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# Closure of legacy waste rock piles: can we achieve passive treatment to manage residual seepage in the short term?

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## Abstract

*Closure and reclamation of waste rock piles using engineered cover systems and water treatment of seepage is a common technique for management of acid rock drainage/metal leaching (ARD/ML) to mitigate adverse impacts to the receiving environment. Linking cover system performance (i.e. net percolation and/or oxygen ingress) to impacts to the receiving environment provides a rational basis for cover system design criteria. Two general models are used in the mining industry to predict long-term seepage and closure costs. One model assumes that loading of contaminants to the environment will remain unchanged under reduced flux rates (net percolation) where contaminant concentrations increase proportionally as a function of decreasing flow; the other assumes that a reduction in the flux rate will result in decreased contaminant loads for constant contaminant concentrations in flow. Both models will have a transition point at which reduced loading to the receiving environment will occur as the flux decreases.*

*Enterprise Cape Breton Corporation implemented a program for the closure of historic coal mines located near Sydney, Nova Scotia, Canada, with Public Works and Government Services Canada providing project management. The Victoria Junction waste rock pile was reclaimed with an engineered cover system comprising a 60 mil HDPE geomembrane, a granular drainage layer and an overlying growth medium. It was estimated following closure that active treatment of seepage waters impacted by ARD/ML would be required for ~20 years before passive treatment systems could be established; however, this has occurred within seven years with significant cost savings realised. Reclamation of the site reduced the risk of potential loading under various failure scenarios. An acidity mass balance was used to provide an understanding of past (uncovered waste rock pile and active treatment), current (covered waste rock pile and passive treatment) and long-term (100 years) loading under progressive changes to water collection and treatment activities. The acidity mass balance will serve to inform management decisions in ongoing closure planning. The case study presented here demonstrates that it is possible to eliminate the need for active treatment of seepage for legacy sites.*

## 1 Introduction

Challenges exist in developing closure costs for mine waste storage facilities, primarily owing to uncertainty in predicting long-term pore-water quality and the subsequent acidity load associated with a range of cover system performance (i.e. oxygen and water ingress) scenarios. Two conceptual models are used in the mining industry to estimate acidity generation for managing acid rock drainage/metal leaching (ARD/ML). In model 1, ARD/ML load increases linearly with flow. Concentration in model 1 is assumed to be constant, so the increase in load is produced by the increase in flow. Conversely, the ARD/ML load in model 2 is assumed to be constant with changes in flow. This is produced by high concentrations at low flows and low concentrations at high flows due to the effects of dilution. Both models are depicted graphically in Figure 1.

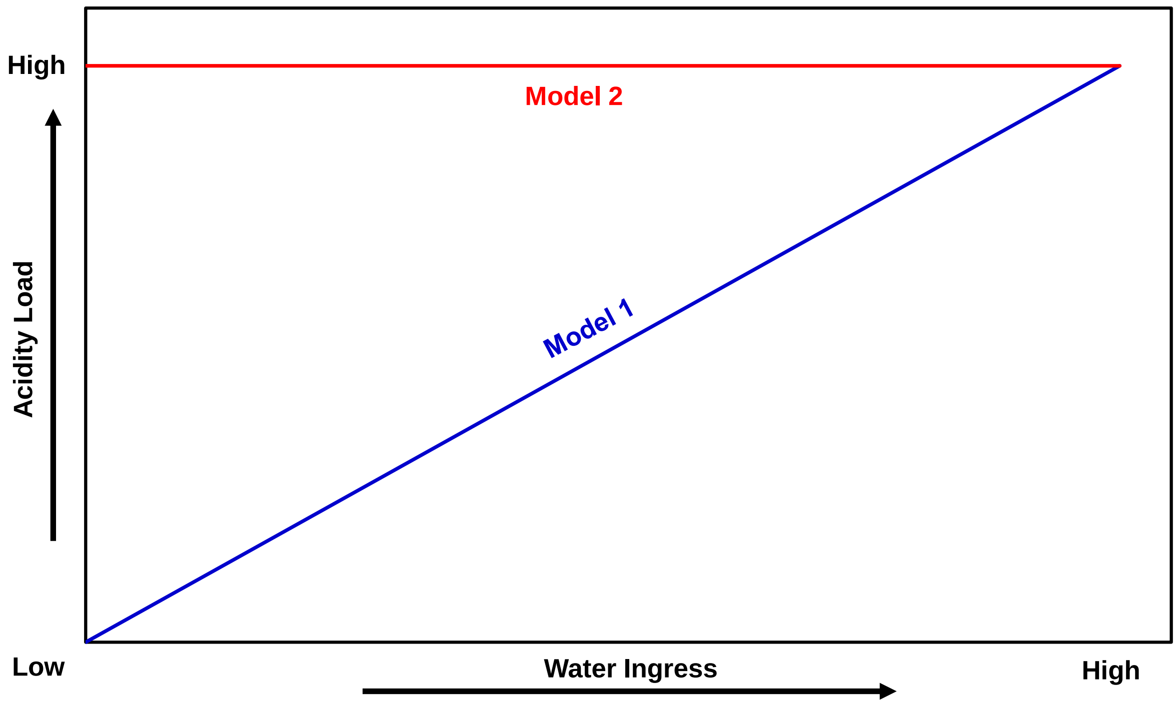


**Figure 1 Model 1 and model 2 ARD/ML loadings**

While neither model could exist in its perfect form in practice, given the range of site-specific controls such as flow, residence time and other physiochemical factors, it is likely that a combination of the two models will exist. Assuming physiochemical factors remain constant, it is anticipated that the concentration change in model 2 will be more prolific during periods of high flow, reducing residence time and allowing for dilution. Conversely, the constant concentration of model 1 will be more prolific under low rates of water ingress, allowing adequate residence time for pore-water to equilibrate with the in situ conditions. O'Kane (2014) proposed an alternate model for estimating acidity load generation in waste that takes into consideration changes in physiochemical factors in response to changes in oxygen and water ingress. Given the uncertainty in predicting long-term pore-water quality under various rates of water and oxygen ingress, challenges exist in understanding the benefits of implementing a cover system compared to collection and treatment in perpetuity.

While there are challenges in predicting pore-water quality, observed loadings to the downstream receptor mark the culmination of progressive changes to mining and reclamation, and water collection and treatment. It is essential that an understanding of observed loadings to downstream receptors be demonstrated prior to developing predictions of long-term performance. Through the development of a conceptual model, this paper has made use of an acidity mass balance to provide an understanding of past (uncovered and active treatment), current (covered and passive treatment) and long-term loading under progressive changes to water collection and treatment activities. Constant concentration (i.e. model 1) was observed for past and current conditions. Constant concentration was used in the acidity mass balance to predict long-term (100 years) loading given the observed internal temperature and low water and oxygen ingress to the reclaimed waste rock pile (WRP).

The conceptual model and mass balance provides an understanding for the site and, should it be required, serve to inform on numerical modelling completed within subsequent components of the project. The reclaimed Victoria Junction WRP is a unique case study because of its pilot scale, which allows a transition from active to passive treatment to occur in a relatively short time period following cover system placement compared to what would be expected for larger waste facilities typical of the mining industry.

## 2 Background

Cape Breton Development Corporation (CBDC) was established as a Crown corporation in 1967 in order to reorganise and rehabilitate the coal industry on Cape Breton Island, Nova Scotia, Canada. In 2009, CBDC was dissolved, and its assets and liabilities were transferred to Enterprise Cape Breton Corporation (ECBC), a federal Crown corporation. Under the transfer arrangement, ECBC acquired stewardship obligations stemming from CBDC's past operations, including land holdings and environmental remediation.

