Caving 2026 – A van As, D Cumming-Potvin & J Wesseloo (eds)

doi:10.36487/ACG_repo/2645_68

Design, construction and initial operation of a water re-use plug at New Afton mine

Andy Thomas [a,*], William Joughin [a], Heba Khalil [a], Sam Owre [b], Yoko Yanagimura [b]

[a] *SRK Consulting, Canada*

[b] *Coeur Mining, Inc, Canada*

Abstract

Mining operations often require sealing of underground openings for range of reasons. At New Afton mine, a plug was conceptualised to impound water for the purpose of underground water re-use and to route water to existing water management infrastructure. This paper describes the advancement of this concept through to design, construction and initial operation of a parallel-type concrete plug. Defining the operating environmental and loading conditions of the plug was particularly challenging due to the uncertainties in the conditions of the overlying muckpile. This was approached by establishing a range of scenarios and checking the designed plug for energy absorption capacity relative to possible resultant inrushes. Plug design was advanced using a novel approach for concrete mass pour, incorporating filler holes from an overlying drift, to fill the core between front and back mesh-reinforced fibrecrete form walls. To control the risk of thermal cracking and for sulphate resistance, a project-specific concrete mix was developed which had high flowability to ensure complete and tight filling. Post-construction testing included a full reservoir fill, during which the plug maintained its integrity but exhibited seepage above target rates. A pressure grouting program was conducted which reduced the rate significantly. This case study provides useful information for operations in a similar situation, along with key learnings and recommendations from the process.

Keywords: *underground mine plug, plug design, plug construction, inrush, water-retaining structure*

1 Introduction

New Afton mine has recently embarked on several projects aimed at reducing reliance on external mine water sources. One such project involved the development of an underground water re-use system to reduce freshwater demand in the underground mine. To achieve this, New Gold Inc. (which was acquired by Coeur Mining, Inc. in March 2026) proposed constructing a water-retaining plug in late 2023 to form an underground water reservoir. The reservoir was designed to capture and store existing mine reclaim water which originates from the Afton Pit Tailing Storage Facility (APTSF). By utilizing this water source, it was estimated that the operation could effectively re-use 350 million litres per year. In addition to reducing freshwater demand, the project was anticipated to lower New Afton's energy consumption by reducing pumping requirements.

SRK was engaged to support the project with engineering services which were carried out in two project phases. The projects included assessment of drainage scenarios and sealing options to achieve the objectives. A plug was selected as the approach to create the underground water reservoir and route spill water to existing water management infrastructure. Also evaluated were possible current and future overlying muckpile conditions for the potential for inrush of mud or water. Given the inherent uncertainty in conditions of the historical muckpile this was challenging. This paper provides information on the process of evaluation of event scenarios leading to selection of the design event, operating environmental conditions and plug loads. It also describes the unique methodology that was selected for the plug construction to eliminate

* Corresponding author. Email address: athomas@srk.com

concrete cold joints. The aim of the paper is to present an interesting case study taking the reader through the framing of the problem, plug design, construction and operations aspects.

2 Context

New Afton mine is a caving operation with multiple cave lifts. It is located approximately 10 km from Kamloops and 350 km northeast of Vancouver in British Columbia, Canada. The haulage recovery level (HRL) was developed to extract previously unrecovered ore from Lift 1 and is located beneath the southwestern portion of the east cave footprint. It comprises 2 sublevel caving (SLC) levels with crosscuts oriented north–south. Figure 1 illustrates the spatial relationship of the HRL to adjacent cave lifts and identifies development drifts relevant to this case study.

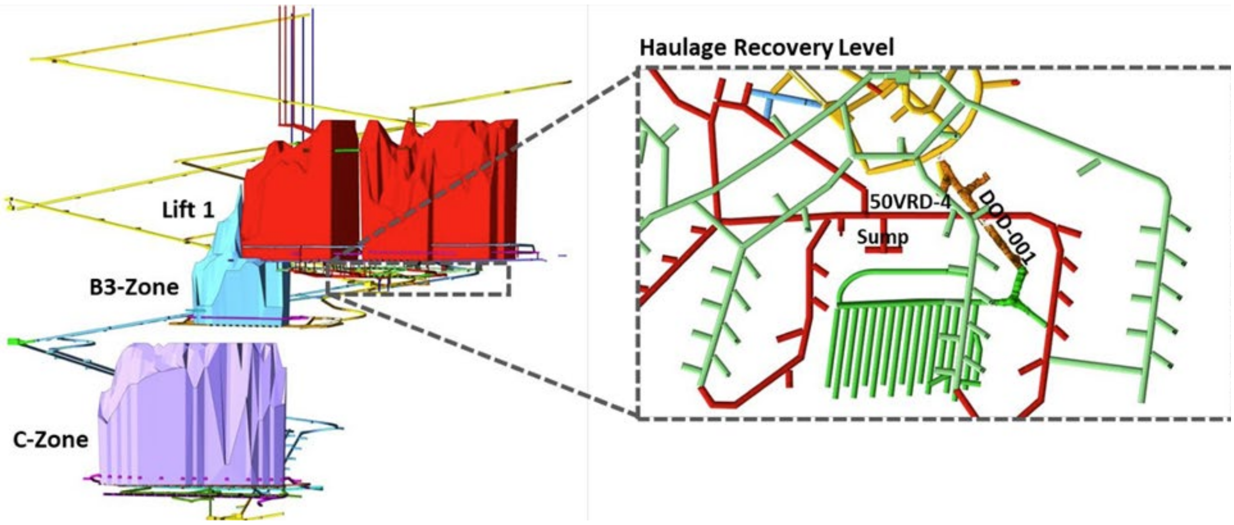


Figure 1 Location of the haulage recovery level and relevant development drifts

After completion of mining from the HRL, to facilitate drainage and reduce hydraulic head on the slurry pump in MSD-402, a 10-inch (254 mm) drainhole (DH-01) was drilled from 50VRD-4 into MSD-402 (see Figure 2). A mud-push event from MRC-25 then formed a ‘mud pile’ which buried the slurry pump. To enhance drainage of the level and lower the ponded water elevation, a 10-inch (254 mm) drainhole (DH-02) was drilled from DOD-001 to MRC-27 near its intersection with MSD-402. Initial discharge through DH-02 into DOD-001 flowed for several days, temporarily flooding DOD-001, followed by a sustained discharge rate of approximately 5 m^3/h.

