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The whole truth and nothing but the truth? A case study comparing analytical and empirical assessments against site observations

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Abstract

Canadian mining operations have been key contributors to economic vitality for a long time. The closure of historic mines can be challenging as often the level of information available is much less than active mines preparing to close. This paper uses a case study of a closed mine site to evaluate industry standard design methods against observed site conditions to assess the most suitable method to be applied for mine closure. This comparison is of importance as analytical methods and empirical assessment methods are becoming more advanced and are commonly used in the mining industry. It stresses the importance of evaluating results against site observations to assess the validity of results.

This study focuses on Rock Mechanics and discusses the industry standard analytical methods used to assess the stability of the hanging wall and foot wall of the underground stope void, as well as the empirical method used to assess the stability of the crown pillar. The results from these methods are then compared to stability observations using bathymetric and three-dimensional sonar surveys collected of the historic mine workings. The information attained from the assessments is compared to the stability observations to confirm if evidence of the expected potential failure mechanisms exists at this historic mine site.

Keywords*: mine closure, bathymetric survey, three-dimensional sonar survey*

1 Introduction

With more than a century of mining, many historic mining operations in Canada have been abandoned, leading to legacy projects that require mine closure procedures adhering to regulations specific to their location. The project site discussed in this study is in northern Canada and used interconnecting open pit and underground mining methods whilst in operation but has been closed for over half a century. Historic documents indicate that the mine was closed after a failure occurred at depth and backfill material was lost down into the mine. The studies showed that backfill was still present along the western end of the open pit. A pond currently exists where the open pit was located.

Regionally, this deposit is located within a greenstone belt with a history of tectonic deformation, volcanic and intrusive activity. Locally, the deposit was hosted in a medium to coarse grained diorite unit. This site is located near to community facilities in a mining town. The location of this old mine site highlighted the need to conduct detailed stability assessments of the surrounding area. Any instability in the hanging wall, foot wall and/or crown pillar could have substantial impacts on the surrounding area.

The paper will first detail the available information from this site. Next, three of the stability assessments completed for this project site excavation will be described. Additional assessments were completed as part of this work; however, this paper will only discuss three of the assessments, to remain concise. Finally, the conclusions drawn from the assessments will be discussed and compared with the available ground truthing data available for this site.

2 Data

The available data for this site comprised of:

- Collection and review of historic drawings and photographs;
- Review of data obtained from Time Domain Reflectometry (TDR) cables installed to shallow depths in the early 1990s; and
- A bathymetric and 3D sonar survey completed from the pond location and into the connected flooded workings, shown in Figure 1.