Properties covered under the environmental remediation program stem from mining operations that began in 1685 and include more than 50 underground mines, which produced over 500 million tonnes of coal. The history of coal mining in the Sydney coal fields included 720 individual parcels of land on which there were 95 coal related operations covering more than 1,000 $km^2$. Some of the properties required remediation of WRPs produced from the mining operations.

The reclaimed Victoria Junction WRP is located on the site of a historic coal preparation plant approximately 3 km east of Sydney and has a footprint of approximately 26 ha and a height of 40 m. Public Works and Government Services Canada, under ECBC, provided project management for the remediation program. In 2014, ECBC was dissolved into Public Works and Government Services Canada, which took ownership of the site.

The coal preparation plant was commissioned in 1976 and produced 750 short tons per hour (stph) of metallurgical and thermal grade coal, which was upgraded to 1,000 stph in 1981/82. The coal preparation plant operated until the Phalen Colliery closed in 2000, and then a blending facility operated on the site until 2001. The concern after decommissioning was ARD/ML into Northwest Brook, which flows from Grand Lake, located 700 m south of the WRP, to Lingan Bay/the Atlantic Ocean, approximately 4 km north of the WRP. Northwest Brook flows around the east side of the WRP, where it branches into a wetland area and reforms into a channel before monitoring point 2016 (MP2016). Figure 2 is a plan view that shows the reclaimed Victoria Junction site during the period of active treatment.

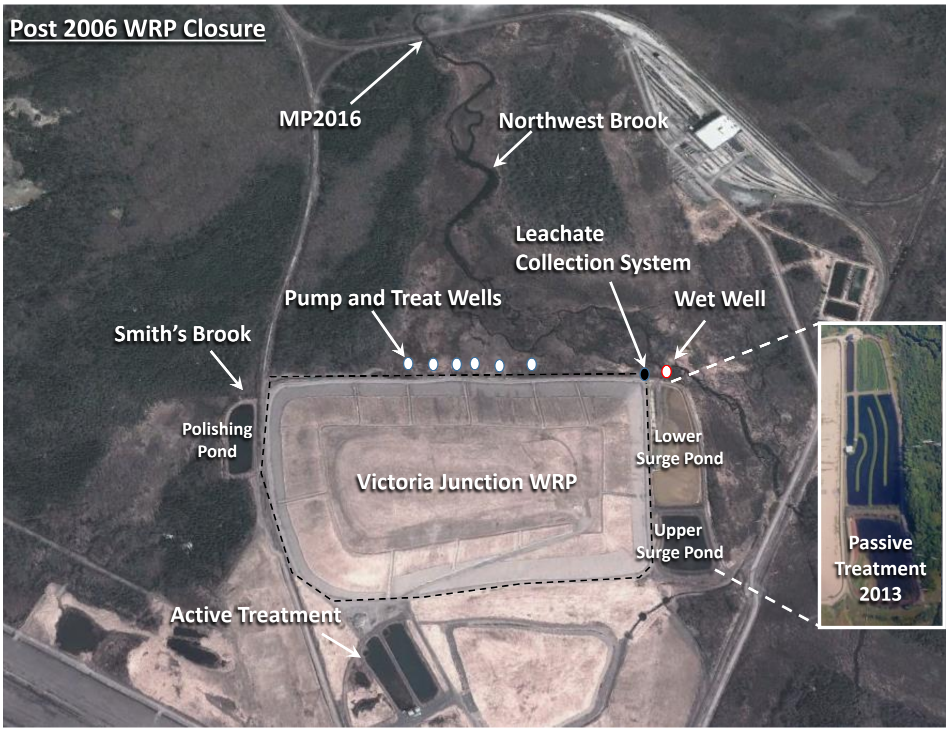


**Figure 2 Victoria Junction post-2006 closure site features; the lower surge pond converted to passive treatment in 2013 is also illustrated**

The facility washed up to 4 million tonnes of raw coal per year, of which 15–20% was placed into the WRP and 3% into fine tailings ponds. Coal tailings ponds (CTP) 1 and 2 were located south of the main WRP. During operations, tailings within these ponds were relocated on an alternating basis and placed within the northwest footprint of the WRP. These tailings ponds were replaced over a period of three years beginning in 1980 by CTPs 3, 4 and 5, located within the central and eastern portion of the WRP, which were eventually covered with waste rock.

In 1987 a 1 m thick bentonite wall was constructed along the north and east toe of the WRP to reduce ARD/ML moving through shallow overburden into the wetland. A toe-drain collected upwelling seeps that were treated through the active treatment plant. The toe-drain was upgraded to the leachate collection system in 2006, during construction of the cover system.

A basic lime slurry addition water treatment plant was located adjacent to the WRP for the collection and treatment of ARD/ML-affected runoff. This was replaced in 1994 with another lime slurry water treatment plant that included various surge, settling and polishing ponds before discharge into Smiths Brook east of the pile. Regulations do not apply for discharge from a coal preparation site on Cape Breton Island; however, voluntary compliance with metal mining liquid effluent regulations was accepted to meet suspended solids, pH and various dissolved metals guidelines. In 2003, six pump-and-treat wells were installed into bedrock to the north of the WRP to intercept deeper impacted groundwater. A wet well was installed north of the lower surge pond to intercept ARD/ML-impacted water percolating through the base/lining of the lower surge pond. The lower surge pond received water from the pump-and-treat wells, the leachate collection system and the wet well through a series of pumps.

An engineered cover system was implemented as part of the closure plan for the WRP in 2006. The cover system was designed based on several closure objectives (AMEC, 2005a):

- providing a measurable positive effect on the environment compared to the impact pre-closure;
- minimising the impact to wetlands connected through Northwest Brook; and
- site aesthetics in accordance with industry standards and public expectations.

The cover system comprises a till growth medium layer placed over a granular drainage layer (GRDL) and a 60 mil HDPE geomembrane, as shown in Figure 3. The cover system was designed to limit the ingress of meteoric water and oxygen, thus limiting the transport of stored acidity and generation of potential acidity.

