Addressing Mine Hazards under Transportation Corridors – The Timmins Experience

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Abstract

Portions of Highway 101 in Timmins Ontario, near the intersection with Highway 655, are located above and in the vicinity of old mine workings of the famous Hollinger and McIntyre mines. The extraction methods employed at these mines were primarily cut-and-fill and shrinkage stoping to exploit a series of parallel, narrow, steeply dipping gold veins within the Porcupine-Destor Fault. Some of the mine openings (stopes and raises) in this area have been mined up to the bedrock surface and backfilled with unconsolidated sand backfill. Over the years, subsidence has been experienced at a number of properties, particularly in the vicinity of the Highway 101-655 intersection. These events have led to mitigation measures being implemented at various stages over the past half century. Such measures have included sand backfilling of open workings, fencing and the construction of concrete capping structures.

This paper discusses the processes used to identify, investigate and remediate the mined out areas where these surface disturbances and potentially unstable crown pillars were threatening to affect surface infrastructure, particularly the main highway. The paper is organized into the following sections: identification of hazardous areas, investigation techniques, assessment and analysis, and remediation options.

1.0 BACKGROUND

The City of Timmins in northeastern Ontario, Canada was founded in 1912 following the discovery of gold in the area. Over the next half century, gold mining in the Timmins-Porcupine District continued at up to 80 producing mines, the most prolific being the Hollinger and McIntyre mines. By the 1950s, consolidation of the historic mining companies had occurred, with less than half a dozen major mining companies still continuing in production. By this time, Timmins had become the most productive gold-district in North America, with total historic production in excess of 62 M oz of gold, rivalling the famous gold reefs of South Africa. This production in most part had come from quartz-carbonate lode systems hosted within volcaniclastic rocks of the Abitibi greenstone belt. Former and current producing mines now dot a corridor over 50 km in length and up to 10 km in width paralleling the regional Destor-Porcupine Fault, which controls the gold distribution within the greenstone belt. The main arterial road that once connected all the historic mines has been upgraded over the years and now forms Highway 101, the main thoroughfare through the modern city of Timmins. This road, however, runs in many sections over the top of extensively mined ground following along parallel with this major regional fault.

The Hollinger Mine closed in 1968 and the McIntyre in 1989. When pumping at the McIntyre Mine ceased, both mines began to flood since the underground workings of the two mines were connected. Water levels began a slow rise and in 1999 a series of dramatic subsidences (sinkholes) occurred at a number of locations on the former mine sites. Most problematic were subsidences on former mined land that had been redeveloped for other commercial, industrial, recreational or municipal uses.
Several sinkholes and crown pillar collapses developed, particularly in zones where stoping had been continued up to or close to surface (examples shown in Plates 1 and 2). Most of the stopes that showed problems had been mined by cut-and-fill methods and had been left sand-filled at the cessation of mining operations. With the rise in water levels, it is believed that timber bulkheads in the old workings collapsed and sand backfill migrated into other unfilled mine workings.

While the Hollinger Mine, for example, had substantial historic production producing 65,890,000 short tons of ore between 1910 and 1968, the records documenting the fill status of various stopes were found to be incomplete with no accurate assessment of the available volume of void space. It was known, though, that significant undefined void space existed associated with the hundreds of miles of interconnected underground drifts and stoping areas, all of which would provide considerable opportunities for the movement of backfill.

Sand backfill was originally placed dry into the stopes. With the rising water levels, the fill saturated, thus lowering the angle of repose and promoting the movement of backfill to lower level openings within the mines. As the backfill moved down within the mines, during the period of rising water levels, the stopes nearest to surface were progressively depleted of supporting sand fill; allowing crown collapse to develop in some areas, and sinkholes to develop in other areas where the stopes had been mined through to surface and where overburden soils were free to move into the open voids. Lines of surface sinkholes developed along the strike of the ore zone with tension cracks developing outwards around an increasing zone of influence as the sand backfill continued to move into the deeper mine workings. In other areas, crown pillars that had been tight filled with sand backfill were left unsupported and stope sidewalls were left without confinement, raising potential risk levels for larger scale collapses to occur.

Compounding the problems of subsidence, the mine’s owner, Royal Oak, was having financial difficulties, and by mid-1999 was under control of a receiver. With high historical liabilities on their properties and low gold prices, there was a concern that no buyer would be found for the Timmins properties and they would become orphaned. Fortunately, the Ontario Government and Kinross Gold Corporation entered into an agreement regarding subsidence liabilities and in December 1999 Kinross acquired the Timmins properties. The properties are now owned jointly by Kinross and Placer Dome CLA through the Porcupine Joint Venture.

At the same time, subsidence risks were agreed to be jointly managed via a Steering Committee made up of personnel from the Ministry of Northern Development and Mines and the Porcupine Joint Venture. This allowed risks to be prioritized and for them to be managed through an Engineered Risk Assessment approach, which could be based on the severity of the hazard and degree of public exposure. Over the period from 2000-2003, covering the works described in this paper, the Steering Committee commissioned numerous geotechnical studies and implemented a series of rehabilitation projects for securing and managing the most severe of the identified subsidence hazards. This paper is, in part, based on work commissioned by the Steering Committee in this period, but also includes some discussion of follow-up work conducted by Golder Associates in subsequent years.
2.0 MINING COMPANY PERSPECTIVE

When Royal Oak Mines closed and went into receivership, the remaining resources under their control were of interest to other mining companies. The presence of significant mining hazards posed a public safety risk/liability, and thus made the acquisition of the mining properties problematic for any potential deal. Further, the receiver saw no future in marketing the assets that held remaining value without including the assets that appeared to be only liabilities. The Ontario Government, rather than having these properties all fall entirely to the public sector, had an incentive to negotiate a cost and risk sharing agreement with any potential buyer, so that they could share the costs and liabilities. As well, in sharing the costs and risks on the properties with liabilities, development of the other properties might be possible, increasing employment and economic activity, delaying closure obligations and increasing the tax base. As such, Kinross and the Ontario Government, through the Ministry of Northern Development and Mines, negotiated an agreement that essentially allowed Kinross to subsequently purchase all the Royal Oak properties.

Part of the agreement involved the active investigation and remediation of the problematic hazard sites on the various properties. Success with the program depended on clear, frequent and consistent communication with the City of Timmins. City counsellors were briefed on the program on a regular basis. As a consequence, very effective cooperation with City Administrators, City Engineering and the police and fire services were secured. The Steering Committee recommended to the City that development of land, take into account mining hazards as part of the approval process, and that new development on or near mining hazards be prohibited or constrained. As well, it was recommended that when the City developed its geographical information system, that mining hazards be documented in that system. It was hoped that through these measures the likelihood would be reduced that hazards would develop in the future.

