving a pillar of adequate dimensions between the current working bench or flitch and the underground workings;
- placing fill materials into the underground workings;
- restricting work to areas clear of the suspect location, with an adequate margin of safety;
- blasting waste rock in the pit floor into voids, followed by further back filling to stabilize the area.

## 6.2 Determination of adequate pillar dimensions

In all open pit mines where there is a risk of intersecting underground mine workings, appropriate studies must be carried out to determine the minimum stable pit floor pillar and/or rib pillar dimensions. The minimum pit floor pillar thickness is defined as the minimum rock cover, measured vertically, above the highest point of the underground workings which provides an acceptable factor of safety against floor pillar failure during all mining activities. The minimum rib pillar width is defined as the minimum rock and/ or soil barrier, measured horizontally, between open pit walls and adjacent underground workings which provides an acceptable factor of safety against wall failure.

The overall dimensions of the pillar, ie length, width (measured horizontally) and thickness (measured vertically) should be taken into account in any analysis of stable pillar thickness. Consideration should also be given to the appropriate factor of safety when selecting pillar dimensions. The factor of safety selected should be commensurate with the level of the risk posed by the extent of the underground workings and the nature of the rock mass.

The determination of the stable pit floor pillar thickness and/ or rib pillar width should be the result of a geotechnical engineering assessment in which specific attention is paid to:

- orebody geometry, particularly orebody dip and orebody width;

- the likely modes of failure of the stope crown pillar or floor pillar, whether controlled by, or independent of, geological structure;
- the likely modes of failure for the immediate hangingwall and footwall rocks whether controlled by, or independent of, geological structure;
- the potential accumulation of water in the open pit due to localised ponding via surface runoff from the surrounding catchment area and/or incident rainfall within the open pit perimeter;
- the loads imposed by equipment or stockpiles on the floor pillar;
- rock mass strength and/or general competence of pillar and wall rocks;
- "worst-case" geotechnical conditions with particular emphasis on structural geological features (planes of weakness), groundwater, variations in rock strength and their likely impact on the stability of the pit floor or rib pillars;
- the influence of open pit blasting on the integrity of the pillars;
- the relationship of pillar thickness to the width and strike length of stoped areas.

The adopted stable pillar thickness or stable pillar width will vary both within an individual site and from site to site, to reflect the extent of the hazard, the variation in controls on pillar stability, the range of geotechnical conditions, together with the extent and dimensions of stoping. The planned cut-back of an open pit wall may produce a situation where the stable rib pillar width that previously existed is reduced to unacceptable dimensions.

## 6.3 Open pit mining issues

Conventional open pit mining methods may need to be modified when mining above or through abandoned underground workings, when:

- mining through floor pillars smaller than the minimum stable thickness (the use of remote control of drilling and explosive charging operations may be required);
- backfilling narrow stopes (experience has shown that narrower stopes are potentially more difficult to backfill due to material "hanging up" or bridging across the stope walls);
- backfilling large stopes (backfill should not be relied upon as the sole means of providing safe working conditions);

- considering the use of mass blasting methods[2];
- mining through pillars and stopes has the potential to destabilise open pit walls[3]. This may have adverse consequences for mine infrastructure within or adjacent to the pit.
- maintaining the minimum safe working width on either side of stoped areas, particularly in the lower sections of narrow pits where mining widths may be restricted;
- controlling access to, and movement on, each bench or flitch, particularly where previous stoping may be continuous along strike.

## 6.4 Safe working procedures

A flow chart illustrating the key activities that need to be considered when an open pit is mining through abandoned underground workings is shown in **Figure 2**. This chart should be used as a basis for framing site-specific procedures, and the review and updating of all mining plans to ensure that an accurate model of the geometry of underground workings is maintained at all times.

Before commencing any open pit mining near or through abandoned underground workings an appropriate set of safe working procedures should be established that address a range of issues, including:

- probe drilling procedures;
- marking out the extent of the underground workings;
- drilling and blasting;
- plant and equipment movement;
- placement of fill materials in unfilled workings;
- rock stability monitoring;
- daylight and night operations;
- plant and equipment specifications;

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[2] Such use should be reviewed on a case by case basis having regard to the stability of the surrounding rock mass, adjacent open pit walls, the potential hazards associated with charging of blast holes in the vicinity of underground workings and the requirement to monitor explosive quantities loaded into each blast hole.

