APPENDIX A
TECHNICAL REPORTS – PROJECT DESCRIPTION

APPENDIX A.1 TOUQUOY INTEGRATED WATER AND TAILING MANAGEMENT PLAN – IN-PIT DISPOSAL, TOUQUOY GOLD PROJECT

TOUQUOY INTEGRATED WATER AND TAILINGS MANAGEMENT PLAN – IN-PIT DISPOSAL - TOUQUOY GOLD PROJECT

FINAL REPORT

July 2, 2021

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1.0 INTRODUCTION

The Touquoy Gold Project (also referred herein as the "Approved Project") is an open pit gold mine operated by Atlantic Mining NS Inc (AMNS) under Industrial Approval (IA) No. 2012-0824244-08. The Mine Site is located in Moose River, Nova Scotia, approximately 63 km northwest of Halifax and 19 km southwest of Middle Musquodoboit (Figure 1.1). The Touquoy Gold Project has a total ore production of 9.2 million tonnes for the recovery of 0.04 million ounces (oz) of gold. The Touquoy Gold Project started mining operations in October 2017 and has an estimated life of four to six years. Pending regulatory approvals, the life of the Touquoy Mine Site will be extended to process ore from other satellite surface mines proposed by AMNS (Beaver Dam, Fifteen Mile Stream, and Cochrane Hills) which are currently at various stages of planning and regulatory review.

AMNS is proposing modifications to the Approved Project to support ongoing operation at the Mine Site. These modifications include: use of the exhausted Open Pit for tailings disposal instead of the existing approved tailings management facility (TMF); expansion of the Waste Rock Storage Area (WRSA); expansion of the Clay Borrow Area; and relocation of the Plant Access Road used to access the Mill Facility, run-of-mine stockpile, warehouse, truck shop, and several administration buildings (referred to as the "Plant Access Road"). These proposed modifications, which constitute "the Project" currently undergoing environmental assessment (EA), will increase the current approved development area, or, in the case of the in-pit tailings disposal, present a new activity not previously assessed in the original EA process for the Touquoy Gold Project conducted in 2007 (CRA 2007a; CRA 2007b).

This report summarizes the water and tailings management plan, including Touquoy tailings deposition and the integrated mine site water balance, in support of the environmental impact statement screening document for the Touquoy Gold Project.

This report is divided into four sections:

- **Section 2.0 Operational Water Management Plan** – outlines the sources of reclaim and make up water during the processing of Touquoy ore at the Mill Facility, manage site runoff, seepage and other flow components.
- **Section 3.0 Conceptual Tailings Deposition Plan** – outlines the tailings deposition methods based on subaqueous deposition, considering seasonality.
- **Section 4.0 Water Quantity Balance** – outlines the predictions of water volume discharged to the exhausted Touquoy pit, water volume available for reclaim in the Touquoy Tailings Management Facility (TMF), required freshwater make-up from Scraggy Lake, and the timing of when water could be reclaimed from the exhausted Touquoy pit rather the Touquoy TMF.
- **Section 5.0 Water Quality Balance -** outlines the predictions of water quality in the pit lake and effluent discharge to Moose River.