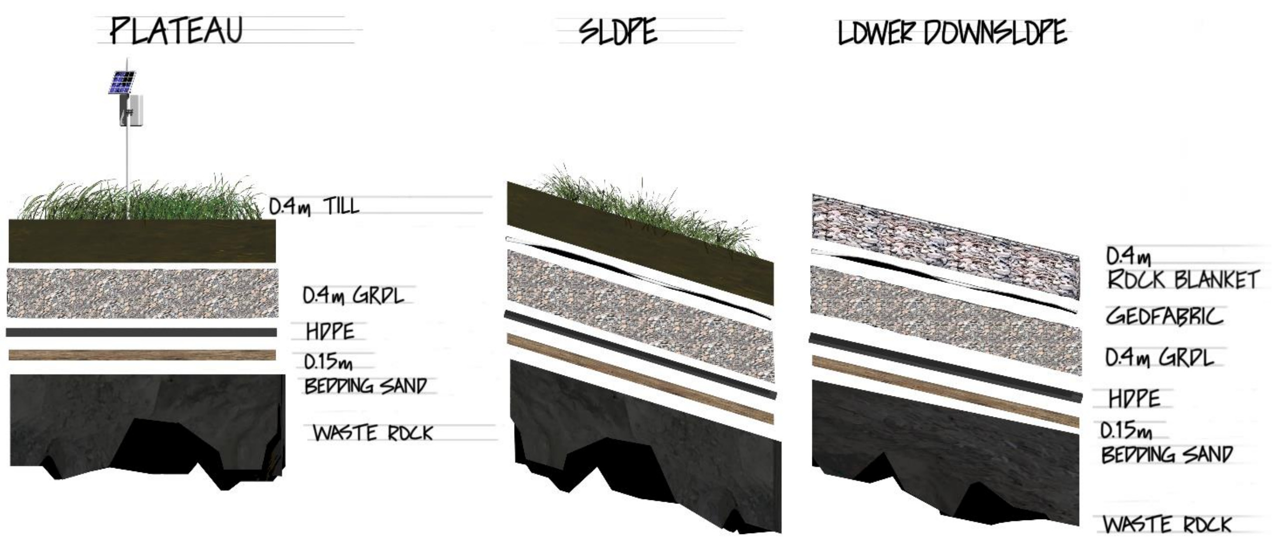


**Figure 3 Victoria Junction cover system profile**

In 2013, the active water treatment plant was decommissioned and a passive water treatment system constructed in the lower surge pond to the east of the WRP, as illustrated in Figure 2. In reality, the passive treatment system is not “passive,” as it is recharged with lime and polymer, and ARD/ML-impacted water is actively pumped from the leachate collection system into the treatment system. However, for the purposes of this paper, the system is considered passive given that the lime slurry plant was decommissioned, limited lime/polymer is added to the natural system and annual operational costs decreased from approximately C$ 200,000 to C$ 1,800.

# 3 Conceptual model

A conceptual model was developed for the site to provide an understanding of past and current conditions to provide information about long-term impacts from the WRP. The conceptual model included physical, flow and geochemical components.

## 3.1 Physical and flow models

The Victoria Junction WRP contains an estimated 10 million tonnes of potentially acid-generating waste rock. Water dynamics at the site include meteoric contributions as well as a groundwater component. The groundwater table in the area is relatively shallow (1–5 mbgs). Tailings ponds within the WRP created strong vertical hydraulic gradients, forcing the ARD/ML plume deep into the underlying bedrock. Migration of the plume is northward with groundwater flow and then upward to the surface of Northwest Brook because of higher water pressures in the underlying bedrock compared to the overlying till. As a result, loading from the pile to the receiving environment can be quantified north of the pile at MP2016.

The saturated hydraulic conductivity ($k_{sat}$) of the waste rock varies considerably, from $10^{-6}$ to $10^{-1}$ cm/s (JWEL, 2002), owing to the construction techniques used and heterogeneity in the material. While there is a considerable range in the $k_{sat}$ of the waste rock, the primarily fine-textured material in the pile is in the range of $10^{-6}$ to $10^{-4}$ cm/s. The tailings buried in the lower portion of the WRP are homogeneous with a $k_{sat}$ in the range of $10^{-6}$ to $10^{-5}$ cm/s (JWEL, 2002), and act as a “lower” permeable layer. Through the completion of a site water balance, AMEC (2005b) characterised water ingress through the bare waste rock at approximately 250–450 mm/year for the side slopes plateau, respectively. These rates of ingress are two orders of magnitude larger than what would reasonably be expected for the engineered cover system.

Water levels in monitoring wells prior to placement of the cover system indicate the presence of perched water within the WRP, as illustrated in Figure 4. The high pre-closure rate of water ingress and low hydraulic conductivity at the base of the WRP are the main contributions to perched water within the WRP. An observed decrease in water levels occurred in the pile following installation of the cover system and associated reduction in water ingress. Line 1 of Figure 4 was generated from water levels observed prior to placement of the cover system and shows water perched to an elevation of approximately 33 m, approximately 10 m above the base of the WRP. Line 2 represents the current elevation of water levels, a decrease of approximately 5 m since 2006. The predicted water table elevation at cessation of drain-down was determined from water elevations up- and downstream of the WRP and wells within the footprint of the pile screened in underlying till. The water elevations for wells completed in the till have not changed over time and are reflective of Line 3, indicating that the final water table will likely mound within the base of the pile at approximately 3 m and diminish to near zero at the toe drain/leachate collection system.

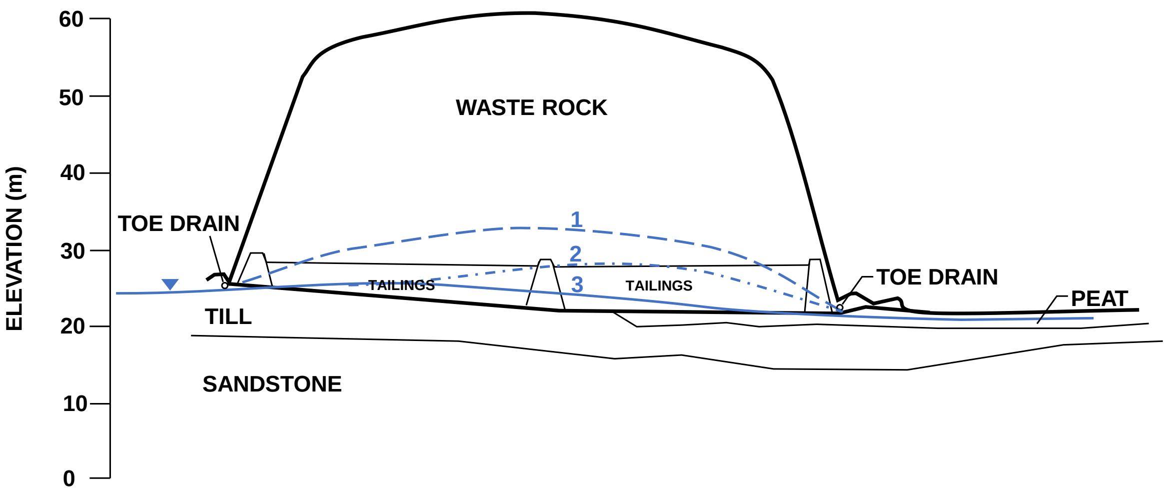


**Figure 4 South-north cross-section of the Victoria Junction WRP with (a) Line 1, perched water elevation at time of cover system placement, (b) Line 2, current perched water elevation, and (c) Line 3, water mounding within base of pile following cessation of drain-down**

There are two components to drain-down for the Victoria Junction WRP: saturated-unsaturated drain-down, which will contribute greater loading over a shorter period; and unsaturated drain-down, which will occur over a longer time frame and generate a lower loading. Figure 5 shows water levels (i.e. saturated-unsaturated drain-down) in two of the monitoring wells that were completed in the tailings layer one year prior to placement of the cover system. The decrease in water levels illustrates a more rapid drain-down shortly after placement of the cover system, followed by a gradual decrease. The trend in the rate of drain-down was extrapolated to estimate the cessation of saturated-unsaturated drain-down, as shown in Figure 5. It is estimated that the drain-down of water perched within the pile will continue for another 10–20 years, at which time the unsaturated drain-down of water will result in a much lower rate of basal seepage. In terms of unsaturated drain-down, under hydrostatic conditions, the maximum pressure head that could be achieved in the uppermost portion of the WRP is 300 kPa. It is assumed that much of the unsaturated drain-down in the upper reaches of the pile has already occurred given the waste rock water retention curve and in situ matric suction. The suction sensors provide matric suction in the range of approximately 100 kPa.