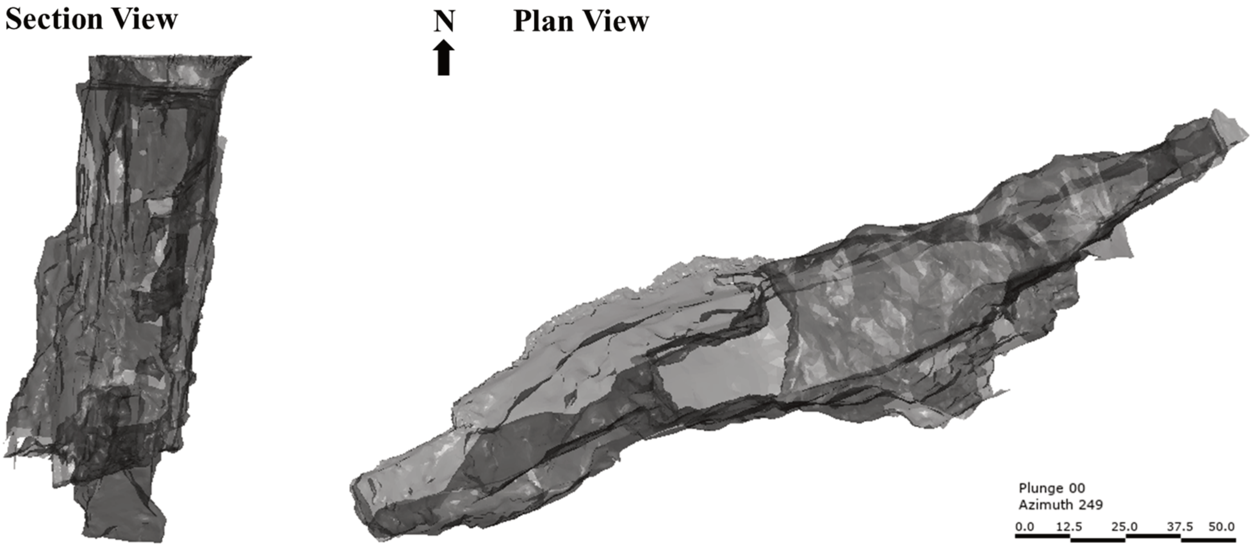

Figure 1 Section and Plan view of the results of the 3D sonar survey showing the underground stope void

Following the completion of the 3D sonar survey, a field programme consisting of five geotechnical drillholes to characterise the rock mass conditions and determine the discontinuity orientations surrounding the pond and historic underground workings, was undertaken. The on-rig logging of the oriented drill core from triple-tubed drilling, provided the basis for calculation of the rock mass parameters used for the stability assessments (RMR76), as presented in Table 1 and discontinuity orientations as shown in Figure 2 and summarized in Table 2.

Table 1 Rock Mass Parameters

Hole ID	Drillhole Depth (m)	TCR (%)	RQD (%)	IRS	FF/m	RMR_{76}
19-01	74.54	100	96	R4	3.2	59
19-02	107.54	99	94	R5	4.2	63
19-03B	14.69	95	74	R5	7.8	56
19-04	71.79	91	82	R5	15.6	60
19-05	12.36	100	74	R4	13.8	47

The rock mass values are inclusive of intact rock material only and do not include the backfill material that was intersected at the bottom of certain drillholes. TCR= Total Core Recovery; RQD=Rock Quality Designation; IRS = Intact Rock Strength; FF/m=Fracture Frequency per meter and RMR = Rock Mass Rating.

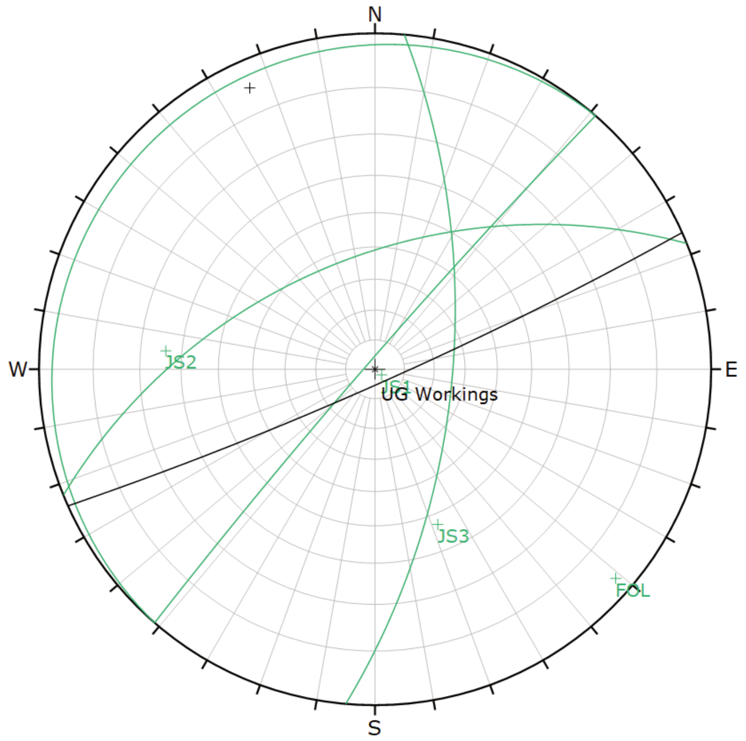

Figure 2 Major Discontinuity Set Orientations as well as the hanging wall and foot wall orientation

Table 2 Major Discontinuity Set Orientations from the Geotechnical Drilling Campaign

Discontinuity	Dip (°)	Dip Direction (°)
Foliation (Fol)	87	311
Joint Set 1 (J1)	03	310
Joint Set 2 (J2)	64	095
Joint Set 3 (J3)	53	338

3 Stability Assessments Results

This section will focus on describing three of the stability assessments completed for this project site. Two analytical approaches were used to assess the stability of the hanging wall and foot wall and an empirical approach was used to assess the stability of the crown pillar.