3.0 IDENTIFICATION OF HAZARDOUS AREAS

With the onset of surface subsidence over widespread zones along the strike of the main ore bodies, it became apparent that a need existed to rapidly identify the surface infrastructure areas of the City that were at most risk due to the influence of the rising water levels in the Hollinger and McIntyre mine workings. Initial actions and investigations that were triggered by the occurrence of the first sinkholes were intended to address the immediate
The first step in this process required pulling together all the information available on the mined geometries of the upper levels of the mines. This process started by collecting all relevant historical mine records. Hundreds of old vellum and linen drawings, many dating back to the earliest years of mining were digitized and integrated into a GEMCOM model using a surface down priority system (Mine Level 200, then 300, then 400, etc.). Challenges in this process included: missing records not found for areas known to have been mined, inaccurate and incomplete records especially relating to the geometry of final cuts or slough zones existing within the near surface stopes, changes in mine ownership, inconsistencies with naming conventions and inconsistencies with surveying and with the various mine coordinate systems that had been employed over the nine decades of mining.

Ultimately, a reasonably accurate and complete 3D model of the stoping geometries of the old workings was completed. The surface topography and the layout of local roads and buildings were also included in the model, as were the surveyed locations and sizes of the known hazards and sinkhole locations. This model focussed on the Highway 101 corridor near the Highway 655 intersection. This area encompassed only a portion of the hazards managed by Kinross and the MNDM. Once complete, however, this digital model became a key element in helping with the identification of additional hazardous areas. As such, it not only provided the basis for planning the necessary investigations, but it also became a repository for additional information as it was generated.

Two types of hazards were the main focus of the early assessment: – (i) openings to surface and – (ii) thin crown pillars, which were potentially unstable. Based largely on the information in the GEMCOM model, eleven sites were identified as top priorities for hazard assessment.

Preliminary evaluation of likely rock conditions and stope layouts, based on available mine plans and cross-sections, were used to define an initial prioritization schedule for initiating the sequence of more detailed investigation and budgeting for the possible investigation and remediation costs. The same basic analytical methods used for the more detailed subsequent analyses were used in this preliminary screening (as listed in Table 1 below). These analyses were used in order to rank the stability state. The risk exposure to the public was then reviewed in the light of the guidelines shown in Table 2 (after Carter and Miller, 1996). The current situations with respect to hazard control (fencing, capping, etc.) were then listed and a prioritization ranking generated to identify the highest to lowest risk sites from the viewpoint of implementing immediate investigation and monitoring (either by visual or instrumentation methods).

<table>
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<tr>
<th>SITE</th>
<th>Analysis Results and Rankings</th>
<th>Stability</th>
<th>Current Exposure</th>
<th>Existing Remedial Works</th>
<th>Need for Monitoring</th>
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<td>Factor of Safety FoS</td>
<td>Probability of Failure Pf</td>
<td>Factor of Safety FoS</td>
<td>Probability of Failure Pf</td>
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Table 1 - Preliminary Ranking of Eleven Hazard Areas for Budget Definition and for Prioritizing Need for Investigation and Monitoring
With the preliminary hazard rankings defined, a sequenced evaluation procedure could be adopted, starting with investigation of the highest priority hazard sites, and then moving on to the sites with assumed lower risks. The investigation approach in some cases had to be specifically tailored to the conditions at the different problem sites to best fit the inferred hazard stability situation. In consequence, during the course of the evaluations, several different investigation methodologies were adopted.

4.0 INVESTIGATION TECHNIQUES

Investigation techniques largely focused on determining the geometry of the mine workings and conversely the geometry of the intact bedrock. Information on the quality of the rock mass, structural fabric of the rock and data on the configuration of larger discrete structural features were also of key interest. The range of techniques utilized during the course of the evaluations of the various identified potentially hazardous near-surface openings included the following: borehole drilling and logging, borehole video surveys, application of various geophysical methods, select open excavation mapping, and a number of cavity monitoring surveys (CMS).

4.1 Borehole Drilling and Logging

Boreholes were drilled using NQ triple tube wireline techniques to achieve the highest possible quality rock core recovery through the partially stable crowns and hangingwall areas of the stope. All the recovered drill cores were geotechnically logged as drilling progressed, so that the most accurate assessment could be made of in situ rock mass characteristics. The rock mass characteristics allowed subsequent stability analyses to be performed to check the condition of the various crown pillars and stope wall zones that were considered potentially hazardous. Detailed borehole logs were prepared based on the core logging results. Each borehole was cased to bedrock and following drilling, the casings were left in place, cut off flush with ground level and capped. The reason for this was to provide future access for subsequent video inspection of ground conditions as well as provide potential cost-effective locations for instrumentation installation and monitoring.

In combination with downhole videos and CMS surveys, this technique of leaving a fully cased hole allowed very accurate evaluation of both the top of bedrock and of the location of the excavation boundary for each surveyed
stope. Retrieval of high quality cores also allowed the rock mass quality and structural fabric of all hazard zones to be properly and adequately assessed.

4.2 Borehole Video Surveys

Following drilling, a downhole camera survey of each borehole was carried out to aid in identifying the location, nature and orientation of any discontinuities or ground conditions that might be influencing stability. Borehole camera logging was carried out using a SeeSnake borehole camera. This compact, self-illuminating, water-resistant camera has a field of view of 77 degrees (in water) providing a black and white image of the entire wall of the borehole. As the camera was lowered down each borehole, an image was continuously displayed on a TV monitor and recorded by a VCR onto standard VHS videotape.

Video analysis of the camera surveys allowed the strike and dip of visual features to be identified and described. Analysis also allowed the relative perspective of the location and orientation of the identified features to be classified as visually identifiable open joints or “breakouts” for each site. Breakouts were defined as measurable voids or open fractures with apertures in excess of approximately 4 cm (approximately half the diameter of the NQ boreholes).

Stereonet analysis comparing typical orientation of the open joints relative to the breakouts was also carried out as a result of the camera survey analysis. This technique allowed further understanding of the rock mass fabric to be gained, as well as information on discrete structures (i.e. breakouts, open joints, faults) to be collected. As many of the holes were deliberately drilled to breakthrough into the stopes, data on the termination of each borehole also provided significant, useful information not only on crown rock conditions, but also on differences between actual rock geometry and mined geometry as inferred from available original stoping drawings. Using the videos, details were also collected on extent of caved rock, on the geometry of sand backfilling within the stopes, and wherever sand or other stope fill existed, details were collected of any gap between the base of the crown pillar and the backfill. For the case of open stopes, similar details were collected on the geometry of the stope crown and sidewall conditions.

4.3 Geophysical Techniques

Several geophysical techniques were considered for helping in defining the extent of mined workings (Phillips et al., 2003). These techniques included seismic refraction, seismic reflection, spectral analysis of seismic surface waves (SASW or MASW), ground penetrating radar (GPR), electrical resistivity imaging (ERI), and microgravity. It was found that the suitability of the various techniques was quite dependent on the geological environment, the depth to and geometry of the workings, and the potential for interference from infrastructure at surface.

Because of the expected depth to bedrock, the soil and rock types and the presence of infrastructure at surface, seismic refraction methods were chosen as being the most suitable overall geophysical approach. The objective of the seismic refraction survey was to estimate the depth to bedrock and hence define the elevation of the upper bedrock surface in the areas where stoping likely extended through to surface or came close to surface.