[3] Experience suggests that stope hangingwall instability may be more extensive with the potential to undercut open pit walls, particularly in large unfilled stopes.

- personnel movement;
- regular communication of information and discussion of issues of concern with all those involved.

These safe working procedures should be progressively reviewed as the open pit depth increases.

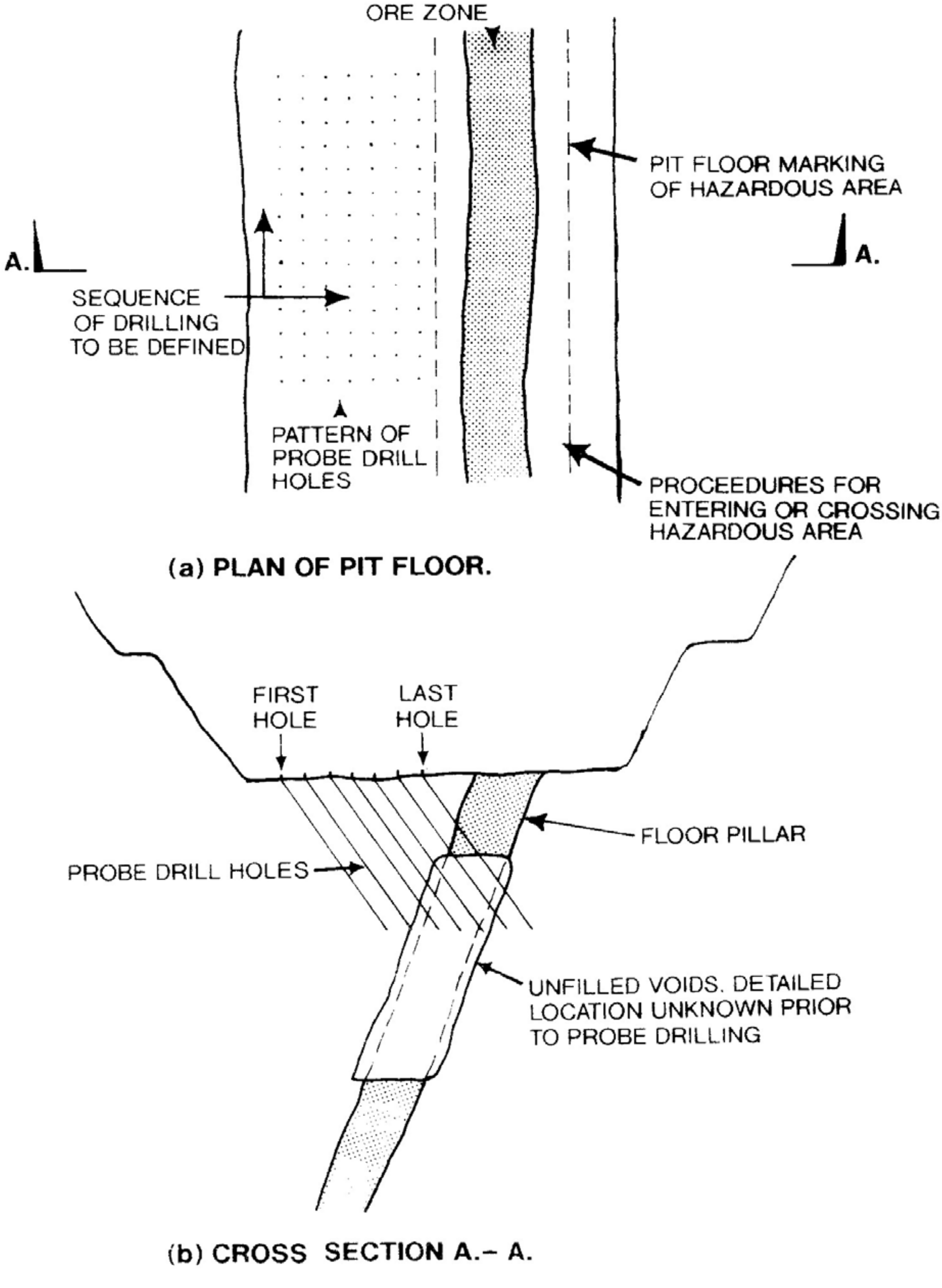


Figure 1. Probe hole drilling to locate underground workings

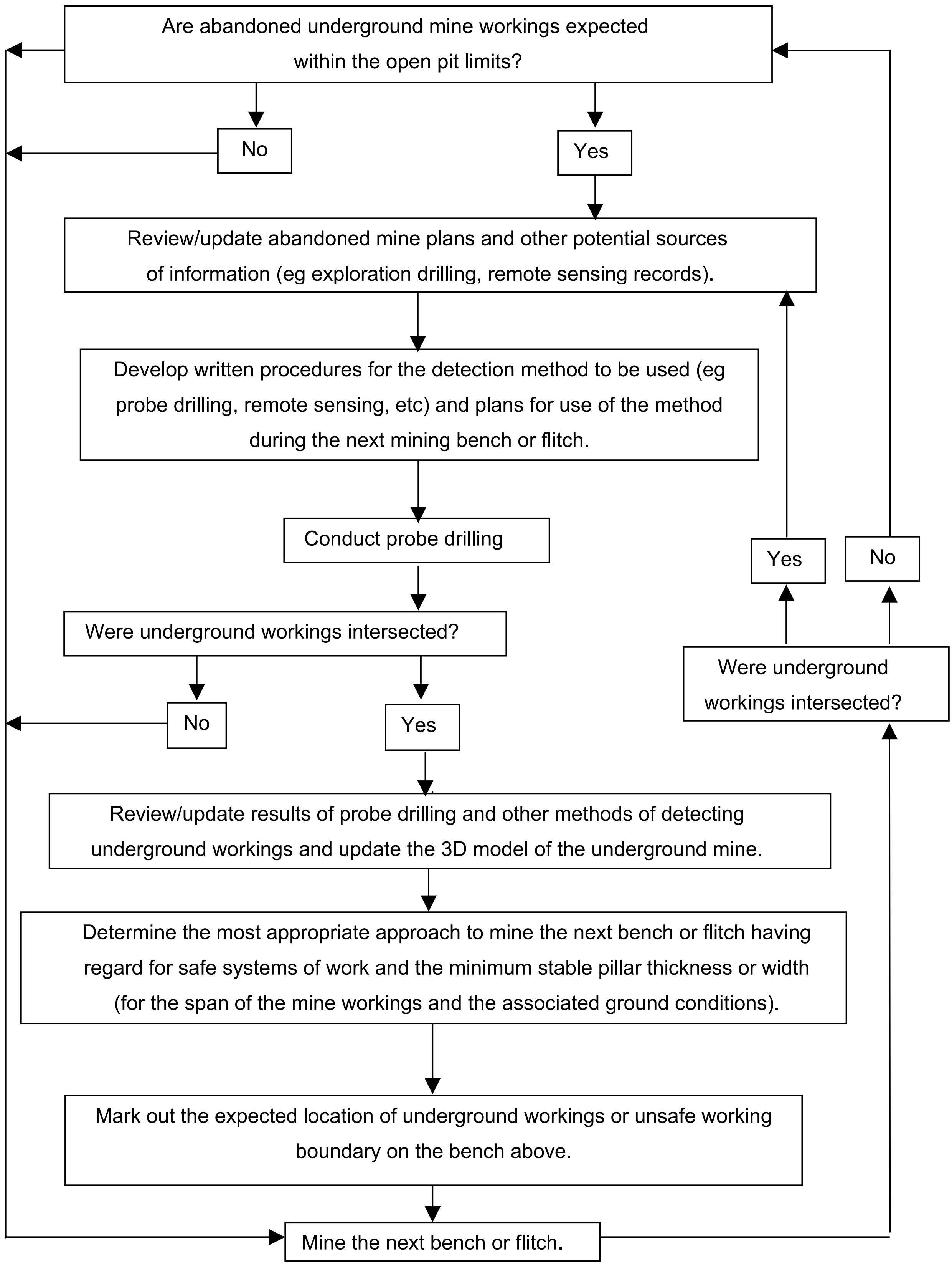


Figure 2. Flow chart for each bench or flitch, as appropriate, showing the key activities in open pit mining through abandoned underground workings