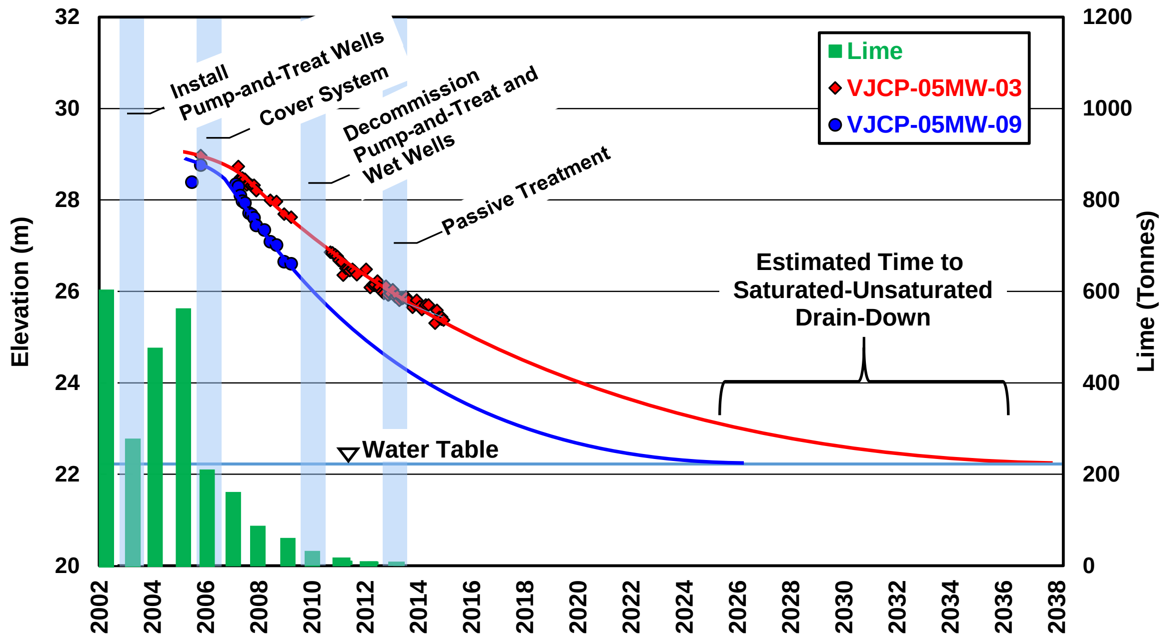


**Figure 5 Water mounding trends, estimated time to drain-down, and lime usage**

Annual lime consumption for the active treatment plant from 2002 to 2013 and the passive treatment system from 2013 is also presented in Figure 5. During the active treatment phase, the lime slurry feed was set to achieve a pH of 8.5. There are two important trends worth highlighting in Figure 5, with the first being that lime consumption is highly variable prior to placement of the cover system. The variability in lime consumption is attributed to runoff and basal seepage being highly susceptible to atmospheric forcing (i.e. rainfall intensity and volume). Subsequent installation of a cover system attenuated peak discharge and reduced the burden on the water collection and treatment facility. In addition, installation of the pump-and-treat wells in 2003 added to the acidity load for treatment.

The second and equally important trend in lime consumption is the constant decrease following placement of the cover system. The rate of drain-down is currently on the order of 75 mm/year and currently contributes the bulk of loading from the pile. Once drain-down has ceased, loading to the receiving environment will be significantly reduced and mainly consist of net percolation and the residual effects of groundwater mounding.

Net percolation, or leakage through the geomembrane, was estimated for a range of defects and heads of water that could form above the layer. Water will flow through the geomembrane defect, then laterally some distance between the geomembrane and underlying material and then into the underlying material. The distance water flows between the geomembrane and underlying material is dependent on the contact between the two layers. Analytical solutions are available for “good” and “poor” contact conditions (Giroud et al., 1992). Leakage rates estimated for the Victoria Junction WRP are based on a poor contact given that bedding sand was placed between the waste rock and geomembrane providing a conduit to flow. Net percolation was estimated to be in the range of 2 to 5 mm/year.

While the aforementioned low net percolation and trend in lime consumption support lower loadings, uncertainty exists in the relative benefit realised from the cover system given the culmination of factors affecting loading to the receiving environment. For example, as identified in Figure 5, installation of the pump-and-treat wells in 2003 would have increased loading to the active water treatment facility, while decommissioning of the pump-and-treat and wet well in 2010 would have had the opposite effect. In order to better understand long-term impacts to the receiving environment, acidity mass balances were developed to quantify changes in loading associated with the installation of the cover system, water mounding, drain-down and progressive changes to the water collection and treatment facility; these are discussed in the following section.

## 3.2 Geochemistry conceptual model

An estimate of stored and potential acidity in the Victoria Junction WRP was developed based on a preliminary geochemical assessment. It is assumed that all sulphate is present as melanterite ($FeSO_4$) and that two moles of $H^+$ will be produced per mole of sulphur. This is a conservative assumption and assumes a maximum stored acidity load. As noted above, there are two distinct materials in the WRP: waste rock and tailings. The average total sulphur contents of the waste rock and tailings are 2.4 and 1.3 wt%, respectively, and the acid potential can be calculated by:

$$AP\ (kg\ CaCO3/tonne) = wt\%\ Total\ Sulfur \times 31.25 \tag{1}$$

Where:

AP = the acid potential in $CaCO_3$

31.25 = a conversion factor based on stoichiometry of the neutralisation reaction of $FeS_2$ by $CaCO_3$.

A conservative approach was assumed with the entire WRP having a sulphur content of 2.4 wt%. This will be refined following refinement of the WRP biography. The potential and stored acidity for the WRP were calculated using the acid-generating potential determined by JWEL (1995) and are summarised in Table 1. The bulk of the acidity in the Victoria Junction WRP is present as potential acidity, which will be generated as a function of oxygen ingress and then transported to the receiving environment as a function of net percolation.

**Table 1 Waste rock acid base accounting**

| Waste rock (t) | Sulphur (wt%) | Acid-generating potential (kg $CaCO_3$/t) | Total acidity (kg $CaCO_3$) | Potential acidity (kg $CaCO_3$) | Stored acidity (kg $CaCO_3$) |
|---|---|---|---|---|---|
| 10 million | 2.4 | 72.1 | 750,000 | 720,907 | 29,093 |

Potential and stored acidity were used to support the acidity mass balance and long-term predictions of loading to the receiving environment.

### 3.3 Acidity mass balance and loading

Loading to the receiving environment has evolved over time and is characterised by three distinct periods: pre-cover system with active treatment, post-cover system with passive treatment and long-term post-cover system with passive treatment. An acidity mass balance was developed for each phase, with the pre-cover mass balance shown in Figure 6. In all phases it is assumed that oxygen ingress is sufficient to generate potential acidity at a rate greater than the mobilisation of stored acidity by net percolation. Under these conditions, the generation of potential acidity will contribute to the total stored acidity and eventually be mobilised with net percolation. The limiting factor on loading to the receiving environment is therefore net percolation. This is a conservative assumption, as oxygen concentrations decrease with depth and diffusion across the HDPE membrane is low. It is likely that there is a depth within the WRP at which oxygen concentrations are insufficient to generate acidity at the mobilisation rate, given that oxygen transport is limited to diffusion.