The refraction surveys were found very successful in defining the presence of open cuts and for delineating depressions in the bedrock surface where stoping of geological veins likely existed. Data on the competence of the rock mass (based on P wave velocity determinations) was also found useful for aiding definition of potential mined zones. It was found that in most cases a two-layer system could be identified – an upper layer with velocities typically of less than 400 m/s., indicative of unsaturated overburden conditions, and – a lower layer, with velocities in the range of 2500 m/s to 3300 m/s., consistent with that of the crystalline bedrock. The data on the depth to and elevation of the upper bedrock surface for each spread was then included in the GEMCOM model as a further aid for assisting the overall assessments of the various areas from the viewpoint of helping identify mined zones, and influence extent.

4.4 Open Excavation Inspections

In areas where previously documented shafts or raises had been recorded to exist, and/or where geophysical or drilling techniques had proved unsuccessful at defining the location of known caps, open cut excavation methods
were employed to expose the cap and the surrounding area, so that its condition could be checked and the exposed bedrock mapped. For the bedrock in the immediate vicinity of each cap, structural information on the nature, orientation and aperture of joints and foliation features was recorded and data was collected on any infill material. Particular note was made of any open features, as these could reflect distress effects from underlying stoping.

Prior to completing the mapping and backfilling of each open excavation, the geometry (size, thickness and configuration) of the cap was surveyed and a photographic record of the cap was made. This information, along with available data on the construction of the cap and on the geometry of the crown pillar, shaft or raise excavation were combined into the database of areas of potential instability.

4.5 Cavity Monitoring Surveys

In order to determine the subsurface geometry of identified open stopes and stopes where fill (usually sand backfill) had subsided to create a void between the crown and the fill, in-stope laser rangefinder surveys were conducted. These surveys, designated Cavity Monitoring Surveys (CMS) were carried out to collect detailed data on the size, shape, and position of underground cavities that were deemed to be sufficiently close to surface or large enough to potentially pose a hazard. The CMS system, which comprises a laser rangefinder within a revolvable mechanical head, can be positioned in a stope using extension rods such that it can be programmed to survey the stope automatically. In this automatic mode, the mechanical head performs concentric circles at defined angle increments (min. of 1°), allowing over 53,000 points to be referenced on the stope boundary. Once all of the data points are collected, the CMS produces a three-dimensional polyline mesh diagram, which can then be downloaded to a PC and converted into the coordinate system of the mine.

In total, about a dozen CMS surveys were undertaken, mostly of mine openings where the sand fill was found to have subsided. In general, the results of the surveys, which allowed both vertical and horizontal sections to be cut through the CMS 3D model geometries of the mine openings, showed good correlation with the GEMCOM model sections of as-mined conditions. In some cases, though, significant differences were identified between the CMS sections and the GEMCOM model sections, suggesting that spalling and degradation had occurred of the stope crown or sidewalls.

Based on the experience gained from conducting the surveys, it was established that no one technique was adequate, when used alone. In fact, it was almost always the case that proper definition of hazard conditions was only feasible if focussed use was made of all techniques appropriate to the geometry being evaluated.

5.0 ASSESSMENT AND ANALYSIS

The investigation techniques discussed above provided the raw information and data necessary to further assess and analyse the mine hazards under consideration. The collected data was then analysed to define potentially unstable crown pillar geometry zones. This was accomplished by essentially employing three parallel analytical approaches – (i) using the empirical scaled span procedure (Golder, 1990; Carter, 1992, 2000; Carter et al, 2002), – (ii) utilizing the CPillar block and beam crown pillar analysis code (Hoek, 1989; Rocscience, 2005) and – (iii) using a hangingwall caving assessment approach (based on the analytical solution developed by Brady and Brown, 2004, and establishing the competence of the stope zone through use of mining open stope design methods (based on Mathews et al, 1980 and Potvin and others, 1989 and 1990). As basic input to these analytical solutions for assessing the stability of the various defined voids, standard rock mass quality assessment procedures were utilized to determine Q, RMR and GSI values for the rock mass conditions of concern (Barton et al, 1974; Bieniawski, 1976, 1989; Hoek, Kaiser and Bawden, 1995).

Data on the geometry of the openings and on the crown pillar geology and rock mass competence were abstracted from the GEMCOM database and from the information collected from the field investigation drilling, geophysics profiling, cavity surveys and downhole videos. Data on rock mass fabric conditions and on the extent of sloughing and degradation of crown and sidewall rock mass zones were also documented, based on information established from the core-logging records, from the mapping of the exposed bedrock around the various caps or derived from detailed processing of borehole camera footage.
5.1 Crown Pillar Analyses

As evidenced by several breakthroughs to surface and hangingwall retrogression problems, various crown pillars that had remained stable for several decades were now obviously becoming susceptible to failure, mainly due to loss of sand support to the crown zone. Crown pillar stability analyses were therefore completed for a number of the sites where the geometry of the crown pillar was such that it could potentially fail in this manner. Where possible, two analytical techniques were used in parallel to assess each crown zone. This was principally done to provide some verification check and overlap in the evaluation approach procedure. The two methods that were most commonly used were (i) the empirical Scaled Span Concept Method and (ii) limit equilibrium analyses utilizing the CPillar program.

5.1.1 Empirical Scaled Span (Cs) Concept Method Assessments


In concept, for any given stope crown and crown pillar geometry, the stability of the surface crown pillar can be considered to be dependent on the rock mass quality of the crown and the abutment zones. The scaled span concept aimed to capture this precedent experience as a basis for empirically relating rock mass quality, defined using the NGI-Q (Barton et al., 1974) or CSIR-RMR (Bieniawski, 1976) or GSI (Hoek et al., 1995) classifications, against a scaled crown pillar span index, Cₛ, calculated as follows:

\[
C_s = S \left\{ \frac{\gamma}{t (1 + S_R)} \right\}^{0.5}
\]

where:
- \( S \) = crown pillar span (m)
- \( \gamma \) = density of the rock mass (t/m³)
- \( t \) = thickness of crown pillar (m)
- \( S_R \) = span ratio = \( S / L \) (crown pillar span / crown pillar strike length)
- \( \text{dip} \) = dip of the orebody or foliation (degrees) (assumed for the Timmins situation to be typically 90 degrees for these analyses, principally due to the fact that almost all of the vein systems comprising the ore zones were sub vertical).

It will be evident from inspection of this equation that all of the parameters are related solely to the geometry (volume and mass) of the crown pillar.

Values of the scaled span for each pillar zone of concern were then compared against the inferred rock quality of the pillar zone, using the case record experience as a guide to whether or not the comparison would be adverse or not. In these assessments, the effects of groundwater and clamping stresses, which also exert some control of stability state, were included within the determination of the rock mass quality (Carter, 1992). Once the rock quality, Q, was known, an estimate of the maximum stable span for the rock mass quality in the pillar zone of concern was made by reference to the critical span (S_c) determined from the precedent database. This was accomplished through application of the best-estimate regression equation to the crown pillar database records, thus allowing the critical span for any rock mass quality to be calculated as follows:

\[
S_c = 3.3xQ^{0.43}x\sinh^{0.0016}(Q)
\]

where:
- \( S_c \) = critical span (m)
- \( Q \) = NGI-Q value = \( \exp (\text{RMR} - 44) \)
- \( \text{RMR} = \text{GSI} = \text{RMR}_{76} \) using the original CSIR Rock Mass Rating parameter definition tables and correlation.
Deterministic Analysis Approach

As an initial assessment of the degree of potential instability, rock quality and basic geometry $S_c/C_s$ information was assessed for each of the crown pillars investigated by drilling. This was then used to define a mean estimate scaled $C_s$ value. Then, based on a best-estimate single value representative rock quality for the inferred controlling weakness characteristics of the crown, a best-estimate critical span, $S_c$ was defined.