3.1 Hanging Wall and Foot Wall Kinematic Stability Assessment

The discontinuities present in the rock along which an excavation is created, will influence the stability of that excavation. The rock mass is made up of an interlocking matrix of discrete blocks, with block size being controlled by discontinuity orientation and persistence. When exposed in an excavation, these structures interact, with the potential for the formation of tetrahedral blocks or wedges that may be unstable.

Two analytical approaches, Unwedge and Swedge, were used to assess the stability of the hanging wall and foot wall of the underground workings and the open pit slopes respectively.

3.1.1 Unwedge Analysis

Unwedge version 5 (Rocscience, 2019) was used to evaluate the potential unstable blocks that might form along the hanging wall and foot wall of the underground excavation, considering the major joint set orientations identified in the field programme, presented in Table 2. Three scenarios highlighting areas where wedges might potentially form along the surveyed void as shown in Figure 3, for analyses.

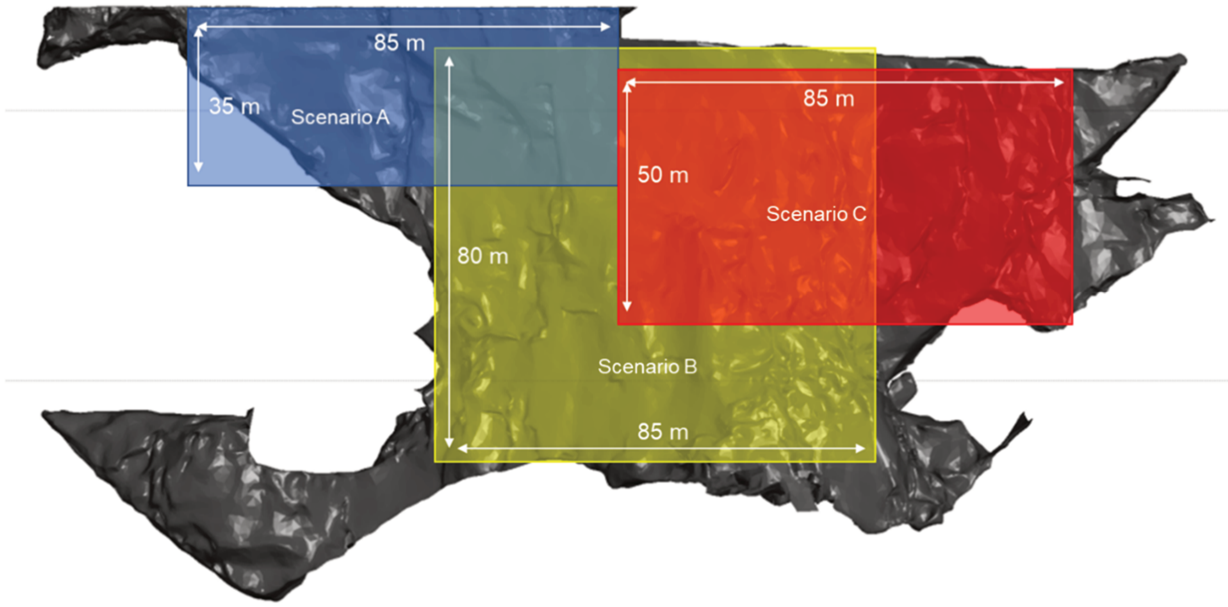

Figure 3 Sonar surveyed excavation (grey area) of the underground stope void, including three scenarios considered in the Unwedge assessment. Image facing north

The initial joint properties in Table 3, were considered to be conservative for the host diorite. These properties were modified using the approach proposed by Barton and Bandis (1990) for rock mass anisotropy to better consider joint persistence and the impact of rock bridges. For this assessment, a conservative 90% persistence (10% rock bridges) was assumed for all the joint sets identified in the drilling investigation, with the resulting joint and rock mass properties summarized inTable 4 below.

Table 3 Initial Conservative Discontinuity Parameters for the Major Joint Sets

Discontinuity Parameter	Friction Angle (°)	Cohesion (MPa)	Tensile Strength (MPa)	Water Pressure (MPa)
All major discontinuity sets	25	0.1	0	0

Table 4 Discontinuity Parameters of the Major Joint Sets after Barton & Bandis approach

Feature	Parameter	Foliation	J1	J2	J3
Joint	Persistence (%)	90	90	90	90
Joint and Rock mass	Equivalent Cohesion (kPa)	1272	1272	1290	1272
	Equivalent Friction Angle (°)	31	31	35	27

For Scenario A, the Unwedge assessment showed that no unstable wedges were present along the hanging wall, foot wall, and sidewalls of the underground stope void.