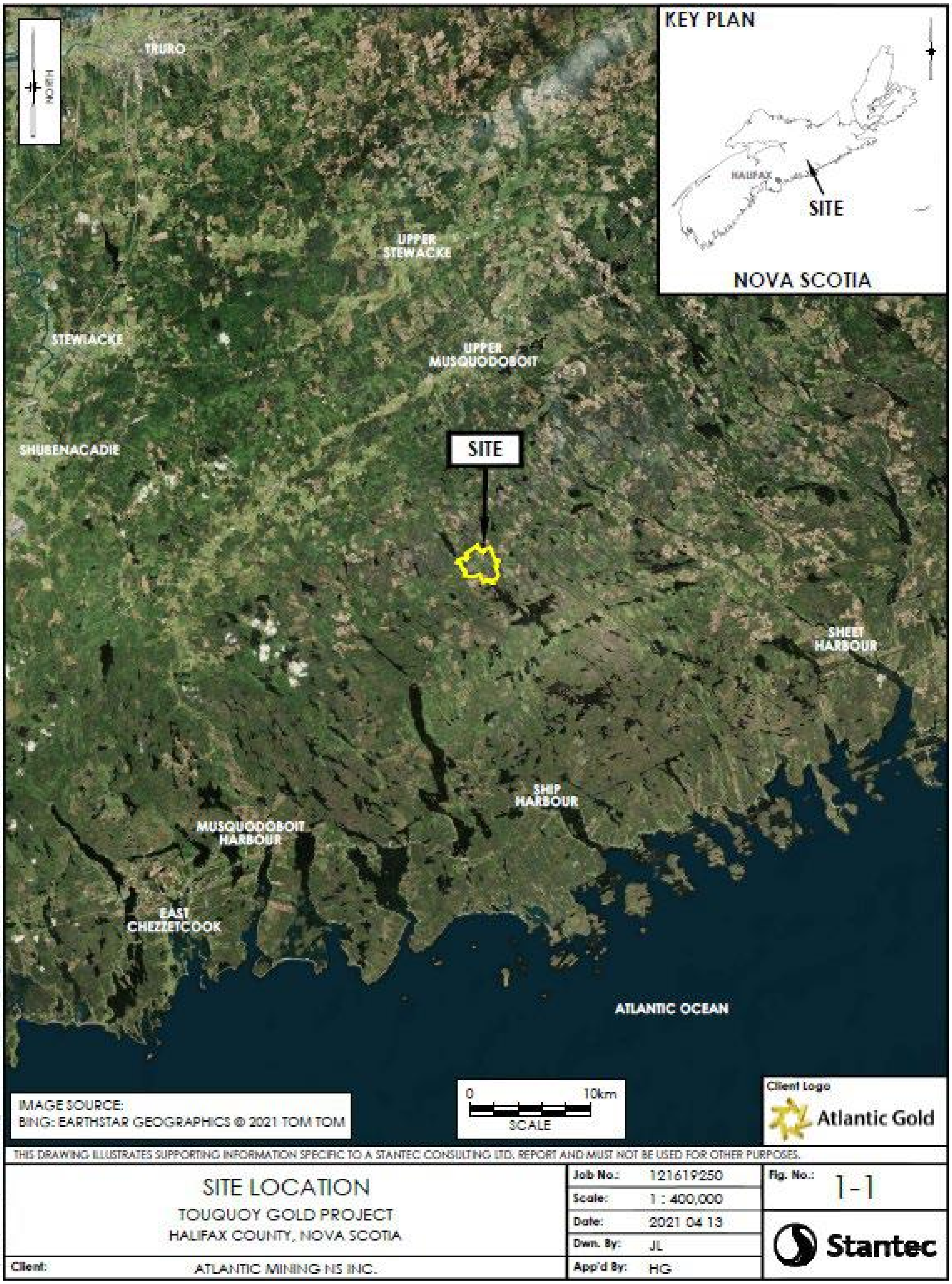


Figure 1.1 Site Location Plan

2.0 OPERATIONAL WATER MANAGEMENT PLAN

Components of the operational water management plan at the Touquoy Mine Site, including the existing mill site, TMF, effluent treatment plant, and the ultimate extent of the exhausted Open Pit are depicted on Figure 2.1. Water management at the Touquoy Mine SIte is described in more detail in the water management plan (Stantec 2017a) and the Water Balance Report (Stantec 2016), excluding integration of the use of the exhausted Open Pit for tailings deposition. Figure 2.1 also illustrates the direction of flow between components, effluent discharge locations, mine component drainage areas, and locations of MDMER final discharge point(s). The MDMER final discharge point for Touquoy operations is located at SW-14 at the outlet of the Touquoy TMF polishing pond that ultimately drains to Scraggy Lake. Routine water quantity and quality monitoring is conducted to satisfy MDMER, inform water management at Touquoy, and identify project effects throughout operation as required in the Industrial Approval issued by Nova Scotia Environment and Climate Change (NS ECC).

An overview of key features of the Touquoy water management plan for the Touquoy Gold Project are provided in the Table 2.1. Water management is presented by project phase (Operation, Reclamation, and Closure) as it pertains to the Approved Project and the Project (i.e., modifications to the Approved Project). Consistent with the Approved Project, the Project manages site drainage by directing contact runoff to the tailings management facility, with the exception of WRSA runoff that is locally treated and drained to Watercourse #4 to result in a no net loss of flow. The exhausted Open Pit t will become the tailings management facility for the Project.