The concept was that by flooding the 5030 haulage level, the ponded water would spill into and down 50-VRD-4 to the existing permanent sump, connecting into existing water management infrastructure. Spill would occur either via DH-01 or at the intersection of MSD-402 and 50-VRD-4 (RL 5040). The approaches considered were by sealing:

1. DH-02 and MD-OP-01
2. DH-02 and DOD-001 southeast of DH-02 collar
3. DOD-001 northwest of DH-02 collar.

In consultation with the mine site, option 3 via a plug was selected, in part because the other options would be more challenging with the limited access to the HRL.

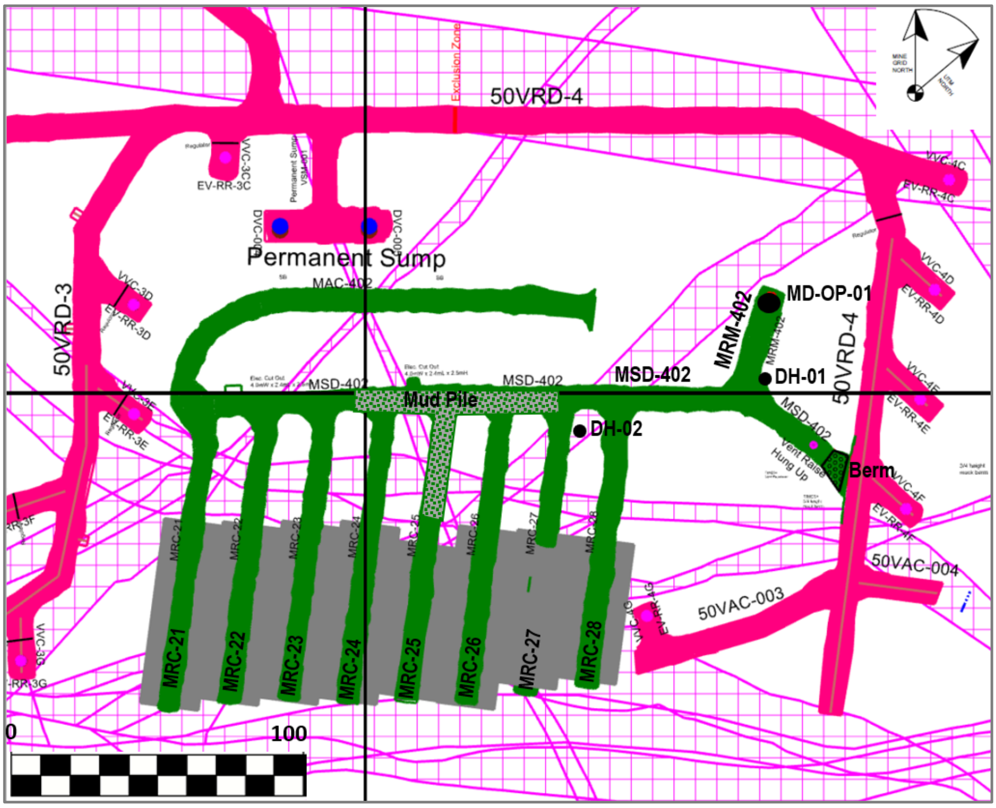


Figure 2 Plan view showing 5030 haulage level (green) with connected development and modelled faults (pink hatched)

3 Design methodology

This section presents the methodology of plug siting, inrush scenario assessment, and plug design.

3.1 Siting

To support initial siting of the plug in DOD-001 lithological and structural models were reviewed to evaluate ground conditions. The drift is within crystalline and fragmental volcanics (BXF) and diorite (DI) units. The drift interacts with modelled Fault-R at the southeastern extent and Fault-I20_B toward the northwest. Based on the New Afton mine ground control management plan, the BXF and DI units for the HRL are generally of 'Fair' to 'good' rock mass quality (Bieniawski 1989), and fault zones exhibit reduced quality, typically ranging from 'poor' to 'fair'. Intact rock strength data from laboratory testing and core logging indicated that both lithological units are predominantly classified as 'medium strong' to 'strong' in accordance with ISRM (1981). These data suggested that the rock mass intersected by DOD-001 was generally competent, with localised reductions in quality associated with fault zones. Based on these findings, the team went forward with initial siting of the plug within the DI unit between Fault-I20_B and Fault-R, where ground conditions were expected to be most favourable.

For localised assessment, seepage data and historical development face mapping data for DOD-001, including intact rock strength estimates and rock mass classifications were reviewed. These data were broadly consistent with the initial review, confirming no current seepage inflows and that most of the drive is characterised by 'fair' to 'good' rock mass conditions, with lower quality conditions in faulted zones. This was supplemented with a site visit for inspection and confirmation. Final siting placed the centre of the plug approximately 30 m from the southeastern end of DOD-001 (Figure 3).