For Scenario B (Figure 4), there were two kinematically possible wedges, that might form with Factor of Safety (FoS) values less than 1. The roof wedge with an apex height of 1.56 m, that would have formed along the roof of the crown pillar, was most likely removed during the excavation of the underground workings. The kinematically possible wedge along the hanging wall side of the underground stope void has an apex height of 4.63 m.

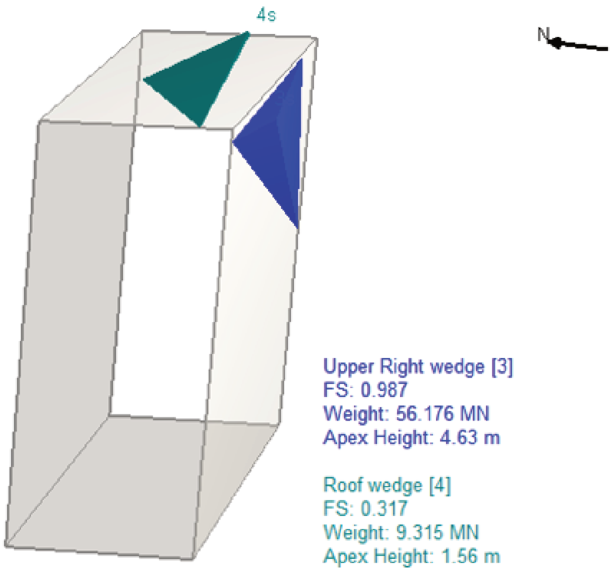

Figure 4 Kinematic Assessment Results for Scenario B (Rocscience, 2020)

For Scenario C (Figure 5), there are two kinematically possible wedges that might form with FoS values less than 1. The roof wedge has a maximum apex height of 7.01 m. The kinematically possible wedge along the hanging wall side of the open pit pond has an apex height of 5.29 m.

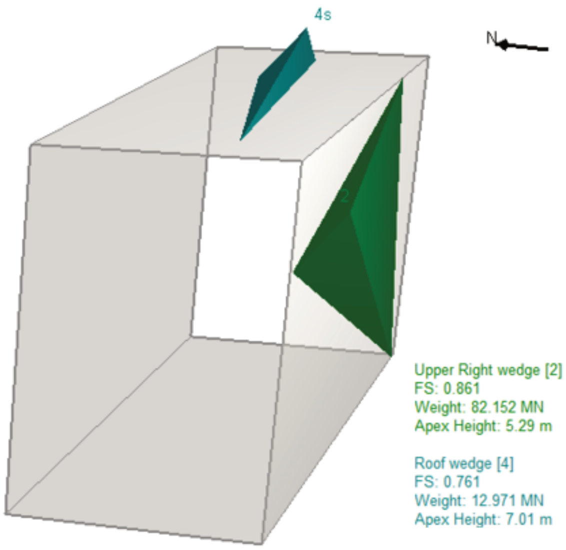

Figure 5 Kinematic Assessment Results for Scenario C (Rocscience, 2020)

3.1.2 Swedge Analysis

The Swedge code (Rocscience, 2020) is an analysis tool for evaluating the geometry and stability of surface wedges in rock slopes. The combinations and the probabilistic analysis were used to assess the wedge stability of the slopes of the open pit pond area.

This first step was to identify kinematically possible wedges utilising the conservative joint properties presented in Table 3. The two scenarios considered, included the whole length of 215 m of the underground excavations, assuming that the crown pillar was not present, and a reduced length considering the open pit pond only, with a length of 107 m. The analysis was conducted for both the hanging wall and foot wall. The analyses did not consider the presence of the large quantity of backfill that remains in the western end of the open pit and as such these assessments are considered to be conservative.

The results obtained from the initial combinations analysis were used to identify potentially adverse joint combinations, to be assessed further in the probabilistic analysis. Statistical distributions for the joint orientation parameters were used to account for variability in the joint orientation based upon the kinematic data collected during the field programme. The probabilistic assessment used the joint parameters presented in Table 4.