For all of the analyzed hazard sites, it was found that the calculated stability points plotted on the stable side of the controlling regression curve, with some closer to the critical stability line than others. Accordingly, in order to assess the degree of instability of each of the various cases, an assessment was made of the divergence of each particular case from the best-estimate critical span line. In order to achieve this, the $S_c/C_s$ ratio (which can be thought of as a crude safety factor) was calculated and tabulated for each crown. Then, as typically, based on the case record database, rock mass quality variability about the mean rock quality for any particular crown zone is of the order of 10 RMR units (Figure 1), an inferred probability of instability $P_f$ based on these single value estimates was roughly estimated from:

$$P_f = 1 - \text{erf} \left( \frac{2.9 \frac{S_c}{C_s} - 1}{4} \right)$$

where \( \text{erf} \) = error function

$$F_c = \frac{S_c}{C_s} \quad \text{(Carter, 2000)}$$

This algebraic approximation approach provided a first stage check, not just on the evaluation process, but it also provided some measure of the likelihood of instability of each of the crowns, that could then be compared to values computed by more rigorous probabilistic simulation methods.

![Scaled Span Graph for establishing stability of Crowns relative to Precedent Experience](from Carter and Miller, 1996)

**Figure 1** - Scaled Span Graph for establishing stability of Crowns relative to Precedent Experience

Probabilistic Analysis Approach

Because of its simplicity and validity due to its being based on real precedent data, application of the Scaled Span method in a deterministic manner was used to rapidly assess likely instability conditions, so that all potentially hazardous zones could be readily defined for follow-up, more detailed assessment. For these initial quick appraisals, single value discrete input parameters were selected that were deemed representative of field conditions. However, in order to better replicate inherent variability in rock mass parameters, more detailed follow-on analyses were conducted using a more rigorous probabilistic approach for the determination of the scaled span and critical span values.

So that the analyses could be carried out probabilistically using raw data for each parameter in the rock mass classification calculations directly taken from the borehole logging results, a specially formulated @Risk spreadsheet was used allowing the statistical distributions for each variable to be sampled using the Latin Hypercube...
sampling method. This analysis method allowed probabilistic examination of each discrete data sample point with reference to its $C_s$ value to estimate its stability with reference to the critical span $S_c$ value for that “sampled” rock quality.

From the probabilistic analyses of 500 data evaluations, a distribution of stability estimates was made allowing a probability value for instability to be computed. This was completed both in terms of Barton’s Q-System as well as using GSI (based on Bieniawski’s RMR$_{76}$ system values), and the results tabulated for comparison against the more simplistic, essentially single value, deterministically based $P_f$ estimates. As this more rigorous approach attempted to better address true variability in rock mass conditions, and hence provided a more representative assessment of possible rock quality, these probabilistically calculated rock mass ratings were then also used as input for the CPillar analysis, discussed in the following section.

5.1.2 CPillar Analyses

The program CPillar (Hoek, 1989) was used for checking the empirical scaled span assessments of the stability state of the various identified hazard zones. This program uses limit equilibrium techniques to compute a factor of safety and probability of failure against several modes of failure, including various cracking modes, including vertical downward sliding of a rectangular, crown pillar block (i.e. plug failure, assuming a rigid block model). Input variables include geometry, rock mass strength, in situ stresses and groundwater conditions (with the option to input standard deviations of controlling parameters, if known).

A number of basic assumptions were made for the CPillar analyses performed to evaluate the stability of the various sites along Highway 101. These assumptions were:

i. that only the rigid block model rather than the voussoir model was deemed representative as almost all of the stopes were narrow vein geometry;

ii. that groundwater levels were assumed to be near surface but below the level of each crown pillar. This assumption was supported by measured water levels determined during drilling. Although at the time of the evaluations water levels were known to be rising, it was believed that final water levels would in general not reach the base of these particular crown pillars;

iii. that low in-situ horizontal stresses prevailed. This was also deemed a conservative assumption as it minimized potential clamping in the analyses. This assumption that low confining stress conditions prevailed for the various crown pillar areas analyzed is supported by field evidence of open jointing seen in the borehole camera surveys. Because the magnitude of any in situ horizontal stresses which might exist and which could also affect crown pillar stability were not known with any degree of certainty, a horizontal to vertical stress ratio ($K$) = 0.3 was used for one set of analyses and a second set were performed with a $K$ value of unity;

iv. that, as a worst case scenario, the dip of the orebody would always be assumed vertical;

v. that no fill was present in the stopes supporting the crown (i.e. all the stopes were assumed open beneath the crown pillar);

vi. that the rock mass fabric in the crown zone at all sites was dominated by “worst case” ore-zone conditions with “three joint sets plus random” being the classification designation utilized for the Q/RMR assessments. This was done despite the fact that not all the drilling results exhibited three joint sets, in part due to orientation bias of the boreholes;

vii. that for comparative purposes, because statistical variability data was often lacking, a standard deviation of 10% of the mean value for most input parameters would be utilized. Selection of a uniform 10% standard deviation spread was checked to be reasonable for the cases where sufficient information existed to make checks, and therefore was used for generation of spread statistics for all other cases where insufficient raw factual data was available to define full statistical distributions; and finally

viii. that GSI/RMR values and associated standard deviations would be defined based on the histogram plots developed from the probabilistic assessments undertaken for the Scaled Span analyses.
In addition to the various assumptions discussed above, it was recognized that there were many uncertainties inherent in the precision of the inferred geometries of the openings, not least of which was making the assumption of accuracy of the available records as most of these crowns are quite old (in some cases 50-100 yrs). Although the geometries for the crown pillars were obtained from the best available contemporary plans and sections, these were often very old. Where possible, they were always updated with data obtained from the drilling, but in some situations this was not feasible. In cases where significant uncertainty existed and/or where checks to the latest available mine plans and sections of the stopes in question seemed to suggest the potential for ravelling or caving, pessimistic geometry arrangements were adopted.

5.2  Stope Sidewall Analyses

With the influence of the rising water levels contributing to movement of sand backfill within the upper level mine workings, surface and drillhole evidence suggested that many stope crowns and sidewalls which had been previously supported by fill, had now become exposed, and thus were susceptible to failure, principally by ravelling, sloughing, wedge release or block falls. This potential for ravelling and sloughing of the upper level workings, and more particularly of the sidewalls, had in turn the potential to progressively lead to increased crown spans and ultimately to the possible destabilization of the overlying crowns.