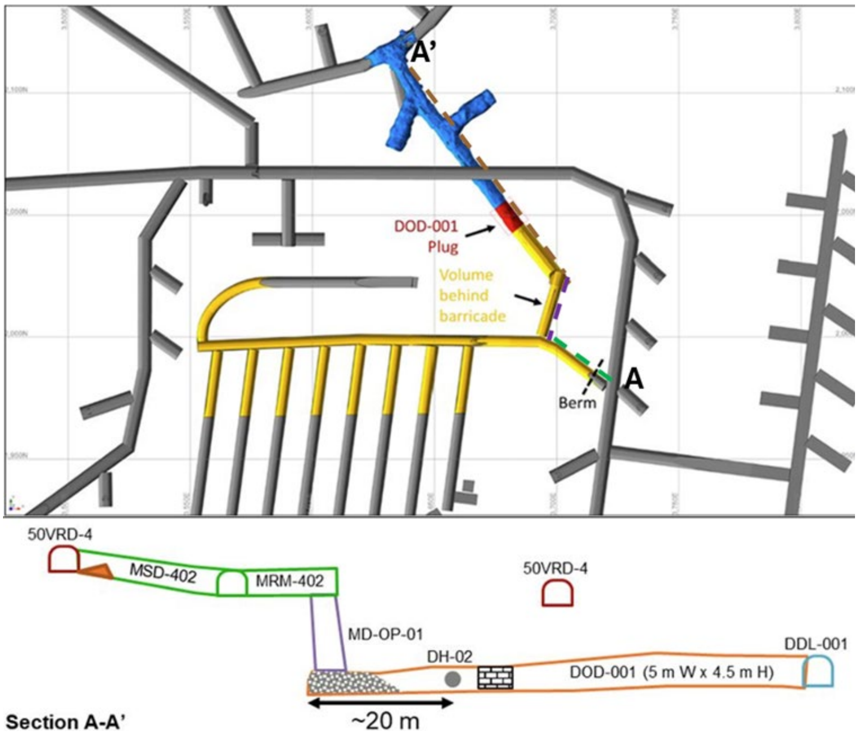


Figure 3 Plan view (top) showing design development shapes and the plug location which illustrates the resulting underground reservoir ponding extents (in yellow), assuming hydraulic connectivity through the mud pile in MSD-402. Section A-A' (bottom - not to scale) shows the infrastructure in the vicinity of the DOD-001 plug location

3.2 Scenario assessment

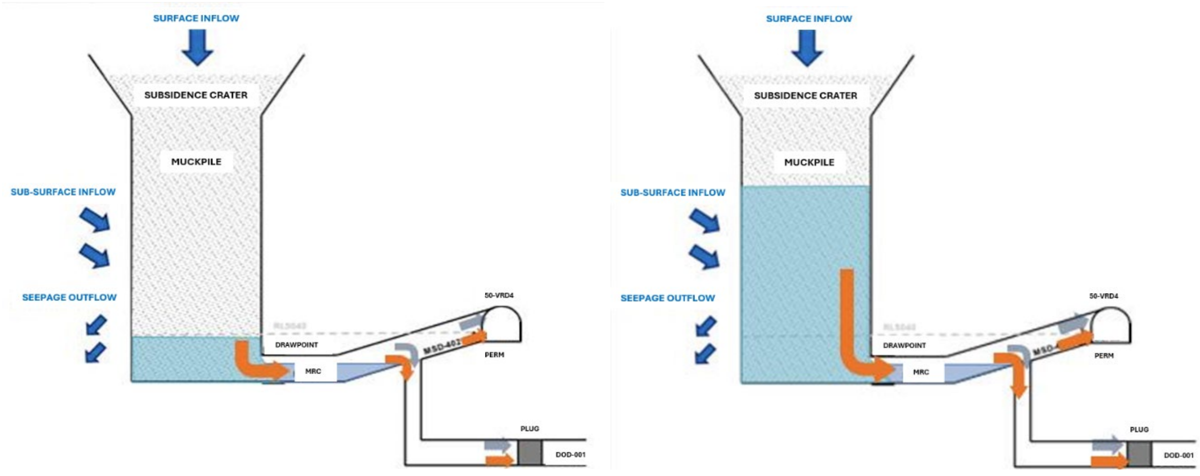
The current and future geotechnical and hydraulic condition of the caved muckpile above the HRL are largely unknown. A range of potential conditions were considered as part of an inrush scenario assessment to investigate the plug operating environmental conditions. The Phase 1 project evaluated potential events that could result in rapid, uncontrolled inrush of mud or water from the MRC drifts. It found that that inrush events were unlikely due to the absence of triggering mechanisms, e.g. mucking. However, static liquefaction was deemed theoretically possible if pore pressures in the muck became high enough to overcome shear strength.

The Phase 2 project expanded the assessment to consider a broader range of scenarios reflecting uncertainties in the muckpile properties, hydraulic conditions, and reservoir levels. Since the Phase 1 project, there were actual and planned changes to the site environment which needed to be considered. Changes included backfilling of the overlying New Afton pit with thickened and amended tailings, and the planned development of an exploration drive taking off from DOD-001 (DED-101) northwest of the plug.

Because the reservoir will be used for water supply, the pond level will fluctuate, giving rise to a range of possible scenarios of muckpile and inrush receiving environment hydraulic conditions. As shown in Figure 4, the highest risk for inrush was found to be the scenarios with:

- low muckpile transmissivity
- an elevated muckpile phreatic surface (relative to the MRC drift elevations)
- muckpile inflow exceeding outflow
- the reservoir fully drawn down.

Under such conditions, there would be potential for rapid inflow of saturated material, with flow directed toward 50VRD-4 and, to a lesser extent, toward the DOD-001 (more convoluted path). Consequently, impact loading was considered possible and incorporated into the plug design process.



(a) (b)

Figure 4 Simplified schematics of the highest-risk scenarios. (a) The muckpile is saturated to RL5040 and the reservoir is drawn down; (b) The muckpile is saturated above RL5040 and the reservoir is drawn down

3.3 Design assessments

With reference to Lang (1999), the most common types of water-retaining structures in underground mines are configured parallel, tapered, or hollow core. Typically, tapered plugs are employed to increase shear strength and reduce concrete volume and hollow core plugs are used in large cross-sectional dimension tunnels to facilitate contact grouting along the plug length. In this case, a mass concrete (unreinforced) parallel type was selected.