A FoS distribution from which a probability of failure is calculated was obtained from the results of the probabilistic analysis. The results of the Swedge assessment are summarized in Table 5Table 5 Summary of results from the Swedge probabilistic analysis.

Table 5 Summary of results from the Swedge probabilistic analysis

Scenario	PWP and Seismic Forces	Joint combination	Wedge Height	Break back distance from the crest	FoS	PoF (%)
	Hanging Wall					
Whole length	No	Fol and JS2b	30 m	16 m	> 5	0
Whole length	Yes	Fol and JS2b	30 m	15 m	> 5	0
Open pit pond	No	Fol and JS2b	15 m	8 m	> 5	0
Open pit pond	Yes	Fol and JS2b	15 m	8 m	> 5	0
	Foot Wall					
Whole length	No	Fol and JS2a	90 m	11 m	> 5	0
Whole length	Yes	JS2a and JS3b	90 m	36 m	0	90
Open pit pond	No	Fol and JS2a	45 m	5 m	> 5	0
Open pit pond	Yes	JS2a and JS3b	45 m	17 m	0	90

PWP = Pore Water Pressure, FoS = Factor of Safety, PoF = Probability of Failure

The results indicate that that probability of failure in the hanging wall for these joint combinations is zero and the FoS is greater than 5. However, the probability of failure for two of the joint combinations along the foot wall indicate the potential for unstable wedges.

3.2 Empirical Crown Pillar Assessment

The rock mass situated above the upper limits of a mine is referred to as the surface crown pillar. The crown pillar is required to prevent inflows of soil, rock, and water from surface into the mine while protecting infrastructure, the environment andland users occurring along the ground surface area of the mine.

The conventional approach to assessing subsidence effects over metal mines is to consider the stability of the crown pillar formed between the upper limit of mineral extraction and the ground surface. With this approach, it is essential to gather geotechnical information from the rock mass that will form the crown pillar.

In general, failure of underground openings is progressive and will not stop until:

- There is a significant improvement in rock mass properties;
- The effective span is reduced by the support provided by the failed rock;
- A stable rock arch is formed; or
- The space available for failed rock to occupy is insufficient with the result that the failure chokes off.

3.2.1 Scaled Span Crown Pillar Design Approach

To address the challenges with assessing crown pillar stability, the Scaled Span Crown Pillar design approach was initially developed by Carter in 1992 and has been regularly updated by the same author and others (Carter et al., 2008). This empirical method divides the assessment into a geometric assessment of the crown pillar, the Scaled Crown Span, and rock mass quality derived component, the Critical Span. Both relationships are defined by the expressions presented below.

Scaled Crown Span:

$$C_S = S \times \sqrt{\left\{\frac{\gamma}{t \times (1 + S_R).(1 - 0.4 \times \cos\theta)}\right\}}$$

Where:

- Cs Scaled Crown Span (m)
- S crown pillar span (m)
- t crown pillar thickness (m)
- γ specific gravity of the rock mass
- SR span ratio = crown pillar span / crown pillar strike length

Critical Span:

$$S_C = 3.3 \times Q^{0.43} \times sinh^{0.0016} Q$$

Where:

- Sc Critical Span (m)
- Q NGI Q rating value

In cases where the Scaled Crown Span is less than the Critical Span, the crown pillar would be considered as stable, where this relationship is reversed the pillar would be considered unstable. The 2008 update of the design approach provides guidelines for levels of risk that could be deemed acceptable for given situations.

3.2.1.1 Design Guidelines

The existing underground excavation at the project site had failed to surface in certain locations. In other locations the underground excavations were approximately 7 m from surface. This excavation consisted of mined out blocks, failed pillars and backfill material.

This empirical assessment had been completed using the average rock mass characteristics determined from the field programme, considering only the thickness of rock. The input parameters included the specific gravity of 2.91 g/cc for the rock mass. From the 3D sonar survey the dip of ore body was measured to be approximately 85°, the strike length of the crown pillar was measured to be approximately 104.2 m and the crown pillar thickness was measured to be approximately 7 m.