## APPENDIX A - HAZARDS ASSOCIATED WITH OPEN PIT MINING THROUGH CURRENT UNDERGROUND WORKINGS

Currently operating underground mines may face a number of potential hazards when open pit mining is conducted through underground workings associated with the underground mine.

The location and extent of the current underground mine workings should be known with a much greater level of confidence than is the case with abandoned underground mine workings. The use of current underground mine surveying methods and equipment should largely eliminate any uncertainty as to the location of current underground mine workings. The presence of large unfilled stope voids may result in large scale collapse of the surrounding rock mass into the stope void. When this occurs the extent and location of the boundaries of the underground workings (eg walls, backs, etc) will obviously change.

The hazards associated with open pit mining through current underground mine workings include:

- flooding of the underground mine workings from large water and/or mud inrushes via the open pit and surrounding catchment areas;
- flooding of filled stopes, by accumulated drainage or by inrush, containing uncemented or inadequately cemented fill materials, that may become saturated, causing bulkheads to fail under hydrostatic pressure, resulting in a fill or mud rush in the mine;
- instability of the open pit walls and the surrounding surface areas, including any mine infrastructure (ventilation fans, shafts, headframes, winders, buildings, mobile equipment, any underground mine excavations in close proximity to the open pit, rising mains, electric cables in bore holes, fill passes, bore holes used for delivery of fill materials, etc);
- adverse effects on the underground mine ventilation system (short circuiting, ingress of open pit blasting fumes and dust, rock falls in large open stope voids creating dust which is drawn into the main intake airways, etc);

- potential for collapse of large unfilled stope voids that may cause a significant change in the underground mine geometry;
- deficiencies in co-ordination, communication and control of mining activities between the open pit and underground mines.

Each of these hazards must be adequately investigated and controlled by appropriate means according to the identified risk.

# APPENDIX B - LEGISLATIVE FRAMEWORK

The *Mines Safety and Inspection Act 1994* sets objectives to promote and improve occupational safety and health standards. The Act sets out broad duties and is supported by more detailed requirements in the *Mines Safety and Inspection Regulations 1995*. A range of guidance material, including Guidelines, further supports the legislation. The legislative framework is set out in **Figure 3**.

Guidance material includes explanatory documents that provide more detailed information on the requirements of the legislation and include codes of practice and guidelines.

Guidelines contain practical information on how to comply with legislative requirements. They describe safe work practices that can be used to reduce the risk or work-related injury and disease and may also contain explanatory information.

Figure 3: Legislative framework

| | |
|---|---|
| MINES SAFETY AND INSPECTION ACT | ← Major provisions:<br>• The General Duties<br>• Management of Mines<br>• Enforcement of Act and Regulations<br>• Resolution of issues<br>• Safety and Health Representatives<br>• Safety and Health Committees |
| **Supported By** | |
| MINES SAFETY AND INSPECTION REGULATIONS | ← The Mines Safety and Inspection Regulations set minimum requirements for specific hazards and work practices, including reference to National Standards, developed by the National Occupational Safety and Health Commission, and Australian Standards developed by Standards Australia. |
| **And** | |
| GUIDANCE MATERIAL | ← • Codes of Practice approved for Western Australian mines in accordance with Section 93 of the Act.<br>• Guidelines produced by the Mines Occupational Safety and Health Advisory Board.<br>• National Codes of Practice and National Standards developed by the National Occupational Health and Safety Commission.<br>• Australian Standards developed by Standards Australia. |

The information included in a Guideline may not represent the only acceptable means of achieving the standard referred to. There may be other ways of setting up a safe system of work and, providing the risk of injury or disease is reduced as far as practicable, the alternatives should be acceptable.