The load to the receiving environment pre-cover system consists of groundwater mounding, net percolation and runoff from the site. There is also an alkalinity contribution from the polishing pond. The load for each component was estimated based on average calculated acidity concentration and flow. Groundwater mounding and net percolation used an average acidity concentration of 3000 mg/l and are conservatively estimated to remain at this concentration until the total acidity in the WRP is depleted. A groundwater flow rate of $1 \times 10^{-5}$ cm/s was determined based on JWEL's (2002) hydrogeological investigation. Mounding was estimated to extend 3 m into the base of the waste rock (Figure 4). A net percolation rate of 350 mm/year was used based on estimates from AMEC's (2005b) closure plan feasibility design, and the cross-section was considered to be the entire 26 ha footprint of the WRP.

The active treatment system collects flow from site runoff and the wet well, toe drain and pump-and-treat wells. Active treatment system loads were estimated from JWEL's (2002) hydrogeologic investigation and resulted in an average input flow and acidity concentration of 1,500 l/min and 1,000 mg/l, respectively, and a 5 mg/l output concentration. Input to the active treatment system was proportioned between runoff from the site and pump/treat, where the pump/treat load consists of the toe drain, wet well and pump-and-treat wells (AECOM, 2009). The pump/treat load was calculated from average monthly flow and concentration. Runoff from the site was back-calculated from the pump/treat and active treatment system loads.

Natural groundwater alkalinity is conservatively assumed to be able to reduce acidity by 2 t/year based on groundwater flow rates and background water quality. Considering all of the acidity contributions and "sinks" for basal seepage, there is a calculated load of 161 tonnes per year. The observed load at MP2016 based on historic concentrations (JWEL, 2002) and estimated flows (exp, 2014) is 150 t/year. The overall error of 1% would suggest that the flow model used to close the acidity mass balance is representative of site conditions.

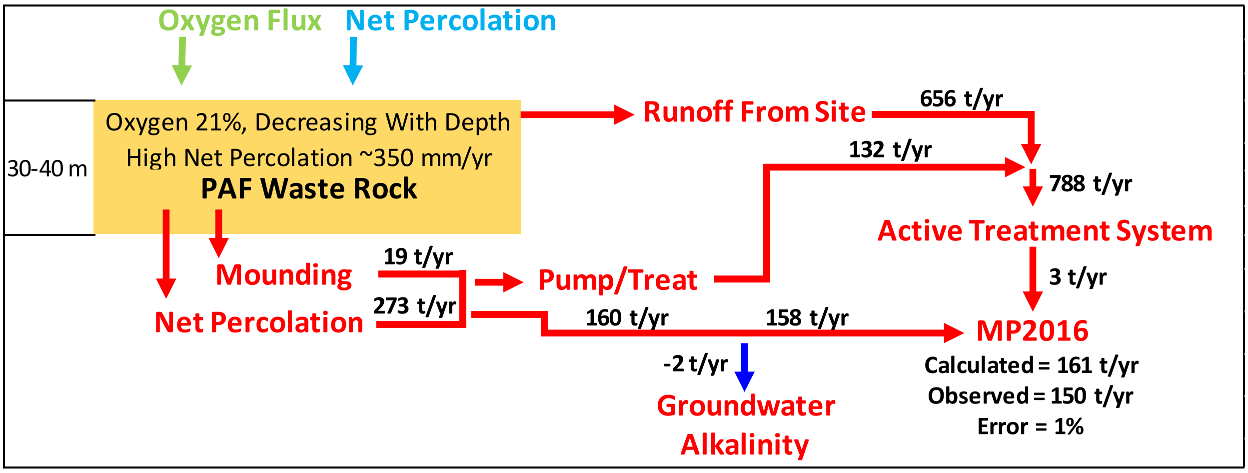


**Figure 6** Victoria Junction WRP acidity mass balance pre-cover system

The total acidity generated is 948 t/year, of which runoff is the largest contributor at 656 t/year. The active treatment system neutralised 785 t/year, while 2 t/year were neutralised by natural alkalinity in groundwater. In terms of basal seepage, approximately 45% is intercepted by the active treatment system, with the remaining reporting to groundwater flow, which is key in developing an understanding of loading to the receiving environment.

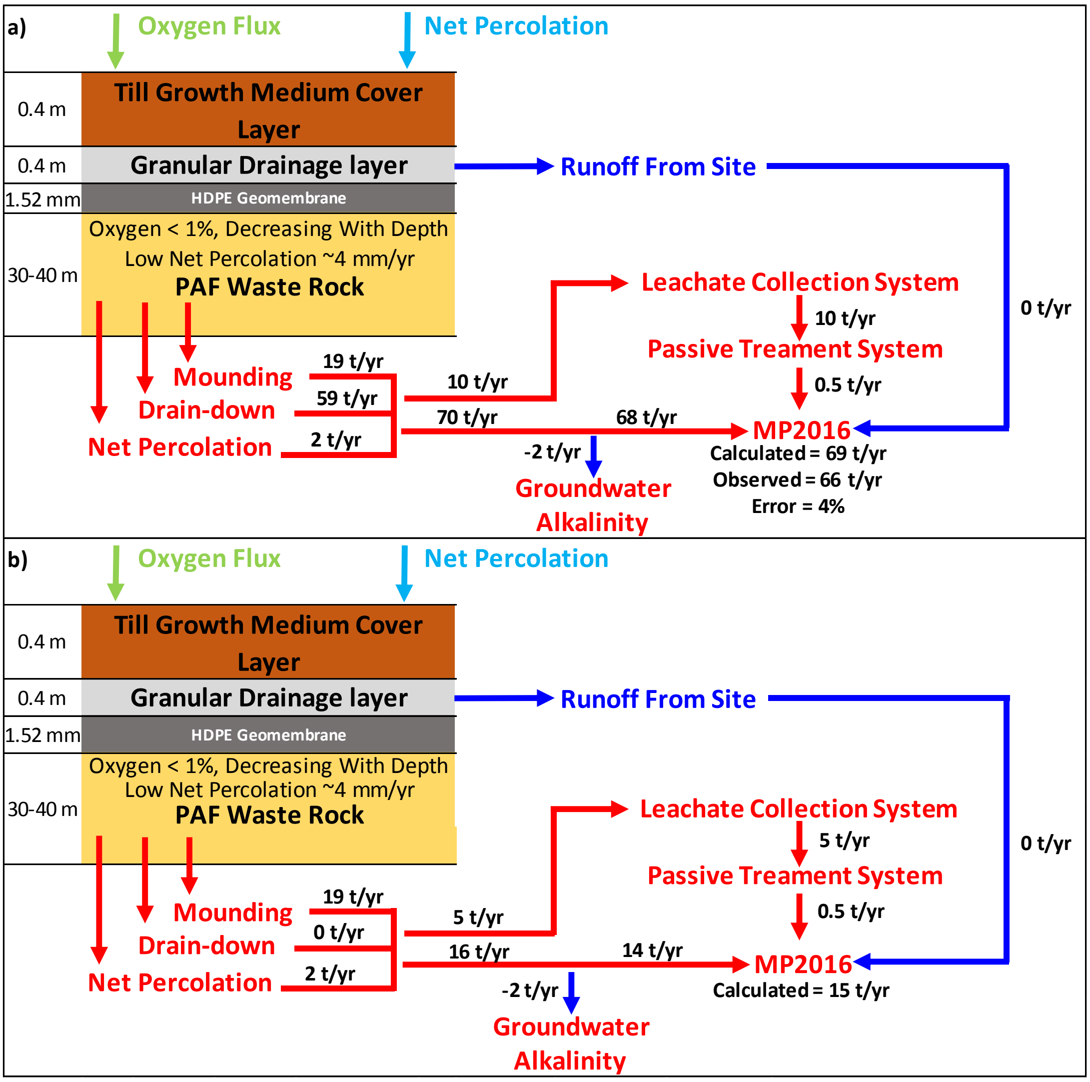


**Figure 7** Victoria Junction WRP acidity mass balance: (a) post-cover system, (b) 100 years post-cover system

The mass balance substantially changes after installation of the low flux cover system and remediation of the site, in terms of both the load produced from surface runoff and basal seepage (Figure 7a). Clean runoff from the site resulted in a reduction of approximately 656 tonnes per year. The total acidity generated from the site is reduced from 948 t/year to 80 t/year owing to the transition to passive treatment. However, the wet well and pump-and-treat wells were decommissioned, and the leachate collection system contributes solely to the passive treatment system. Acidity collected in the leachate collection system would be attributed to basal seepage and groundwater mounding.