Figure 6 show the plots for the minimum crown pillar span of 10 m and the maximum crown pillar span scenarios. The block represents the range of spans between 10 m to 35 m. Figure 6 shows that for the 10 m crown pillar span, the crown pillar is classified as a Class D to E where the guideline (Table 6) describes this as semi-temporary crowns with a lifespan of 5 to 20 years. The 35 m crown pillar span is classified as a Class A where the guideline (Table 6) indicates that the serviceable life of a Class A pillar is effectively zero (less than half a year).

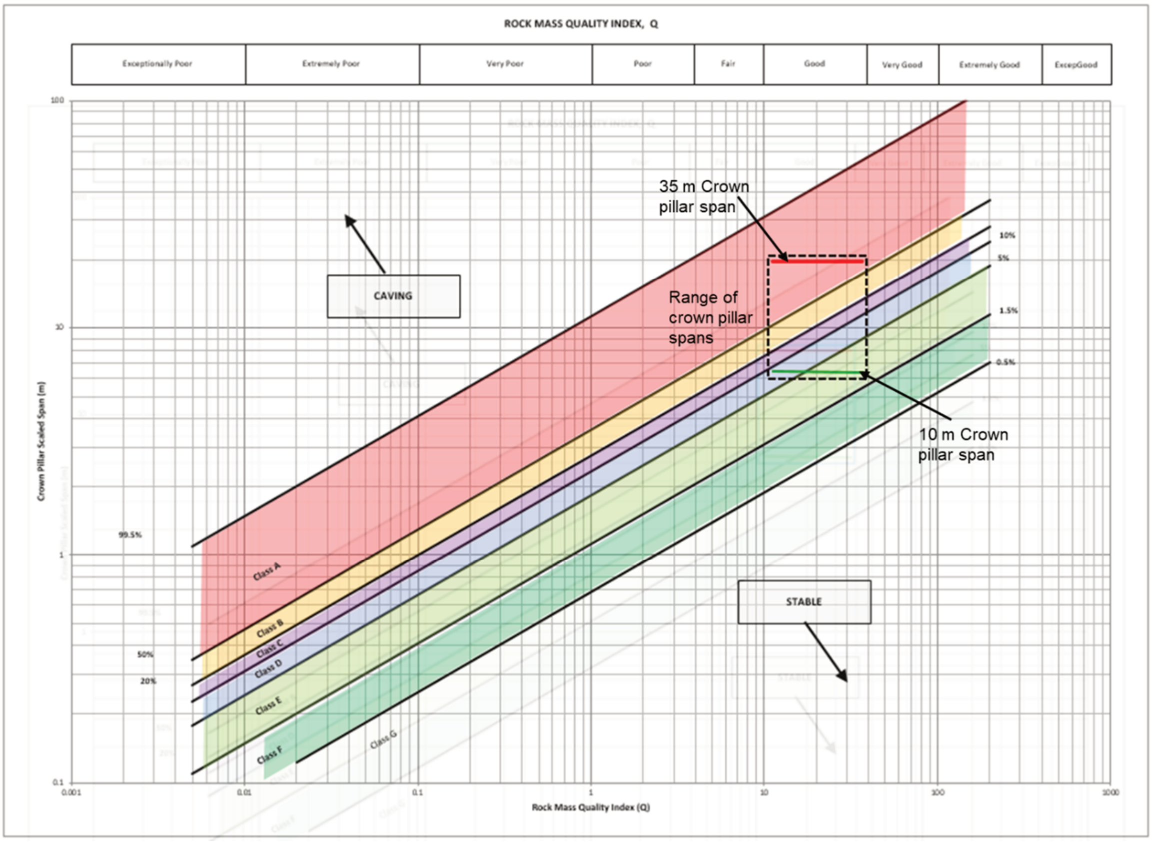

Figure 6 Scaled span plot for a crown pillar span of 10 m to 35 m

Table 6 Acceptable risk exposure guidelines (Carter et al., 2008)