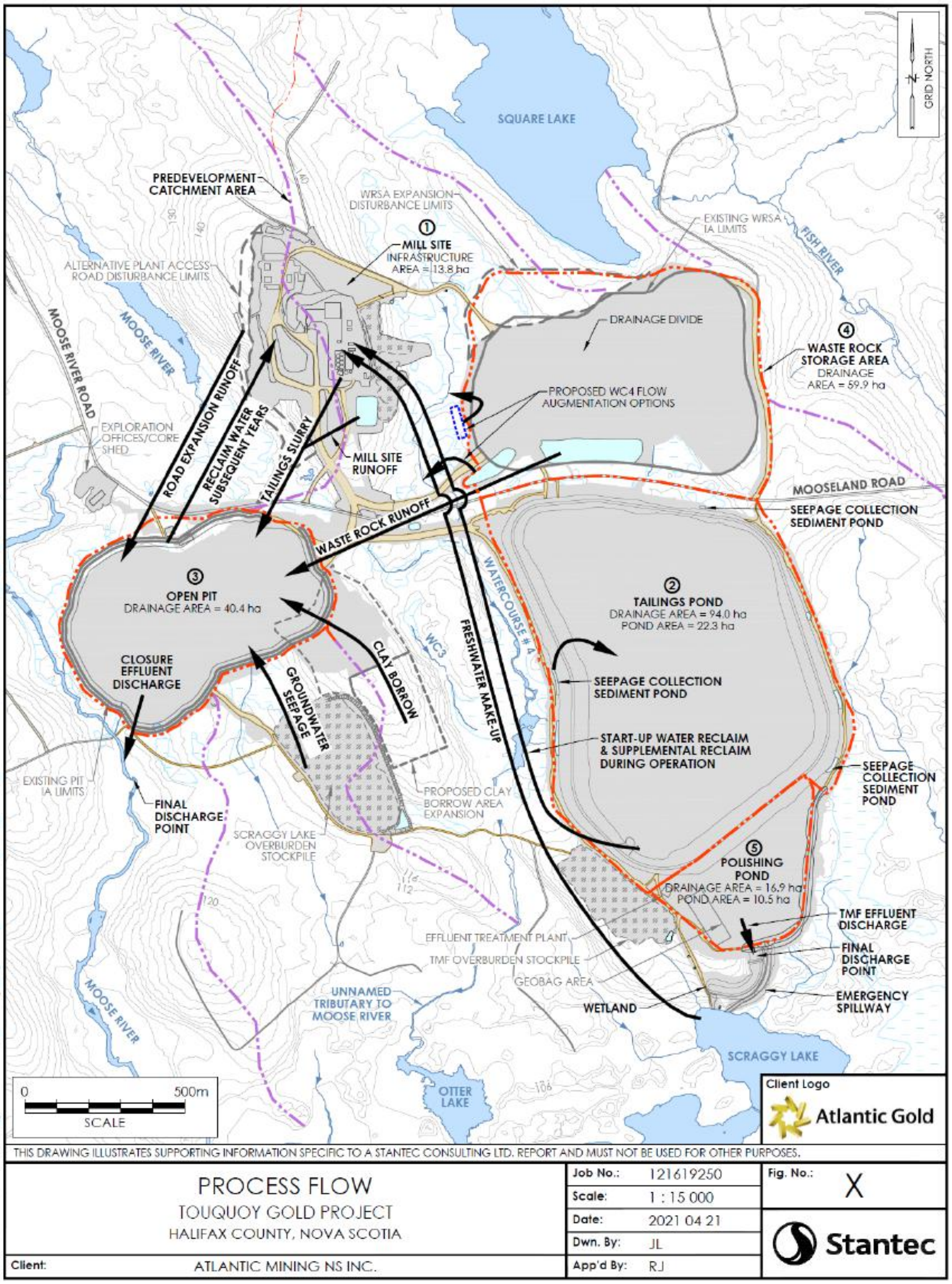


Figure 2.1 Process Flow of Major Mine Site Components at Touquoy

Table 2.1 Touquoy Operational Water Management Plan

<table>
<tr><th>Approved Touquoy Gold Project</th><th>Proposed Modifications (the Project)</th></tr>
<tr><td colspan="2">The operational phase of the Project corresponds to the period of tailings disposal in the Touquoy pit.</td></tr>
<tr><td>WRSA