The reduction in net percolation from approximately 350 mm/year to 4 mm/year following placement of the cover system introduces a drain-down component to the mass balance. As identified in the flow model, drain-down is currently approximately 75 mm/year. The contribution from groundwater mounding remains unchanged, given the total stored acidity in this region of the WRP has not been lost. Approximately 13% of basal seepage is intercepted and treated before being discharged. The total calculated acidity load at MP2016 is 69 t/year, compared to the current observed load of 66 t/year.

While total acidity generated from the site was reduced by approximately 90% (948 to 80 t/year), comparatively the acidity load at MP2016 decreased by approximately 56%. The pre-cover system (Figure 6) and current (Figure 7a) mass balances provide context for the observed water quality at MP2016 in that a proportional decrease in loading was not observed after changes in water collection and treatment that included acidity loading from the decommissioned wet well and pump-and-treat-wells. Although a more significant decrease in loadings at MP2016 was not observed, closure plan objectives are still being met.

Using the mass balances and conceptual model for pre-cover system and current conditions, a mass balance was developed to predict loadings to the receiving environment 100 years post-cover system construction (Figure 7b). It is estimated that saturated-unsaturated drain-down will be completed in another 10 to 20 years and the bulk of water from unsaturated drain-down will have occurred in 100 years; therefore, the acidity load will be negligible. Loading due to mounding and groundwater flow will remain, given the total acidity in this region is expected to be lost over 3,000 years. The total acidity in the remainder of the WRP with net percolation occurring at a rate of 4 mm/year is estimated to be lost over 240,000 years. Contributions to the leachate collection system, and, therefore, the passive treatment system, are anticipated to decrease as drain-down completes. As a result, the load captured by the leachate collection system was reduced to 5 t/year. The main contribution is anticipated to be from groundwater mounding and seasonal fluctuations in groundwater elevation. With further development of the conceptual flow model, loading to the leachate collection system will be adjusted accordingly. The mass balance closed for the 100-year period would suggest a decrease in acidity to 15 t/year, approximately one order of magnitude from current conditions.

This provides context for what benefit the passive treatment system currently provides. Based on the mass balance, it captures approximately 13% of basal seepage, reducing the acidity generated from 80 t/year to 70 t/year. Loadings to the receiving environment without the reduction from the passive treatment system would still be below pre-cover system loadings and site closure objectives would still be met. The load captured by the passive treatment system is anticipated to decrease as unsaturated-saturated drain-down completes. A strategy for decommissioning the passive treatment system may be to maintain its operation for 10–20 years until unsaturated-saturated drain-down is complete and further improvements to the wetland are observed.

Although these mass balances were completed during the post-closure phase, it is evident such an analysis would be beneficial during the initial phases of operation. A mass balance completed within the closure planning phase could be used to provide information about the cost/benefit of implementing various cover system designs. For example, in this instance, a moisture store-and-release cover system would have reduced the demand on the water collection and treatment system through the associated clean runoff. This highlights the importance of developing a conceptual flow model and acidity mass balance (sources and sinks) to inform management decisions in mine closure planning.

### 3.4 Geochemistry conceptual model validation

Although the acidity mass balances closed with small error, greater confidence in the results can be achieved by using multiple lines of evidence to support the conceptual model. Basal seepage in the conceptual mass balances is represented in Figure 8 as load versus flow. In general, the trend closely follows model 1, where load is proportional to flow. As flow from the WRP is reduced, there is a corresponding decrease in load, which demonstrates the benefit of installing the cover system from the perspective of loading.

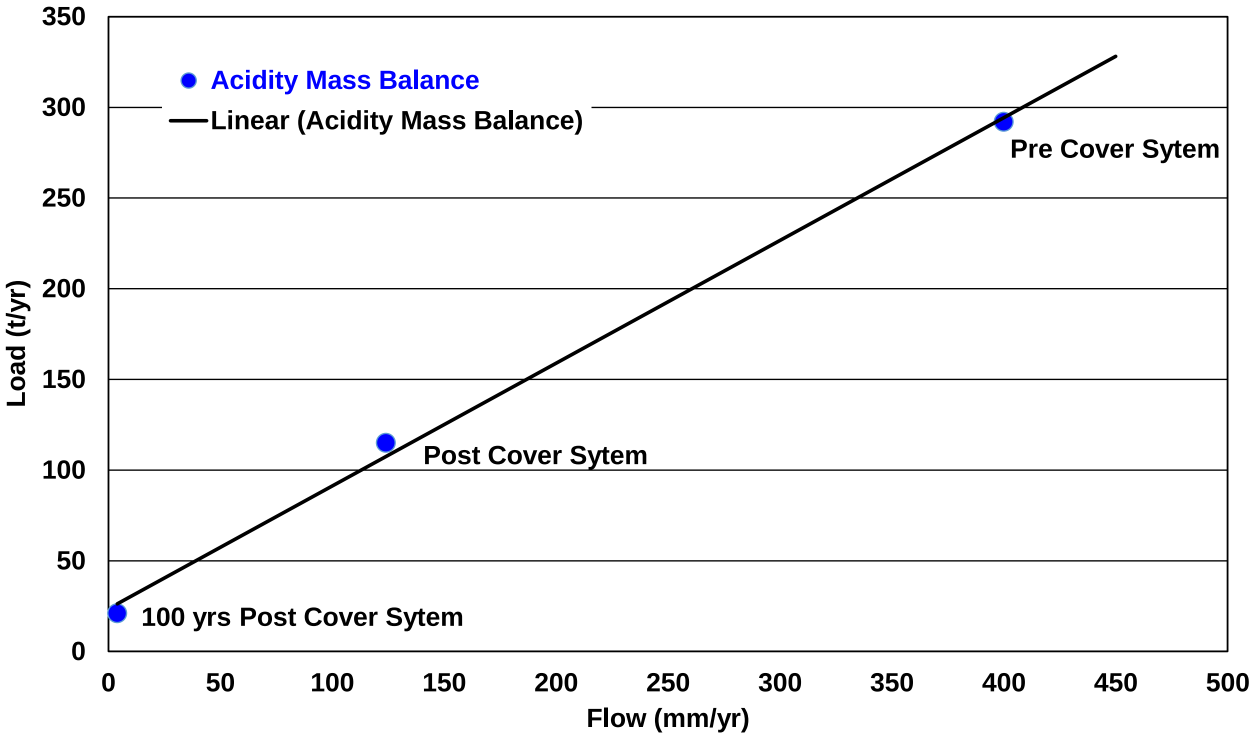


**Figure 8 Load versus flow used in the conceptual model for calculating acidity through basal seepage**

As discussed earlier, loading from the pile can be quantified at MP2016, and a plot of load versus flow can be generated. It is anticipated that MP2016 will follow a similar trend to the conceptual model for calculating long-term acidity (i.e. model 1) given that MP2016 is a downstream focal point for WRP loading. Figure 9 presents load versus flow observed at MP2016 over the 2010–2014 monitoring period. This period is primarily influenced by basal seepage, as the WRP cover system was installed in 2006 and therefore is comparable to load versus flow from the conceptual model.