Class	Probability of Failure (%)	Minimum Factor of Safety	Maximum Scaled Span, Cs (=Sc)	ESR (Barton et. al. 1974)	Design Guidelines for Pillar Acceptability/Serviceable Life of Crown Pillar				
					Expectancy	Years	Public Access	Regulatory position on closure	Operating Surveillance Required
A	50 - 100	<1	$11.31Q^{0.44}$	>5	Effectively zero	< 0.5	Forbidden	Totqally unacceptable	Ineffective
B	20 - 50	1.0	$3.58Q^{0.44}$	3.0	Very, very short-term (temporary mining purposes only; unacceptable risk of failure for temporary civil tunnel portals)	1	Forcibly prevented	Not acceptable	Continuous sophisticated monitoring
C	10 - 20	1.2	$2.74Q^{0.44}$	1.6	Very short-term (quasi-temporary stope crowns; undesirable risk of failure for temporary civil works)	2 - 5	Actively prevented	High level of concern	Continuous monitoring with instruments
D	5 - 10	1.5	$2.33Q^{0.44}$	1.4	Short-term (semi-temporary crowns, e.g. under non-sensitive mine infrastructure)	5 - 10	Prevented	Moderate level of concern	Continuous simple monitoring
E	1.5 - 5	1.8	$1.84Q^{0.44}$	1.3	Medium-term (semi-permanent crowns, possibly under structures)	15 - 20	Discouraged	Low to moderate level of concern	Conscious superficial monitoring
F	0.5 - 1.5	2.0	$1.12Q^{0.44}$	1.0	Long-term (quasi-permanent crowns, civil portals, near surface sewer tunnels)	50 - 100	Allowed	Of limited concern	Incidental superficial monitoring
G	< 0.5	>>2	$0.69Q^{0.44}$	0.8	Very long-term (permanent crowns over civil tunnels)	> 100	Free	Of no concern	None required

5 Conclusions

5.1 Hanging Wall and Foot Wall Assessment Conclusions

The results of the 3D sonar survey increased confidence in the stability of the underground void and remaining sill pillars at the time of the study. The 3D sonar survey is of high quality and even detected features underwater such as a satellite dish and a boat. The survey confirmed that there was not extensive rubble on the floor of the excavation, which would have buried these items. This suggested that the hanging wall and foot wall failures were limited.

Careful review of the 3D sonar survey data did not indicate evidence of the kinematically possible failures identified in the Unwedge and Swedge assessments, or crown pillar failures. This survey did not show conclusive evidence of the formation of tetrahedral rock blocks along the 3D sonar survey boundary. Therefore, it is most likely that the spacing of the joints, the in-situ joint strength and/or the confining stresses in the vicinity of the excavations are contributing to the stability of the surrounding rock.

5.2 Crown Pillar

Using the Scaled Span method to assess the crown pillar stability, it was found that in the section of the crown pillar that has a span of 35 m, the crown pillar would be expected to fail within 6 months of being mined out. In the area where the crown pillar span is smaller (approximately 10 m) the crown pillar would be expected to fail in 5 to 20 years.

When the results of the scaled span method are compared to a detailed review of the 3D sonar survey and review of the available historical photographs there appeared to have been little, if any, degradation of the rock forming the crown pillar. The underside of the crown pillar appears to have been formed on the sub-horizontal planar joint, J1, resulting in a smooth planar surface visible in the 3D sonar survey image with no indication of degradation. This crown pillar has been stable for over 90 years without failing, or showing any significant degradation, which is at odds with the empirical assessment results.

The experienced gained at this project site indicate that the empirical and analytical tools used to complete the stability assessment did not correspond with the observed excavation stability. This case study serves to further reinforce the importance of collecting observations of excavation performance, where possible, to determine the behaviour of the excavation. These observations should be considered so that an appropriate engineering judgement can be made.

References

Barton, NR & Bandis, SC 1990 'Review of predictive capabilities of JRC-JCS model in engineering practice'. in N Barton & O Stephansson (eds), *Int. symp*. *on rock joints*, Loen, Norway, Rotterdam: Balkema. pp. 603-610.

Carter, TG, Cottrell, BE, Carvalho, JL, & Steed, CM 2008 Logistic Regression improvements to the Scaled Span Method for dimensioning Surface Crown Pillars over civil or mining openings, in *The 42nd U.S. Rock Mechanics Symposium (USRMS)*, San Francisco, California

Rocscience Inc. 2019, *Unwedge Version 5.* www.rocscience.com, Toronto, Ontario, Canada

Rocscience Inc. 2020, *Unwedge Version 5*. www.rocscience.com, Toronto, Ontario, Canada

Rocscience Inc. 2020, *Swedge Version 7.* www.rocscience.com, Toronto, Ontario, Canada