The most likely failure modes would then be: (i) if the stope geometry was near vertical, by direct crown collapse due to widening of the span, giving rise to plug or ravelling failure of the type examined with the Scaled Span and CPillar checks, or (ii) if the stope geometry was relatively inclined (say at 75° or less) to potential backbreak up along prevailing geological structure on the footwall side preferentially, leading to progressive hangingwall caving development.

For checking the potential for the crown and/or sidewalls of the stopes to slough and destabilize, the Mathews-Potvin Stability Graph method was utilized, with rock mass classification data input for the sidewalls added to the database already prepared for the crown zone (as used for the Scaled Span and CPillar analyses). If adverse sidewalls or crown conditions were identified, the progressive hangingwall cave method of analysis was then used to assess the potential extent of the instability so that remediation solution planning could be efficiently optimized.

5.2.1 Stability Graph Method Evaluations

The stability graph method initially developed by Mathews et al. (1980) and updated by Potvin (1988), Potvin and Milne (1992), and Nickson (1992) is an empirical method for mined excavation stability determination based again on comparison of rock mass quality and stope zone excavation geometry.

In the stability graph method, the stability number, \( N' \), is defined as:

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

where:
- \( Q' \) = the base Q classification value, without the stress or groundwater terms, i.e. \( Q' = \frac{RQD}{Jn} \times \frac{Jr}{Ja} \);
- \( A \) = a rock stress factor (value between 0.1 to 1.0), with lower values corresponding to less confinement;
- \( B \) = a joint orientation adjustment factor (value between 0.2 to 1.0), with lower values representing more adverse orientations; and
- \( C \) = a gravity adjustment factor ((values between 0 and 10), again with lower numbers reflecting less favourable conditions.

In an analogous manner to the Scaled Span concept logic, the Mathews-Potvin Stability Graph Method was initially developed from review of precedent experience of relating rock mass quality to stope geometry for stable and failed open stope situations from a wide variety of underground mines. Over the period from 1981 to date, the method has been improved and refined and now utilizes over 350 case histories to define the various stability zones on the stability graph, which plots values of \( N' \) (the stability number), a basic rock mass quality and stress term, versus Hydraulic Radius (HR), a geometry term describing the stope shape, defined as the stope area/stope perimeter.
For the various crown zone stopes of concern, input data for the stability graph analyses was obtained for each stope geometry from the GEMCOM model, and for the rock mass quality and structural assessments from the borehole and downhole video data. The N' and HR values for the stope sidewalls of each stope of concern were then estimated and plotted on the stability graph as a basis for checking wall condition stability – determined as either – stable, stable with support, supported transition zone or potentially caving.

5.2.2 Progressive Hangingwall Caving Assessment

When stopes that had potentially unstable and/or potentially caving sidewalls were identified through the Stability Graph assessments, hangingwall caving analyses were carried out to check the possible extent of break-back influence, so that surface infrastructure vulnerability could be verified.

With the loss of sand backfill in many of the near-surface stopes, the likelihood of crown failure, due to increased spans was determined to be considerable. The fact that several open cuts also existed, which were supported by sand fill created yet another potential hazard, as sloughage of the sidewalls into these open cut stopes could more easily develop into extensive backbreak which could lead ultimately to progressive backward hangingwall caving, as illustrated on Figure 2.

In order to determine the potential limit extent of such progressive caving, analysis of the potential backbreak for each was carried out using the technique documented by Brady and Brown (2004), based on earlier work by Hoek and by Brown and Ferguson. This analysis revealed that a number of the stopes along the highway would likely be subject to hangingwall backbreak.

For these stopes, particularly those of relatively long strike length, critical failure mechanisms controlling the long-term stability of the steep sided walls of the stope could be identified, either:

- through evidence of **local wall scale progressive degradation** by ravelling, by block slides and/or possibly by local scale toppling, all of which appeared to be kinematically controlled; and/or
- through **deeper, larger scale progressive toppling and caving** of the hangingwall, developing backwards to some limiting break-back line, defined by the geometry of the original stope/open cut wall configuration in combination with controls exerted by drawpoint and sill geometry at the maximum depth of mining in the vicinity.
For each of the Highway 101 stopes that had been identified as potentially unstable to the extent that hangingwall backbreak might be a real issue, analysis cross-sections (typically spaced 50 ft apart) were generated from the GEMCOM model so that variations in the geometry of the stope and sidewalls along strike could be checked.

Assuming the depth of mining \(H\) on each section to be measured down from the top of the bedrock surface to the lowest level of drawpoint development, an envelope of potential backbreak was computed for each stope zone of concern, utilizing the following parameters (ref. Figure 16.19, Brady and Brown, 2004):

- The ratio of depth of caved material to depth of mining \(H_c/H\) was estimated as 1 (i.e. \(H_c=H\));
- The rock mass density \(\gamma\) was estimated as 27 kN/m\(^3\);
- The friction angle \(\phi\) was estimated as 25° and the effective cohesion \(c\) was estimated as 200 kPa (which represents the upper estimate of a poor rock from Brady and Brown, 2004); and
- Based on the above parameters, the back-break \(\psi_b\) angles and failure plane \(\psi_p\) angles were estimated from the angle of break and failure plane angle charts.

Utilizing the back-break and failure plane angles for each section, as per the definition layout on Figure 2 and Figure 16.19 (Brady and Brown, 2004), zones were identified adjacent and along strike to each stope of concern where conditions could be deemed as unstable and partially stable, respectively. Then, irrespective of other remediation considerations, the entirety of each unstable zone was defined as an area requiring fencing at a minimum.

6.0 REMEDIATION APPROACHES

As discussed in the analysis review above, in almost all of the stopes at the time of the investigations, evidence indicated that sand backfill placed within the stopes had shifted. However, the investigation data also revealed that most stopes had not lost all of their supporting fill. The philosophy for remediation, therefore, was to design on the basis that the remainder of the fill might be lost at any time, but that measures would be put in place to (i) monitor the status quo, prior to, during and after remediation measures were under construction and (ii) to design and construct the most cost-effective remediation measures to improve the stability of the crowns and margin zones to the stopes, to mitigate any possibility of failure should the remainder of the fill subside. The typical remediation approach was therefore to ensure that improvements were made to the existing backfill arrangements and that sufficiently comprehensive monitoring was installed so that any adverse problems could be identified and fixed as part of contracted remediation works programs. In addition, on an as-required basis (a) fencing, and (b) capping and/or stope plugs were employed to (i) isolate potentially hazardous areas, and (ii) to improve crown stability.

Details of the geometry, geology and rock mass conditions of the various stope crowns and abutment zones were examined in the light of the following matrix table of feasible approaches for stabilizing the various hazards. Crown and stope geometry and rock mass competence generally governed the remediation works design approach.