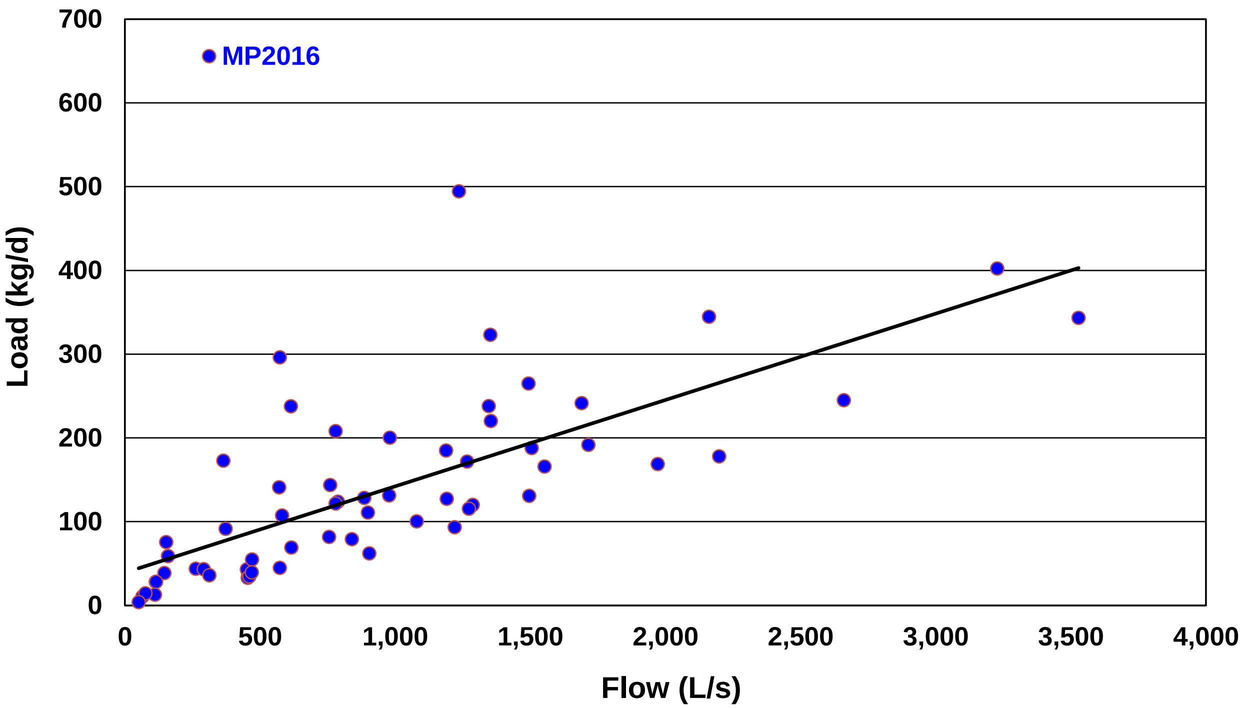


**Figure 9 Observed load versus flow at MP2016**

Load versus flow at MP2016 also follows the general trend of model 1; an increase in load is observed with an increase in flow. There is some spread in the data away from the linear trend line, which is primarily attributed to changes in surface water inputs prior to MP2016. The flow path for MP2016 is approximately 1 km down-gradient of the WRP, and this distribution is likely introduced as a result of various processes occurring in Northwest Brook, including background water levels, recharge, ephemeral flow, drought, spring melt, and biological and other wetland processes.

A considerable amount of time was required to develop the conceptual model for the Victoria Junction WRP. The conceptual model should be considered the primary step in demonstrating a strong understanding of the system, as it provides the basis for moving into more advanced numerical modelling. A well-defined objective for numerical modelling is made clear as part of this process. Without this step, it would be difficult to validate results from the numerical modelling, likely resulting in greater time requirements.

## 4 The cost of water treatment

There are significant initial capital investment and ongoing operation and maintenance costs associated with operating a water treatment plant. The Victoria Junction water treatment plant began operation in the 1970s, was relocated in 1994 and transitioned to a passive treatment system in 2013. Operating costs for water treatment at the site from 2002 to present are presented in Table 2.

**Table 2 ARD/ML water treatments costs**

| Date | Water treated (US gal) | Lime cost | Polymer cost | Electrical and labour costs | Cost/1,000 US gal | Total cost |
|---|---|---|---|---|---|---|
| 2002 | 216,570,000 | C$ 84,420 | C$ 16,000 | C$ 125,000 | 1.04 | C$ 225,420 |
| 2003 | 231,563,000 | C$ 46,920 | C$ 16,000 | C$ 130,000 | 0.83 | C$ 192,920 |
| 2004 | 163,556,000 | C$ 91,580 | C$ 8,000 | C$ 110,000 | 1.28 | C$ 209,580 |
| 2005 | 201,535,000 | C$ 118,650 | C$ 7,000 | C$ 120,000 | 1.22 | C$ 245,650 |
| 2006 | 229,806,000 | C$ 56,700 | C$ 7,000 | C$ 130,000 | 0.84 | C$ 193,700 |
| 2007 | 120,327,000 | C$ 40,320 | C$ 4,000 | C$ 100,000 | 1.20 | C$ 144,320 |
| 2008 | 155,522,000 | C$ 21,600 | C$ 2,000 | C$ 110,000 | 0.86 | C$ 133,600 |
| 2009 | 123,142,000 | C$ 16,200 | C$ 1,500 | C$ 100,000 | 0.96 | C$ 117,700 |
| 2010 | 67,310,000 | C$ 8,100 | C$ 700 | C$ 80,000 | 1.32 | C$ 88,800 |
| 2011 | 31,137,000 | C$ 2,700 | C$ 200 | C$ 40,000 | 1.38 | C$ 42,900 |
| 2012 | - | C$ 2,700[1] | C$ 200[1] | C$ 40,000[1] | - | C$ 42,900[1] |
| 2013 | ~7,000,000[2] | ~C$ 600[2] | - | ~C$ 1,200 | 0.27 | C$ 1,800 |
| 2014 | ~7,000,000[2] | ~C$ 600 | - | ~C$ 1,200 | 0.27 | C$ 1,800 |

[1] *Estimated based on previous years*

[2] *Estimated based on average flow (AECOM, 2009)*

Placement of the cover system in 2006 significantly reduced the ingress of meteoric water to the WRP and collection of runoff, and subsequently a decrease in treatment costs was realised. Although the total cost of active treatment began to decrease, there was an increase in cost per unit during 2010 and 2011. The active water treatment system becomes less efficient at lower volumes, as electric and labour costs remain relatively high. The transition to the passive treatment system reduces the total cost of treatment and the per unit cost dramatically. The savings realised ultimately correspond to the significant decrease in electric and labour costs and decrease in volume of water treated. A decrease in load was observed at MP2016 despite the decrease in water treatment, suggesting that the reduction in load is due to the reduction in flow (i.e. model 1).