Design methodologies, equations and acceptance criteria were informed by Lang (1999) and the UK Health and Safety Executive (2005) which provide established guidance for the design, construction, and operation of underground water-retaining plugs. Design guidance indicates that plugs must be evaluated against key failure mechanisms of hydraulic jacking, shear failure along the tunnel interface, and failure by deep beam flexure. Hydraulic jacking, which occurs when hydraulic head exceeds the overburden pressure, was deemed implausible at the planned plug setting. Accordingly, the design assessments focused on deep beam flexure and shear failure mechanisms. Design was undertaken by calculating Factor of Safety (FoS) values comparing driving and resisting forces. For detail on the equations for calculating driving forces, the reader is referred to Lang (1999).

3.3.1 Driving forces

Due to the uncertainties associated with DH-02, the maximum static hydraulic head was selected to be dictated by the elevation of the berm at the intersection of MSD-402 and 50VRD-4. This condition corresponds to a maximum hydraulic head of approximately 28 m (neglecting pressure loss due to lateral travel) at the plug location.

The design of hydraulic seals in tunnels exposed to seismic ground motions needs to account for hydrodynamic pressure due to water hammer. For detail on the approach and equations, the reader is referred to Westergaard (1931). Mine site seismic data and an earthquake event database of over 30,000 records of accelerations and velocities were reviewed to establish site-specific seismic hazard model parameters. Peak ground accelerations were defined for a range of annual exceedance probabilities (AEP) from the model. For the design, an AEP of 1:10,000 years, a peak ground velocity value of 0.06 m/s was selected.

3.3.2 Resisting forces

3.3.2.1 Deep beam flexure

The plug was analysed as a deep beam, and flexural resistance was analysed using yield line theory as shown in Figure 5. The formula for load capacity (w_c) was derived for a rectangular concrete slab using the work method described in Kennedy & Goodchild (2003):

$$w_c = m_c \times \frac{48\,(h+w)}{3hw^2 - w^3} \tag{1}$$

where:

- m_c = moment capacity
- h = plug height (m)
- w = plug width (m).

$$m_c = 0.15 \times f_{tu} \times b \times l^2 \tag{2}$$

where:

- f_{tu} = concrete tensile strength (MPa)
- b = unit width (m)
- l = plug length (m).

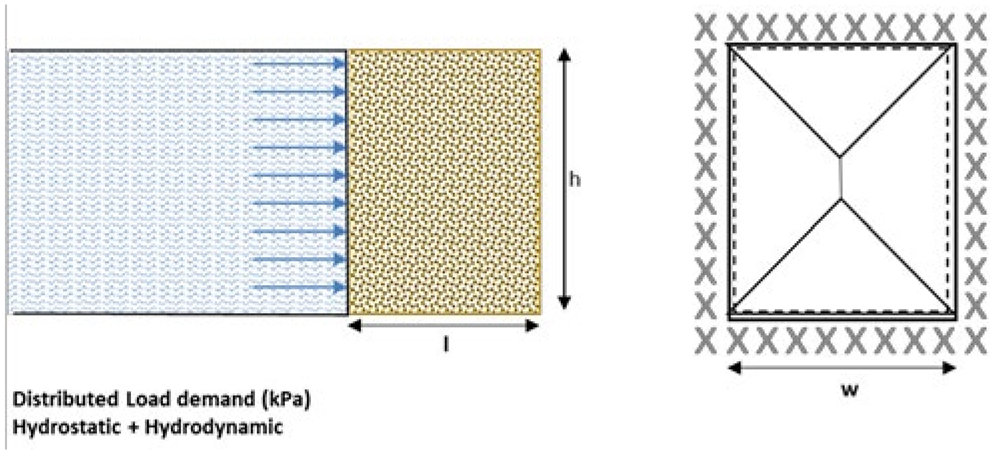


Figure 5 Schematic views of a plug showing dimensions and loading conditions

3.3.2.2 Shear failure

Punch shear resistance along the tunnel interface was calculated as follows:

$$R_{punch\ shear} = 2\,(h + w) \times l \times U \tag{3}$$

where:

- h = tunnel height (m)
- w = tunnel width (m)
- U = shear strength (kPa).

Note that conservative tunnel dimensions and shear strength (600 kPa) representative of 'fair' quality rock mass were adopted.

3.3.3 Impact loading and energy absorption

Impact loading associated with a sudden mudrush was analysed by considering the conservation of energy. The kinetic energy (E) of a mudrush is a function of material mass and velocity:

$$E = \frac{1}{2}mv^2 \tag{4}$$

where:

m = mass of the mudrush (kg) (slurry density x volume)

v = velocity of the mudrush (m/s).

The strain energy absorbed (U) by the plug during a mudrush is determined as follows:

$$U = R_{punch\ shear} \times \mathrm{D} \tag{5}$$

where D is the deformation or distance travelled by the plug.

It is expected that the punch shear resistance would decay as the plug is displaced, due to damage to the concrete and rock at the interface.

3.4 Results

3.4.1 Hydraulic loading

Calculated FoS for a range of plug lengths are presented in Table 1, for both deep beam flexure and shear failure mechanisms. A 4 m long plug was selected to meet the guidance minimum design criterion FoS of 4. This configuration corresponds to a hydraulic gradient (defined as the ratio of static head to plug length) of 7, which is within acceptable limits for ‘fair’ quality rock conditions (Lang 1999).