• Collect WRSA runoff and toe seepage in perimeter ditches that drain to east and west ponds and pumped to TMF</td><td>WRSA

• Expansion of WRSA of 7 ha
• Collect WRSA runoff and toe seepage in perimeter ditches;
– drain west pond to Watercourse #4 to result in no net change to runoff and
– drain east pond to Touquoy pit for accelerate filling</td></tr>
<tr><td>Mill Facility

• Main process water supply from TMF
• Freshwater make-up process water supply from Scraggy Lake within the existing water withdrawal approval limits
• Mill throughput of 8300 tpd
• Collect mill site runoff and manage in mill site pond that discharges in tailings slurry line</td><td>Mill Facility

• Maintain Touquoy ore processing consistent with the existing operation with respect to slurry density, and mill site run-off added to tailings slurry line. Increase mill throughput for processing low grade ore.
• Main process water supply from Open Pit
• Freshwater make-up process water supply from Scraggy Lake within the existing water withdrawal approval limits
Additional process water required in a dry year or to build a reservoir in case of a dry year will be sourced from the polishing pond or Scraggy Lake, subject to NS ECC approval
• Capture road expansion runoff and gravity drain to Touquoy Pit to accelerate filling</td></tr>
<tr><td>TMF

• Tailing's disposal in the TMF
• Process water reclaim from the TMF as a closed loop
• Management of pond levels through routine operation of the effluent treatment plant and downstream facilities
• Collection of WRSA surface runoff, direct precipitation, pit dewatering and TMF toe seepage</td><td>TMF

• Involves the continued use of Touquoy water management facilities</td></tr>
</table>

Table 2.1 Touquoy Operational Water Management Plan

Approved Touquoy Gold Project	Proposed Modifications (the Project)
Open Pit • Dewater the Open Pit to the TMF until open pit mining is complete • Capture runoff that drains from the existing Clay Borrow Area, pumped runoff from the Scraggy Lake Overburden Stockpile, and groundwater inflow of 813 m^3/d	Open Pit • End-of-Pipe deposition of tailings in the OpenPit, via a 16-inch HDPE pipeline • Cease dewatering of the Open Pit • Allow water to accumulate in the pit to accelerate pit filling • Capture natural runoff to the Open Pit, direct precipitation, groundwater inflow (813 m^3/d below 104 m and 408 m^3/d at elev. 104-108 m) runoff from the WRSA, Scraggy Overburden Stockpile and the expanded Clay Borrow Area and tailings deposition • After initial reclaim from the TMF, reclaim water from the exhausted Open Pit as a closed loop, reclaimed through a floating barge that will raise with the water and tailings elevation in the Open Pit, and a reclaim pipeline from the pit to the mill • Maintain a minimum of 1.75 m water cover above the deposited tailings at the reclaim to both facilitate operation of the barge pump and limit the resuspension of tailings solids. The water cover depth varies over the tailings depositional period to limit resuspension of tailings. This minimum water cover is maintained at the Approved Project without issues of resuspension of tailings particles. • Treat pit lake throughout operation as the pit fills as a batch reactor with the objective of adjusting the pH to precipitate metals thus improving discharge quality
When the Touquoy pit is exhausted of ore and the Touquoy TMF has reached its tailings storage capacity, reclamation activities will commence for the Touquoy TMF and downstream discharge facilities (i.e., the geobags, polishing pond, and constructed wetland).	
WRSA • Cover and vegetate pile • Restore pre-development drainage patterns	WRSA • No change to reclamation associated with the Approved Project
Mill Facility • Cease ore processing and commence mill decommissioning • Restore pre-development drainage patterns	Mill Facility • No change to reclamation associated with the Approved Project

Table 2.1 Touquoy Operational Water Management Plan

<table>
<tr><th>Approved Touquoy Gold Project</th><th>Proposed Modifications (the Project)</th></tr>
<tr><td>TMF