A long-term cost projection was completed for a 100-year period to include two scenarios: (1) the reclaimed site with cover system and passive water treatment, and (2) uncovered WRP with active water collection and treatment in perpetuity. Scenario 1 predicts cost based on the actual site reclamation, including placement of the cover system and the progression from active to passive water treatment. Predicted costs are based on actual costs incurred during site progression and consider an inflation rate of 2%. Expected costs in scenario 2, had the cover system not been installed, were developed based on average water treatment costs over a three-year period prior to placement of the cover system and consider a 2% inflation rate.

As shown in Figure 10, costs in both scenarios increase at the same rate until placement of the cover system in 2006, when there is a C$ 13 million increase in the cover system scenario. The no cover system scenario continues to increase at the original rate and exceeds C$ 70 million in 100 years. The Increase in the cost of the cover system scenario from 2006 to 2013 reflects actual active treatment costs. Following construction of the passive treatment system, the rate is significantly reduced. Although the cost of the cover system in 2006 is substantial, it is recouped in water collection and treatment costs by 2047, 41 years after construction. The cumulative 100-year costs for scenarios 1 and 2 are approximately C$ 15M and C$ 70M, respectively.

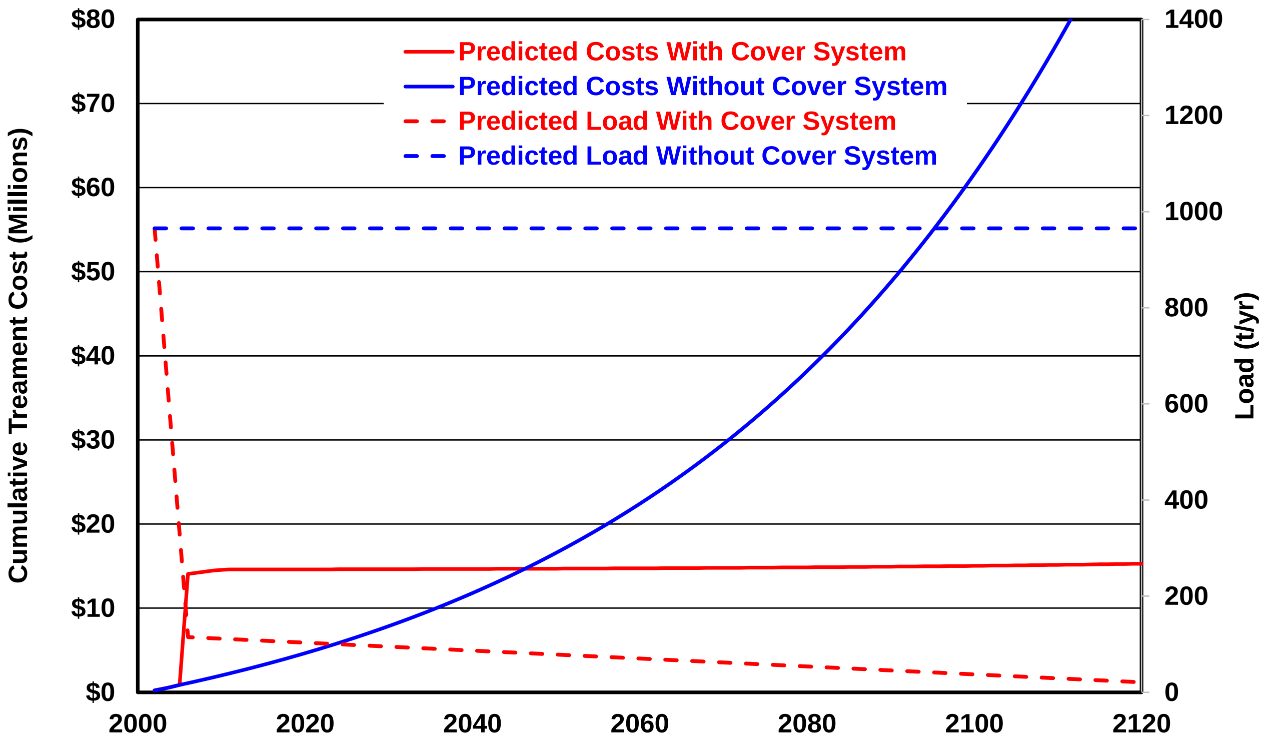


**Figure 10 Predicted costs and load with and without cover system**

While this analysis focuses on water treatment costs alone, factors such as upgrades and maintenance need to be addressed in both scenarios. Given the limited mechanical components of the passive treatment system, maintenance is anticipated to be a small component of ongoing costs. Although the cover system is not a direct component of the passive treatment system, its functioning has direct implications on load to the receiving environment.

The active treatment system has a greater reliance on mechanical systems and is expected to require greater maintenance and upgrades. This system is also greatly influenced by fluctuations in the economy, as it relies more heavily on inputs of lime and polymer.

Risk of failure must also be considered for each scenario, including the probability of occurrence and magnitude of the impact. While the probability of a failure may be difficult to quantify, the impact of a component failure can be developed based on the aforementioned mass balances. In scenario 2, the main component mitigating load to the receiving environment is the active treatment system. A failure of the active treatment system could involve the pump/treat system, runoff from the site or the active treatment system as a whole and result in an additional load of up to 65 tonnes of acidity per month. While a complete system failure may not be likely, it could have a significant impact, and even a low probability of this risk may not be acceptable. The active treatment system inherently carries a greater probability of failure than the passive treatment system, as it has a greater reliance on mechanical systems. The uncovered system is also susceptible to the effect of peak discharges in response to climatic conditions where storage capacity could be exceeded.

A complete failure of the passive treatment system under the reclaimed WRP could result in an additional load of approximately one tonne per month. The impact of this failure is insignificant in comparison to the active treatment system and impacts to downstream receptors. Potential failure of the cover system would potentially occur in isolated regions, allowing risk to be effectively managed. The associated load would be highly attenuated and not result in significant impacts to downstream receptors.

# 6 Conclusions

Through the remediation of Victoria Junction, it has been demonstrated that passive treatment can be achieved to manage residual seepage in the short term while meeting closure plan objectives. An acidity mass balance was key in developing an initial understanding of past, current and predicted loading. It was particularly important for quantifying changes in loading associated with the cumulative effects of net percolation, water mounding, drain-down and progressive changes to the water collection and treatment systems. While a proportional improvement in water quality was not observed at the downstream monitoring location (MP2016) compared to the overall decrease in loading, the acidity mass balance supports the 56% reduction in acidity loading and is a considerable improvement in water quality. The conceptual model was used to demonstrate a strong understanding of the site and provides the basis for moving into more advanced numerical modelling should it be deemed necessary.

While advancements are still required in terms of better predictions of long-term pore water quality, acidity mass balances were closed for the pre-closure and current reclaimed condition using constant concentration (i.e. model 1). The acidity mass balance predicted at 100 years suggests that total acidity will decrease by approximately one order of magnitude from current conditions solely under passive treatment. The acidity mass balances also highlight that a comprehensive conceptual model is beneficial in developing water treatment strategies and meeting closure plan objectives. Acknowledging this provides an opportunity to optimise ARD/ML management in a cost effective manner and inform management decisions. While this was completed in the closure phase, it is noted that there is significant value in such an analysis during the closure design phase to evaluate risks among various closure options.

A wealth of challenges exists in determining if and when a particular site can transition from active to passive water treatment. As demonstrated, a comprehensive conceptual model is fundamental to an effective closure plan and requires adequate information pertaining to the site, including both technical and high-level management. The availability of such information will dictate the ability to develop a comprehensive conceptual model and highlights the need for adequate site study over the long term.

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