Table 1 Factor of Safety (FoS) values for different plug lengths

Plug length	**3 m**	**4 m**	**5 m**
Deep beam flexure	–	–	–
Resistance (R) (MN)	64	113	177
FoS (hydrostatic)	4.6	8.2	12.8
FoS (hydrostatic + hydrodynamic PGV 0.06 m/s)	3.8	6.8	10.6
Shear failure	–	–	–
Resistance (R) (MN)	50	67	84
FoS (hydrostatic)	3.6	4.8	6.1
FoS (hydrostatic + hydrodynamic PGV 0.06 m/s)	3	4	5

3.4.2 Impact loading

Due to uncertainties in muckpile conditions, mud pocket volumes and flow characteristics, reliable quantification of these parameters is challenging. A sensitivity analysis of the energy absorption capacity and deformation response of the plug for a 4 m plug was carried out. It is expected that the shear resistance will reduce as more damage takes place during deformation caused by the impact of the mudrush. The plug design (Section 3.5) incorporates rebar anchored into the rock and the plug, which are not included in the shear resistance calculations, but will improve the shear resistance and toughness of the plug. To absorb the energy of a major mudrush the plug may need to deform a considerable distance along the tunnel, which would ultimately breakdown the cohesive strength of the concrete and rebars, and then the only remaining resistance is due to friction and the mass of the plug. The post-peak strength characteristics of the plug are uncertain, and therefore three possible toughness behaviours were considered. Possible relationships between load capacity and deformation are presented in Figure 6.

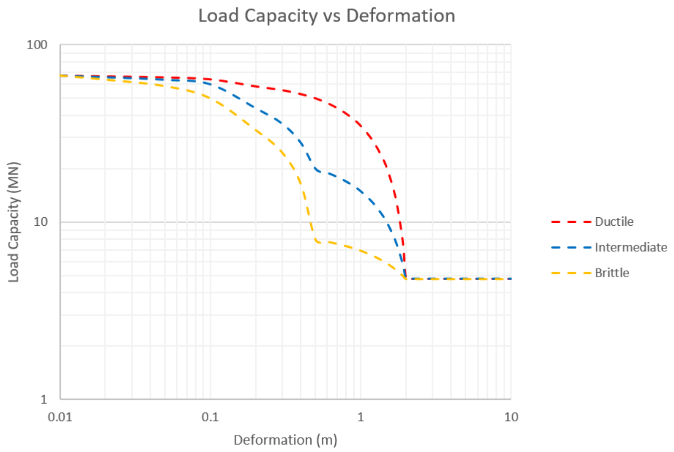


Figure 6 Load capacity versus deformation for different toughness behaviour

Plug performance ranges were estimated and defined based on the deformation of the plug and associated damage caused by an impact as shown in Figure 7:

- Deformations < 0.01 m result in no damage to the plug.
- Deformations between 0.01 m and 0.1 m may cause repairable damage (e.g. sealing via tightening holes).
- Deformations greater than 0.1 m are likely to compromise water retention capability, but the plug will continue to absorb energy, with decreasing shear resistance.
- The ultimate deformation capacity may be several metres, since the damaged plug would be pushed along the tunnel, but would only provide frictional resistance.

The energy absorption capacities for each performance range and plug lengths are indicated in Figure 7. For a 4 m plug, mud with kinetic energies of between 0.7 and 6 MJ would cause damage but would be repairable. The energy absorption capacity for up to 0.1 m is not significantly influenced by the toughness behaviour, but at much larger displacements, the toughness behaviour has a significant influence. The estimated energy absorption capacity for significantly larger displacements are also indicated on the graph.

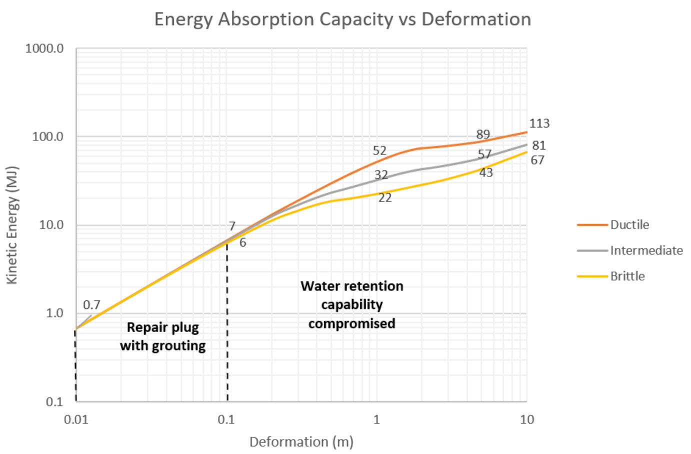


Figure 7 Kinetic energy versus deformation for different toughness behaviour

Figure 8 shows the kinetic energy for a range of volumes of mud with slurry density of 1,500 kg/m^3 travelling at a range of velocities. Mud pushes with velocities of less than 1 m/s are unlikely to cause damage to the plug. However, at higher velocities, damage could be significant and may require repairs or cause permanent damage.

A historical mudrush event at New Afton in 2021, with an estimated volume of 1,200 m³, could generate approximately 48 MJ of kinetic energy if impacting the plug at a velocity of 5.5 m/s (Figure 8). This velocity was conservatively chosen with reference to the International Geotechnical Society's UNESCO Working Party on World Landslide Inventory, and the velocity of the highest category 'extremely rapid' event description. Based on this comparison, it was concluded that while extreme events may result in significant damage to the plug, catastrophic failure that would allow the inrush to be transmitted is unlikely.

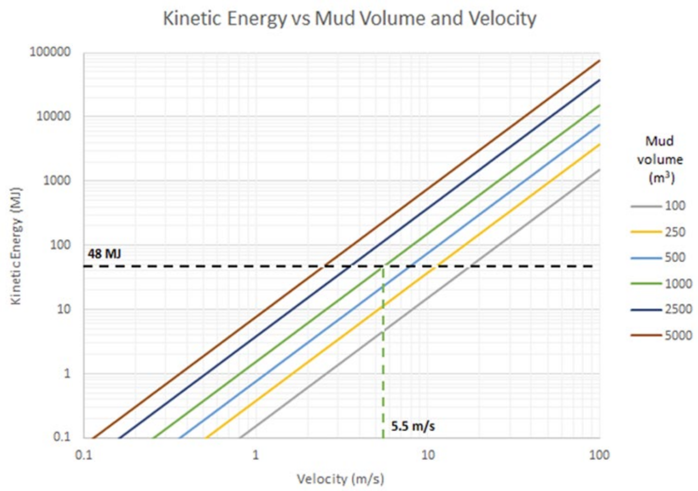


Figure 8 Kinetic energy versus inrush volume and velocity for a mud slurry density of 1,500 kg/m³

3.5 Plug design

In the Phase 1 project the initial concept for the plug construction was for a traditional temporary formwork type approach and multiple concrete lifts. Although the rear wall was planned to be a full height fibrecrete wall, past this there was the potential for lateral flow paths through the plug via cold joints between the lifts. Instead, New Afton mine proposed doing one continuous concrete pour incorporating approximately 30 m long filler holes drilled from the overlying MHD-4 drift. This provided a practical solution and, as far as the authors are aware, is a novel approach for the construction of plugs.