• Continue to collect surface runoff from the seepage collection ditches, and direct precipitation and discharge from the TMF treated in the effluent treatment plant, and from the WRSA until infrastructure is in-place to drain to the Open Pit.
• Maintain perimeter ditching to capture toe seepage from the TMF and waste rock storage area until water quality meets reclamation regulatory water quality requirements as described in the reclamation plan for Touquoy (Stantec 2017b).</td><td>TMF

• No change to reclamation associated with the Approved Project</td></tr>
<tr><td>Open Pit

• Allow the exhausted Open Pit to naturally fill overtime with water from direct precipitation, surface runoff and seepage
• Overflow/spill at elevation 108 metres (m) relative to the Canadian Geodetic Vertical Datum of 2013 (CGVD2013), approximately 2 m below the lowest Touquoy pit elevation to prevent overtopping
• Spill to Moose River via a conveyance channel designed to accommodate the inflow design flood in accordance with the Canadian Dam Association (CDA) guidelines
• Establish Final Discharge point approximately 70 m downstream from the SW-2 monitoring station on Moose River for the Touquoy pit closure (Figure 2.2).
• Maintain roadway access to the pit following the construction of the spillway/conveyance channel
• Maintain the top of tailings 4m below the spillway elevation</td><td>Open Pit

• Maintain water levels in the pit below the spill elevation of 108 m until water in the pit lake meets MDMER discharge limits
• Treat surplus water in the Open Pit or pump and treat in adjacent treatment plant or use of the existing Touquoy effluent treatment plant at a rate of approximately 400 m^3/hr
• Maintain the top of tailings 2m below the spillway elevation to protect the bed sediments from disturbances due to wave action and ice entrainment (i.e. approximately 10% deeper than the maximum ice thickness, MEND 1998)</td></tr>
<tr><td colspan="2">Once water quality meets regulatory reclamation criteria without treatment, the site is prepared for closure, in accordance with the Touquoy reclamation and closure plan.

• Breach the existing polishing pond and wetland dams, drain the ponds and contour and revegetate the entire area, retiring the final discharge point. This will result in reduced water surplus from the TMF.
• Allow surplus water in the exhausted Touquoy pit to discharge via the proposed spillway/conveyance channel (see Figure 2.2) to Moose River, subject to meeting regulatory discharge criteria
• Continue monitoring the existing Final Discharge Point (SW-14) in scraggy Lake downstream of the TMF and in Moose River downstream of the pit spillway
• Closure of the effluent treatment plant, as it is no longer required.</td></tr>
</table>

A start-up process water supply is planned for the expanded project, to account for water contained in the consolidated tailings mass (i.e. lock-up) and to store excess water in the pit to make-up process water deficits in a dry year. As summarized in Table 2.2, the start-up water supply will be sourced from accumulated water in the pit prior to reclaim from the pit and the available pond storage in the TMF, WRSA and Polishing Pond. Throughout processing, the process water supply will continue to be supplemented from water that accumulated in the TMF and polishing pond, and the collected runoff from the Mill Facility, Plant Access Road, Scraggy Lake overburden stockpile, Clay Borrow Area and the east WRSA runoff and toe seepage.

Table 2.2 Start-Up Process Water Supply

Start-Up Water Supply	Assumption (m^3)
Accessible process water reclaim from the TMF until the TMF pond is depleted	262,000
Allow water to accumulate in the lowest bench (i.e. Phase I pit) once mining is completed in this area in September 2021.	188,000
Stop pit dewatering once mining is completed in August 2022	0
Drain WRSA ponds to Open Pit prior to reclaim from Open Pit	21,000
Drain Polishing Pond to Open Pit over the first few months of Processing	75,000

The Project reclamation and closure is consistent with the reclamation and closure plan for the Approved Project, the exhausted Open Pit fills with water overtime and is allowed to spill through a proposed spillway/conveyance channel to Moose River. The location of the spillway and conveyance channel is depicted on Figure 2.2. However, as the exhausted Open Pit will contain both water and tailings under the Project, the pit lake can be treated throughout reclamation as the pit fills and until the pit lake meets the reclamation regulatory water quality requirements or site-specific criteria.