<table>
<thead>
<tr>
<th>Case</th>
<th>‘Bridging’ Solutions</th>
<th>‘Hybrid’ Solution</th>
<th>Filling Schemes</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Concrete Capping</td>
<td>Roller Compacted</td>
<td>Cemented Fill/</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Concrete</td>
<td>Non-Shrink</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Gravity Filling</td>
<td>Concrete / Cemented Paste</td>
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<tr>
<td></td>
<td></td>
<td>Pneumatic Filling</td>
<td>Backfill</td>
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<tr>
<td></td>
<td></td>
<td>Hydraulic</td>
<td></td>
</tr>
<tr>
<td>Incompetent Sidewalls</td>
<td>•</td>
<td>•</td>
<td>•</td>
</tr>
<tr>
<td>Inaccessible Openings</td>
<td>•</td>
<td>•</td>
<td>•</td>
</tr>
<tr>
<td>Overlying Infrastructure</td>
<td>•</td>
<td>•</td>
<td>•</td>
</tr>
<tr>
<td>Flooded Openings</td>
<td>•</td>
<td>•</td>
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</tr>
</tbody>
</table>

Table 3 - Matrix for Assessing Optimum Remedial Options for Stabilizing Identified Hazards

7.0 MONITORING METHODOLOGY

It was recognized that potential risks existed that required time to address, hence interim measures were required to ensure public protection from exposure to these risks. As this required gaining some real in-depth understanding of
these risks, a process of risk assessment, prioritization and mitigation ensued. Time and budgetary constraints were a reality, so engineering judgement was needed to prioritize remediation solutions and their implementation. Inevitably, some risk areas were identified that presented potential hazards, but could not be immediately remediated, through lack of adequate funding or timing resources, and therefore for these locations a comprehensive monitoring program had to be implemented to ensure public safety. In implementing such monitoring, it was recognized that it constituted an interim measure that was not to be continued in perpetuity. It was, however, successful as a stop-gap measure, allowing time for appropriate remediation funding and contracting arrangements to be put in place, so that proper remediation measures could be completed.

As an interim control to safely execute the works needed to remediate the various identified hazard zones, and in order to also help define the extent of possible influence of the subsiding sandfill on stope sidewall and crown stability, monitoring was undertaken of all zones considered to be of concern. Various methods of monitoring were employed ranging from visual, and surveyed surface points, to various inhole methods – inclinometers, time domain reflectometry (TDR) cables and multipoint borehole extensometers (MPBX). In order to aid in prioritization of the remediation works various thresholds of movement were defined in three categories: minor, significant and major. Each category was given specific thresholds based on the monitoring techniques being used and a recommended action plan for each category was developed. This action plan involved various levels of increased vigilance, including increasing the frequency of monitoring until baseline stability was re-established or re-scoping could be implemented to either reset the prioritization for remediation or accelerate the implementation of remedial works at the most adverse movement site, so that this took priority over other areas which were not deemed to be showing as much distress. Table 4 shows an example of the monitoring criteria and recommended action plans. Figure 3 summarizes the logic steps for conducting the monitoring programme, and making decisions regarding remediation priorities.

<table>
<thead>
<tr>
<th>Category of Ground Movement</th>
<th>Criteria</th>
<th>Recommended Action Plan</th>
</tr>
</thead>
<tbody>
<tr>
<td>Major</td>
<td>- Visual assessment of disturbance to surface in the vicinity of the stoping areas comprising of greater than 2 inches of depression, lateral movement or crack dilation, or - Subsidence of stope fill as measured through the fill point comprising of greater than 10 ft of depression, or - Visual assessment of disturbance to ground within fenced off area comprising of subsidence of more than 10 ft of backfill or 2 ft of overburden, or - Seaving of MPBX or TDR cable, or - Displacement of greater than 2 inches registered by MPBX at at least two anchor locations.</td>
<td>- Restrict access to affected area until detailed assessment of monitoring results has been carried out, and - Increase monitoring frequency of all instruments to twice daily - Backfill subsided areas</td>
</tr>
<tr>
<td>Significant</td>
<td>- Visual assessment of disturbance to surface in the vicinity of the stoping areas comprising of between 0.5 and 2 inches of depression, lateral movement or crack dilation, or - Subsidence of stope fill as measured through the fill point comprising of between 2 and 10 ft of depression, or - Visual assessment of disturbance to ground within fenced off area comprising of subsidence of between 5 and 10 ft of backfill or 0.5 to 2 ft of overburden, or - Increase in TDR cable signal of greater than 40 mho, or - Displacement of between 0.5 and 2 inches registered by MPBX at at least two anchor locations, or - An accelerating rate of movement registered by MPBX, inclinometer, TDR or Visual monitoring on review of baseline data.</td>
<td>- Increase monitoring frequency of all instruments to daily - Backfill subsided areas (fill point filling should be done when at least 1 truck load can be accepted)</td>
</tr>
<tr>
<td>Minor</td>
<td>- Visual assessment of disturbance to surface in the vicinity of the stoping areas comprising of less than 0.5 inches of depression, lateral movement or crack dilation, or - Subsidence of stope fill as measured through the fill point comprising of less than 2 ft of depression, or - Visual assessment of disturbance to ground within fenced off area comprising of subsidence of less than 5 ft of backfill or 0.5 ft of overburden, or - Increase in TDR cable signal of more than reading fluctuations (20 mho) but less than 40 mho, or - Displacement of more than reading fluctuations (4/100th of an inch) but less than 0.5 inches registered by MPBX at at least two anchor locations.</td>
<td>- Return to a minimum monthly monitoring frequency until baseline stability re-established</td>
</tr>
</tbody>
</table>

*: Backfill refers to the sand fill currently being used to top up the subsidence zones.

* for example: Period 1 change in readings = 0.5 inch
Period 2 change in readings = 2.0 inches
change in rate = (2.0 - 0.5) / 2 (bi-weekly) = 0.75 inch/wk wk
If this rate is greater than 1/4 the lower bound criteria for significant movement (0.5 inch/wk x 0.25 inch), it can be considered to be a significant event

Table 4 – Monitoring Criteria
As outlined on Figure 3, the philosophy was to address the highest risk sites first so only nominal interim monitoring would be needed to bridge the time gap for engineering, tendering and start-up of remediation. The scale of monitoring was the most comprehensive for the medium risk sites where more time was required to develop a remediation solution. These were the most intensely instrumented, as monitoring would be the only implemented measure until such time as available budget existed to fix such sites. Minor monitoring was used on the low risk sites, while the highest risk sites, were only monitored comprehensively where it was found not feasible (due to budget or timing constraints) for remediation solutions to be implemented immediately.

The objective of the monitoring was to measure potential movement of the fills within the stope, to determine sloughing or ravelling of the sidewalls and/or to detect any movement of overburden soils near the stopes. The objective of the monitoring approach was to establish a baseline level of stability. To accomplish this baseline, monthly readings were taken for the first year. Readings were then made less frequently once a full calendar year of fill and sidewall monitoring had been carried out without any significant movement. This established a stable baseline incorporating seasonal effects. A reduced reading frequency was set at three times per year; once before the spring thaw (typically in March), once immediately after spring thaw (May-June), and once in the fall (September).