This methodology was advanced in the design stage, utilising two mesh-reinforced fibrecrete walls at each end of the plug footprint. The fibrecrete walls had dual purpose as form walls for the mass concrete pour and also as components of the plug length. The walls were connected internally with 'tie bars' to integrate them and provide lateral support for the pour. The majority of the mass pour was designed to be done on the level with tremie pipes via steel 'hatches' embedded in the front shotcrete wall. This would allow higher filling rates and visual inspection, and avoid potential concrete segregation by pouring (and dropping) via filler holes only. Hatches were to be sealed by bolting or welding cover plates prior to the core pour reaching their invert levels. The last plug core fill was designed to be via the filler holes collared in the drift crown to achieve tight fill. An upper steel hatch was also embedded in the front fibrecrete wall at the crown to enable visual confirmation of filling via inspection and observation of concrete spillage. The following are other considerations and elements of the design:

- New Afton informed that some seepage past the plug would be acceptable and which could be managed via existing water management infrastructure in DOD-001, and that pre-grouting would not be done. New Afton preferred to decide on the investment in pressure grouting based on observed seepage rates from the planned trial reservoir fill.
- The plug needed to incorporate 2 flow-through pipes with valves:

    - a 12-inch (305 mm) diameter pipe located near the floor to facilitate rapid drawdown and occasional flushing of settled fines
    - a 6-inch (152 mm) diameter pipe positioned approximately 2 m above the floor for water offtake.

- To address the potential seepage pathway along the pipe lengths, the pipes incorporated puddle flanges and were encased in decoupled concrete blocks formed and poured prior to the mass pour.
- A pressure sensor needed to be installed to enable monitoring of reservoir levels for plug operation management. Thermistors needed to be installed within the plug for monitoring of the concrete temperatures during and after the mass pour.
- To increase plug shear strength with the drift walls the design included installing multiple rings of rebar which protruded into the fibrecrete walls and plug core.

Considering the above and for the established loading conditions, plug calculations and design was carried out. Isometric drawings and construction task details were developed for the key stages in plug construction. Figure 9 shows the final stage.

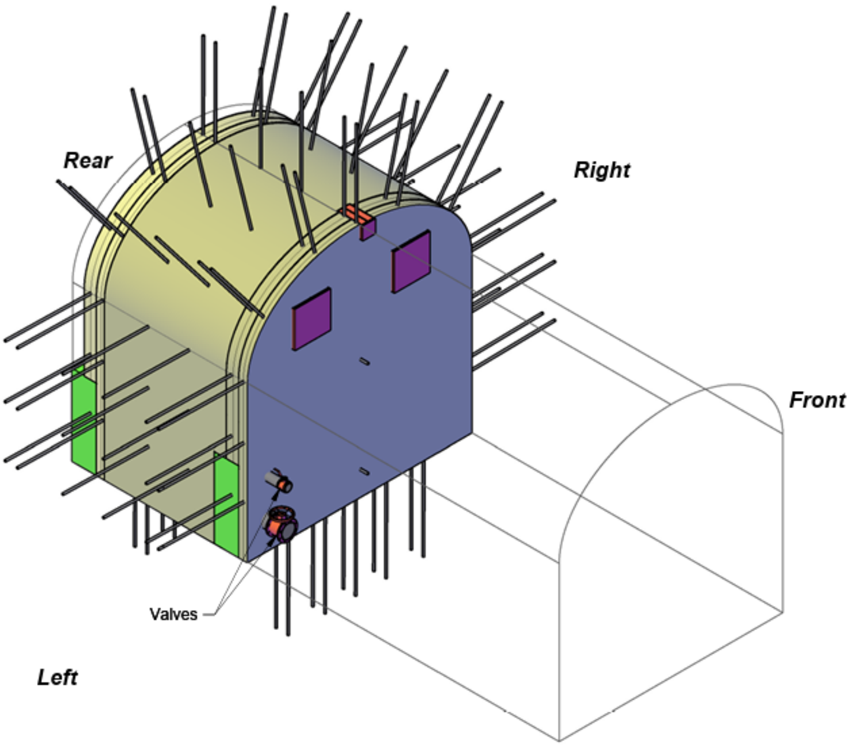


Figure 9 Isometric view showing the final plug configuration and components

4 Construction

Construction of the plug took place towards the end of 2024. Early construction stages included tasks such as preparing the plug footprint and walls, drilling the filler holes, installing protruding rebar in the drift back and walls, and installing the flow through pipes within formed concrete walls. A comprehensive inspection and testing plan was provided and referred to by New Afton mine to collect and document construction progress and for quality assurance (QA).

4.1 Concrete design and testing

A novel mix design was developed for the plug pour. Sulphate resistant cement was selected to mitigate against chemical attack from potentially high sulphide content water. Several mixes were tested with varying aggregate ratios, cement contents, water volumes, and additive levels. Fly ash was used to lower the overall heat of hydration of the mass pour. An aggregate top size limit of 10 mm was used to assist with achieving high flowability.

The final mix used a high cementitious content and super plasticiser (MasterGlenium 3030) dosage to achieve a self-compacting state without segregation of coarse and fine material, while still reaching intended strength targets. This was needed to ensure that the concrete flowed around all the fixtures and tight filled the plug core. Figure 10 shows slump flow testing of the concrete mix. A field test of the concrete was also completed on a waste dump to observe flow behaviour over a rock surface.

(a) (b)

Figure 10 (a) Slump flow testing showing a slump flow value of 685 mm (specification was 660–780 mm for the portion of the plug core poured from MHD-4); (b) Larger scale test pour over rock surface

4.2 Concrete pour

The mass pour was conducted at the end of November 2025 and took approximately 24 hours. Concrete batches of approximately 5m^3 were transported from the surface batch plant to the plug site in Transmixers. Upon arrival, testing including slump (CSA-A23.2-5C), slump flow (CSA-A23.2-19C), visual stability index (CSA A23.2-1 9C), and air content (CSA-A23.2-4C-14) was conducted to verify batch quality against the criteria. Cylinder samples were collected for laboratory compressive strength testing at 7, 21 and 28 days. Nineteen concrete loads were batched but some were rejected due to not meeting concrete flowability criteria, and some were partially dumped due to construction interruptions or being excess to volume requirement. A total volume of 80 m^3 was poured into the plug core.

The pour went mostly without incident; fine cracks started to appear in the front fibrecrete wall at approximately mid-pour height and subsequently started weeping. The crack extents were marked with spray paint and their progression closely monitored. Distance between the wall to fixed string lines set up prior to the pour were regularly measured to monitor for displacement of the wall. The measurements indicated that the wall was not deforming and the crack extents tended to stabilise and did not coalesce.

Figure 11a is a photo of the plug core showing the flat concrete surface indicating high concrete flowability. Figure 11b is a photo of the front fibrecrete wall showing the sealing one of the hatches. Incomplete sealing of this hatch led to leakage of concrete from the plug core and concern of release of a larger volume. When this occurred the concrete was being poured through the filler holes. When observed, the pour was stopped and investigations were carried out. After thorough evaluation of the situation and volume balance it was determined that the plug core was already full. Rather than loading the hatch further the decision was made to stop pouring and return later to 'top up' the filler holes.

(a) (b)

Figure 11 (a) Photo taken through a front wall hatch of the plug core showing the concrete tremie pipe, and flat concrete surface at the level of the upper flow-through drainage pipe; (b) Photo of the front wall showing one of the hatches being sealed, and crack and wall deformation monitoring measures via marking with spray paint and string lines (upper line disconnected and slack at this time)

4.3 Instrumentation

Six thermistors were installed within the plug prior to spraying the front fibrecrete wall. The instrument ends were secured to the tie bars in the core of the plug at 2 separate elevations at 3 locations along the length of the plug. The purpose of the instrumentation was to monitor the heat of hydration and to determine if delayed ettringite formation and subsequent expansion cracking could be possible in the concrete mass. Figure 12 shows that the thermistor that was situated closest to the centre of the plug showed the highest peak reading of approximately 77°C. Knowing that only one of the thermistors rose significantly above the 70°C criteria and that pressure grouting was an option to seal any potential cracking, the decision was made to advance to the test filling of the reservoir.

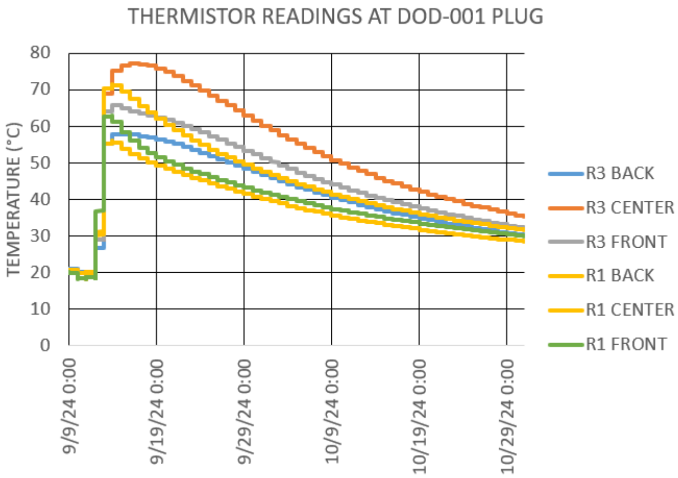


Figure 12 Thermistor data from the plug during the mass pour and concrete curing

5 Initial operation

5.1 First fill

Following construction, plug performance was tested via controlled filling of the reservoir to maximum storage capacity. Based on the drift and raise geometry behind the plug, a theoretical pressure development curve was established for monitoring. Given the expected inflow rates, filling was anticipated to take approximately 7 days, with a maximum pressure of about 250 kPa. Checks of the plug were completed on a routine basis during the fill of the plug integrity and seepage. Estimated and observed pressures at the plug face are shown in Figure 13.

At maximum reservoir capacity, elevated seepage was observed in DOD-001 emanating from the tunnel walls and hatches. Flow was interpreted to be primarily via the formation rock mass and the contact with the plug, with some paths also along the rear of the front fibrecrete wall contact and out the hatches. No new significant flow pathways were observed in any of the adjacent drifts. Seepage rates ranged from approximately 1.3 to 1.9 L/s.

Although the test fill successfully verified the structural integrity of the plug under the maximum reservoir capacity, the seepage rates significantly exceeded the target of 0.5 L/s (Lang 1999). Due to the relatively high strength of the rock mass and absence of softening coatings, erosion of seepage pathways was considered unlikely. However, the elevated seepage created water management challenges and increased operator concern regarding the perceived risk of working in the vicinity of the plug. As a result, the reservoir was maintained drawn down until seepage control measures could be implemented.