7.1 **Inclinometers**

Inclinometers were targeted for areas where hangingwall toppling would be an issue and extensometer geometries were too difficult to position. As typical soil inclinometers are generally not of sufficient precision for picking up small movements (and thus are not typically used for rock mechanics installations), the positioning of the inclinometers was carefully coordinated with the zones where toppling instability mechanisms were deemed most likely. In such situations, inclinometers, if well positioned, can be very effective for monitoring overburden response over destabilizing hangingwall and crown zones. Sets of inclinometers were therefore installed at two sites along Highway 101 and First Avenue. These locations had relatively deep overburden (30-60 ft) where it was considered that such instruments would give the best early indications of rock instability through the movement of the overlying soils. Slotted inclinometer casings were typically installed in near-vertical boreholes through a zone of suspected movement, with the bottom of the casing set in rock, which might be subjected to toppling into the stope, and with routine survey pickup undertaken of the head as the control reference point. Readings were made using the inclinometer downhole probe and control cable and offset plots were prepared to check rates, depth, and magnitudes of displacements detected in the holes.
Multiple Point Borehole Extensometers (MPBXs) were employed at several of the Highway 101 sites in order to monitor ground movement around the stope zones. Six point monitoring arrays were typically utilized, with readout sensors (linear potentiometers) mounted in the head of the MPBX’s, allowing remote reading of the anchors set at various strategically defined locations. The instruments were set to all have 127 mm (5 in) range with a manufacturer guaranteed accuracy of ±2% of this full-scale range. Actually, none of the units recorded this magnitude of movement prior to being remediated.

Well located MPBXs are arguably the best means for detecting small movements and inferring movement rates to establish critical times to failure. Therefore, significant effort was expended in attempting to position them in the most critical zones of the crown pillars. In many of the presumed caving hangingwall stopes where crown span enlargement due to hangingwall ravelling could lead to destabilization of the crowns, interblock movements are typically very complicated (involving shear, tensional failure and also block rotation). MPBXs were therefore positioned based on best estimate understanding of the rock failure mechanisms with the hope of detecting early movement trends. More than a dozen MPBXs were installed in the sites along Highway 101.

Time Domain Reflectometry

Because MPBXs only monitor the differential movement between relatively widely spaced anchors (relative to block size), it is difficult with extensometers alone to precisely define absolute movement locations with respect to individual geologic structures or tension cracks. Used in combination with MPBXs, Time Domain Reflectometry (TDR) cables can allow definition of critical movement zones. The method uses a TDR cable tester to send an electrical voltage pulse along a coaxial cable installed in a borehole in order to determine the locations of any defects in the cable. In most instances, TDR cables were installed alongside an MPBX installation in the same borehole so that any defect within the cable, which would cause the pulse to be reflected and detected by the tester unit, could be correlated with absolute movement data from the extensometer. By analysing the time difference between the initial pulse and the arrival time of the reflected pulse and also examining the shape of the TDR signature, not only was the distance downhole to any detected defect determined, but some estimate was also feasible, with sufficient movement, of any defect’s probable mode of origin, viz., shear, or tensile opening. Precision of the locations of such defects was achieved by ensuring that the TDR cable was pre-crimped at known (either 1.5 or 3 m) intervals (depending on monitoring geometry). With such reference points positioned along the cable, any movement in the borehole detected by the TDRs could be interpreted to help infer precise locations and structural configuration of critical controlling movement zones. By installing the TDR cables in combination with the MPBX installations, the two instruments allowed an increased understanding of behaviour modes that were often very helpful for developing final design of remediation solutions.

Piezometers

One of the key mechanisms responsible for initiating some of the original instabilities was the rise in groundwater levels within the old workings. As a result, piezometric monitoring of changes in groundwater levels was deemed of significant importance for most of the hazard sites. As it was conceivable that, due to further groundwater level changes, additional fill movement and possibly further destabilization of previously sand filled crown stopes might occur, additional “open” piezometers were installed specifically to monitor the rising water levels in the stope system. Monitoring of water level changes in the rock mass and within the sand fills was also deemed critical. Two types of installation were used – simple standpipes, which required manual measurement using a water level finder, and Vibrating Wire installations – which could be read remotely, and thus were the preferred choice for the zones of perceived greatest risk. Monitoring of pressure heads in the sandfill was also done with VW installations specifically in order to predict whether stable sand conditions were being maintained while remediation measures were underway. It was particularly important to verify that stable sand conditions prevailed for the duration that the sand needed to act as a stope plug formwork. Piezometers were also used to provide a long-term check on the rate of change of the rising groundwater in the stope zones.
## REMEDIATION MEASURES

Three basic remediation procedures were utilized for remediating the hazards along Highway 101 – (i) conventional concrete capping, (ii) cemented plug backfilling, and (iii) straightforward top-up filling (either as a cemented or non-cemented fill). Fencing was also employed, but mainly as a temporary measure until full remediation could be implemented. There was reluctance to use fencing as the sole means for public protection where infrastructure might subsequently encroach close to a perceived hazard. It was deemed that fencing would only be considered acceptable in these situations, if such fencing were accompanied by appropriate monitoring and with filling provisions in the event of further subsidence being identified. Fencing was also deemed unsuitable for cordonning off hazards that might exist under roads or under infrastructure, but was acceptable for cordonning off hazards adjacent to, but at some distance back from the highway. In Timmins, fencing was therefore used primarily to enclose the central mining area but was also used to enclose outlying features.

As part of initial hazard delineation, historic fence lines were inspected and, where necessary, upgraded. In this process, it was found that in some cases the fence lines had been poorly placed, or hazard zones had expanded since original placement. In consequence, many fence line layouts were revised, mainly based on perceived stope geometry from the GEMCOM assessments, or because of overburden depth, and/or perceived problematic positioning with respect to the enlarged area potentially subject to progressive hangingwall backbreak. This required fencing at many sites to be relocated to achieve more suitable alignments.

### 8.1 Conventional Concrete Capping

In several of the hazard sites, existing concrete caps were found, that in some instances had deteriorated to the extent that they were structurally unsound. In other cases, they were found to be of inadequate areal coverage, and thus needed extending. While the construction of large reinforced concrete capping structures above these existing stope caps and/or the reinforcement of the existing underlying crown pillars would normally be considered the most frequently implemented technique for stabilization of near-surface open stopes, capping was only deemed feasible in one instance for a small raise located about 30 ft from Highway 101. The construction of large reinforced concrete capping structures above the other stope caps and crown pillars at most other sites was considered impractical, due to the proximity of sensitive services (sewer and gas) and potential interference with the highway. Experience with large caps at other locations in Timmins, however, had showed that adequate caps could be constructed if the appropriate geometry of the bedrock allowed, but it had also highlighted the need for rigorous geotechnical review of founding conditions, ensuring fill points were positioned as accurately as possible. It was also deemed essential that at least temporary monitoring be instituted of: (i) the area where raise capping was completed, and (ii) of all abutment zones around existing caps deemed sufficiently unsound that they needed replacement.

### 8.2 Stope Plugs

Stope plugs were constructed in four stopes along the Highway 101 corridor. These plugs had a thickness of approximately twice the stope span that they infilled. The plugs that were constructed consisted of a relatively low strength concrete (i.e. 5-10 MPa) that was designed to be self-supporting when placed within the stopes. This design approach was based on a methodology originally developed for artificial sill pillars (sill mats) and for mining below backfilled stopes. Figure 4 shows a typical fill plug design for the rehabilitation of stope caps that were deemed inadequate in terms of the Ontario Mining Act (Mine Development and Closure) under Part VII of the Act, O.Reg. 240/00 (hereafter referred to as the Code).
In terms of the suitability of using un-reinforced cemented fill plugs, as a certified closure remedy (in accordance with the Code), it was considered appropriate to approach the certification of these cemented plugs as a hybrid structure since the plug would act as:

- a self supporting capping structure;
- as a backfill remedy, since sandfill is typically introduced before casting the cemented plug (although plug stability is designed to be independent of the sandfill level); and
- as an ‘artificial’ crown pillar by filling the upper portion of the stope with a cemented fill plug providing active support to the stope sidewalls.

Two approaches were applied for the design of the cemented fill plugs, namely (i) limit equilibrium solutions, and (ii) numerical analysis. As a first pass, limit equilibrium solutions were used to evaluate a series of specific stope sections obtained from the mining plans. These sections were spaced at appropriate points along the zone to be plugged, in order to define critical geometry extents for the plug solutions. These assessments were checked at one or two locations by means of more rigorous numerical analysis (using the FLAC program) utilizing actual stope sections as collected with the Optec CMS system. In cases where the stope walls diverged with depth (i.e. the cast plug would form an inverted key block), it was found that the limit equilibrium methods were not applicable. For these zones, the designs were found to be very sensitive to stope geometry and thus extensive numerical analyses were conducted to establish appropriate plug dimensions. In the most difficult geometrical conditions, it was also found that efficiency of the plug design geometry was critically inter-related with stope geometry and with whatever was assumed for the plug-stope sidewall strength properties. In some cases, in fact, it was found that reinforcement of the plug-stope sidewall interface would need to be considered in order to achieve sufficient shear restraint for the minimum concrete volume required to achieve practical placement.

As illustrated on Figure 4, the typical arrangement was that the plugs were ‘cast’ above the sand backfill which previously existed in the stopes. This, in all of the stopes to be plugged, was found to have not subsided beyond the point that it could perform a useful purpose of constituting a base form to provide support to the stope plug during the pouring and curing period, before the plug gained adequate strength to be self supporting. This in turn provided
a stable foundation for the overlying cap. The strength requirements of this plug were evaluated for conditions where the plug would be required to be self-supporting in the event of further sand fill subsidence. The design approach, which is very similar to cemented backfill underhand mining applications, did not include any consideration for surcharge loads. Five possible failure modes of the proposed plug were checked during the plug design stage. These encompassed: (i) arching, (ii) flexural or bending failure, (iii) block sliding due to side shear failure, (iv) rotational failure at the hangingwall contact, and (v) sloughing destabilization of the wall rock margins. The stability methods developed by Mitchell and Roetger (1989), by Mitchell (1991) and most recently by Newman et al (2001), for evaluating the stability of cemented fill backs in underhand cut-and fill mining, as calibrated by laboratory testing, were extended to application for these stope plugs.

One issue with all of the stope plugs that proved a complication more for the practical construction than for the design was defining the geometry and stability of the sand fill that was needed to create the bottom formwork for the plugs. Another complication issue, which was more of a design optimization problem, was that one of the plug designs had an inverted keyblock geometry that effectively created two hangingwalls, with limited confinement. This created two problems: a potential shear problem along the excavation boundary of the stope at the interface with the stope plug, and the potential for insufficient arch confinement within the plug across the stope. This need for arching confinement required that the plug be thick enough to allow arching and also of sufficient strength. This resulted in a balance between plug geometry (i.e. thickness to span) and strength (i.e. cement content) in the plug design. To address these potential issues, extensive laboratory testing was undertaken and a comprehensive numerical simulation model was formulated to analyse the plug geometry. Based on this analysis, the stope plug design was deemed suitable for certification in terms of O.Reg. 240/00, provided that key physical properties of the stope plug were tested to be within the required range of strengths. Plate 3 shows placement operations in progress for one of the plugs.

8.3 Stope Filling

In addition to the concerns raised with respect to unsound concrete caps, and their solution by cemented plug placement, there was also some concern about various open stopes with relatively thin crown pillars that lay beneath Highway 101 that might destabilize if fill support was lost to the stope sidewalls. Of particular concern was the issue of the long-term stability of the hangingwalls of the stopes. Although the crown pillar analyses had suggested that the crowns of the stopes under Highway 101, where this might be an issue, were stable, in order to provide additional certainty and assuage public concerns, tight backfilling was recommended.

As Highway 101 is the main thoroughfare through Timmins, maintaining traffic flow while the filling operations were underway was deemed of paramount importance to public approval. A method of filling the stopes had to be devised that would minimize traffic disruption and yet still provide a reasonably efficient means for accessing the work sites and reintroducing fill into the underground workings. In order to deal with overburden depths of over 50 feet, a dual rotation drill (Foremost DR24 – ref. Plate 4) was selected for the purpose of drilling and casing the 16-inch fill holes needed for the project.
Although it was recognized that there was merit in placing a cemented fill in the stopes, the backfill volumes were considered likely to be high, and thus cost prohibitive. With these stopes not being considered of high risk and therefore an immediate stability concern, the decision was made to backfill them with uncremented fill on the proviso that measures would be put in place for backfill levels to be checked and topped up in the future if necessary. It was therefore stipulated that stainless steel casing (with stainless steel cross bars to provide fall protection due to their large diameter) be specified for the fill holes, principally because of stainless steel’s longevity. A flush mounted manhole cover was also installed as a permanent feature in the roadway in order to provide a means for accessing the fill holes (a) for future inspections, and (b) for refilling and top-up of the fill in the stopes (as necessary).

After drilling breakthrough, each of the stopes was surveyed by CMS so that fill volumes could be reasonably assessed. Plate 5 shows filling underway through one of the 16 inch fill holes. As evidenced in the photograph, fairly rapid filling was obtained when the fill holes broke through above the water table. However, it was found that filling rates were cut by tenfold when the fill hole breakthroughs were submerged.
9.0 CONCLUSIONS

The integrated mutual understanding developed between corporate Mine Management and Government allowed synergetic solutions to be implemented in a timely fashion. The Agreement between the Ontario Government and Porcupine Joint Venture (Kinross and Placer Dome CLA) has shown that Government and Industry partnerships can produce excellent results. Mining properties that could have become orphaned are productive once more, public safety has been enhanced and jobs have been created.

The procedures developed through this mutual Industry-Government cooperation has meant that hazards within an urban setting can be successfully managed using a risk assessment approach. The program, however, also highlighted a clear need for a rigorous process of evaluation and design optimization to be implemented to ensure adequate planning is done for future development. It was early on identified that a long-term view needs to be established throughout all of the remedial planning. This need for forward vision has been a common thread through the implementation of the remedial measures, and is ongoing still as the City of Timmins is currently revising its official plan in order to implement procedures to better integrate historical mine features into its planning process.

10.0 REFERENCES


“Testing Requirements for Certification of a Non-Standard, Alternative Rehabilitative Measure for Stabilising Open Near-Surface Stopes”, (May 2002b)