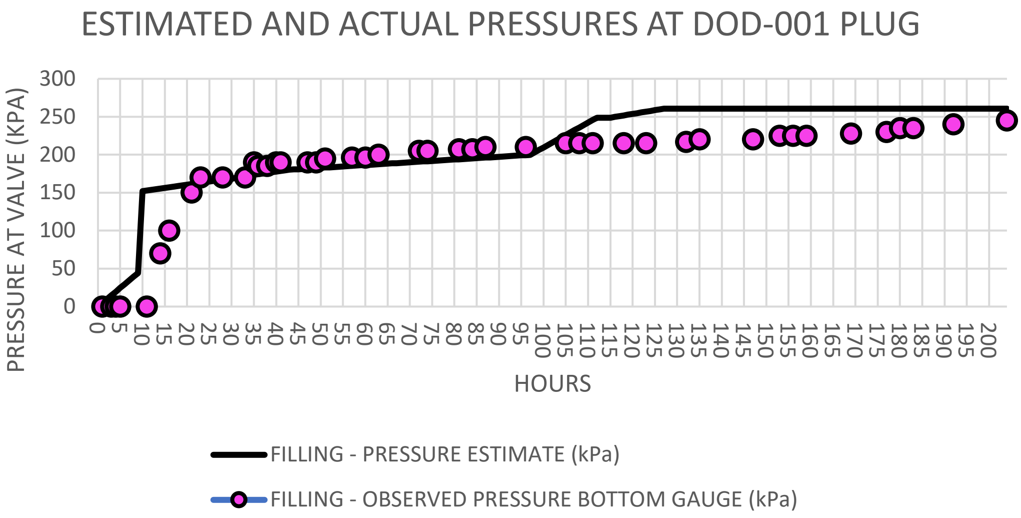


Figure 13 Theoretical and measured pressure curve developed for the DOD plug during filling

5.2 Seepage control

Seepage mitigation was undertaken via a systematic resin grouting program targeting the surrounding rock mass and plug contact. Injection holes were drilled into the front wall and surrounding rock mass at approximately 1 m spacings. MasterRoc MP 351 LV resin was used as the injection product. This product was selected as it expands on contact with water through a foaming process that cures within minutes of injection.

Upon completion of the injection program, a second reservoir filling test was conducted. The test demonstrated a significant reduction, with rates of approximately 0.8 L/s. Visual inspections also confirmed that seepage was materially reduced, including when the plug was again subjected to maximum reservoir capacity. A risk assessment completed for operation of the reservoir identified a residual risk that seepage pathways may enlarge over time; however, the risk was considered acceptable provided that an appropriate monitoring plan was in place.

6 Conclusion

Although the steps in the process of design, construction and operation were well thought through, there were some important learnings from this case study.

It is virtually impossible to know historical cave muckpile physical and hydraulic conditions with any certainty which makes it challenging to define the potential for, and the magnitude of, inrush events. By considering a range of conditions, it was possible to define scenarios of higher risk. A sensitivity check of energy absorption capacity of a range of plug lengths and post-event condition was conducted referring to historical inrush event data and published literature on landslide velocities. The conclusion was that for this event, the plug would stop the inrush material, but it would no longer be functional for water retention. However, the inrush material would block the reservoir, and it would not be possible to remove the mud without removing the plug anyway.

The initial design used traditional formwork and multiple concrete lifts which risked forming lateral flow paths through the plug along horizontal cold joints. The alternative methodology proposed by the mine site of incorporating pouring from filler holes avoided this. It was also a practical way of constructing via single mass pour and assuring tight fill to the drift crown. This case study has shown the methodology to be viable and could be considered by other operations in a similar situation.

The decision to delay the call on pressure grouting until after observation of seepage provided useful insight to the role and effectiveness of pressure grouting. In this case the post-construction grouting of the formation and plug contact effectively reduced seepage rates to acceptable/manageable levels. For scenarios with less favourable formation conditions, conducting pre-construction grouting is advised.

The adopted construction methodology – shotcrete form walls, low-heat mass concrete placed in a controlled sequence through hatches and crown filler holes, and QA testing throughout the pour – helped manage the principal construction risks of thermal cracking, segregation, and incomplete back contact. However, the release of concrete from one of the filler hatches (due to incomplete sealing) in the final stages of the pour highlights the requirement for thorough QA on all aspects of construction.

The goal of establishing a reservoir for water re-use at New Afton mine was safely and effectively achieved and has created a meaningful environmental benefit by reducing the freshwater withdrawal from Kamloops Lake. This is significant and potentially something that other underground mining operations in a similar situation could consider.

Acknowledgement

The authors would like to acknowledge New Gold Inc. and Coeur Mining, Inc. for their collaboration and permission to publish this work. The contribution of site personnel at the New Afton mine is gratefully recognised, particularly for providing data, operational insights, and ongoing technical input that were essential to the development of this study. This publication is based on the authors' operational experience and observations while working at the New Afton mine. The information is provided for general informational purposes only, and does not represent the views, positions or official guidance of Coeur.

References

Bieniawski, ZT 1989, *Engineering Rock Mass Classifications: A Complete Manual for Engineers and Geologists in Mining, Civil, And Petroleum Engineering*, John Wiley & Sons, New York.

ISRM 1981, *Rock Characterisation Testing and Monitoring*, ET Brown (ed.), Pergamon Press, Oxford.

Kennedy, G & Goodchild, CH 2003, *Practical Yield Line Design*, 1st edn, British Cement Association and Reinforced Concrete Council, Surrey.

Lang, B 1999, 'Permanent sealing of tunnels to retain tailings or acid rock drainage', *International Mine Water Association Proceedings*, pp. 647–655.

UK Health and Safety Executive 2005, *The Design and Construction of Water Impounding Plugs in Working Mines*, viewed 1 June 2026, https://www.hse.gov.uk/mining/waterplu.htm

Westergaard, HM 1931, 'Water pressure on dams during earthquakes', *Transactions of the American Society of Civil Engineers*, paper no. 1835.