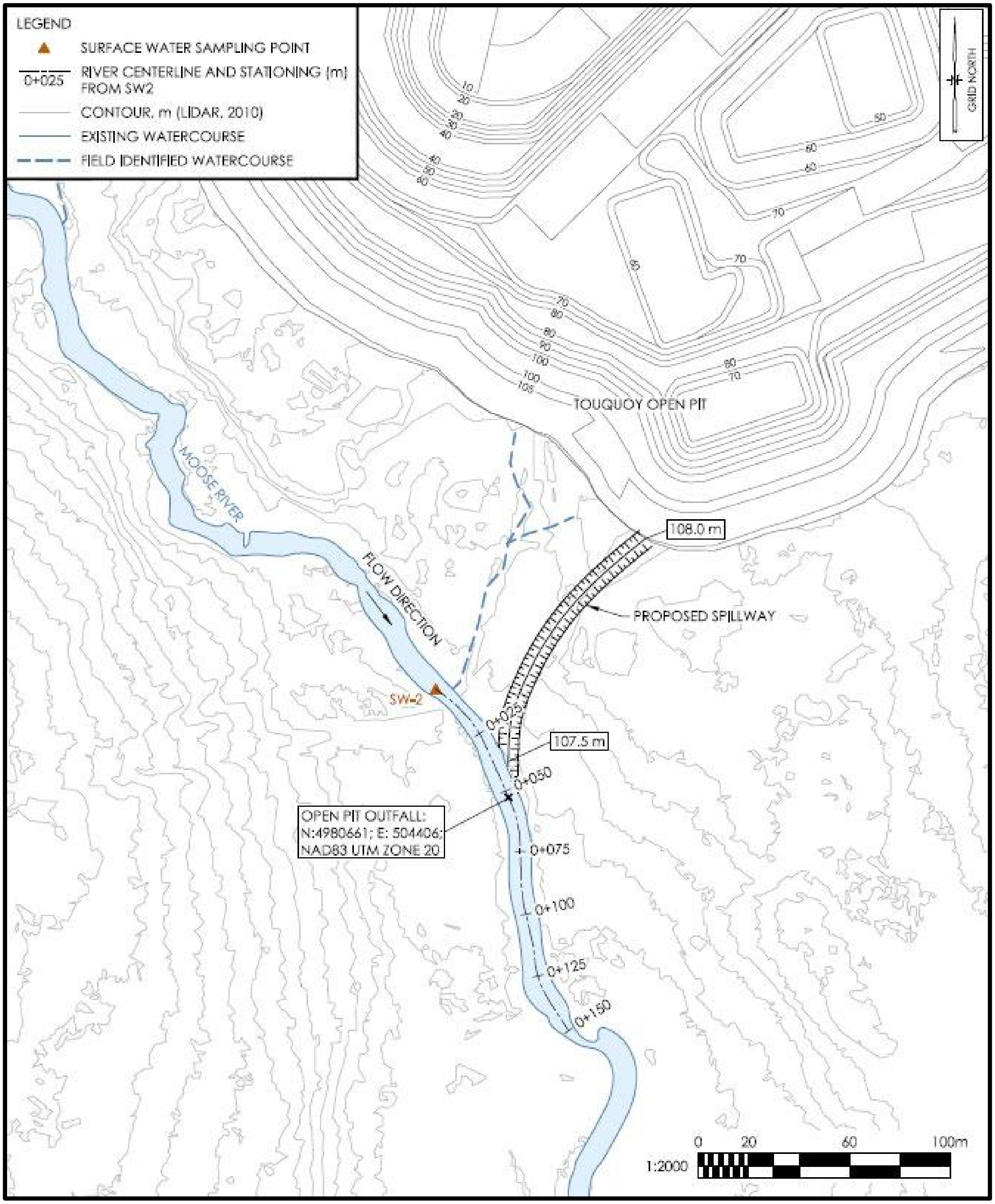


Figure 2.2 Location of Exhausted Touquoy Pit Outfall

3.0 CONCEPTUAL TAILINGS DEPOSITION PLAN

This section presents a conceptual plan for deposition of conventional tailings slurry into the exhausted Open Pit from Touquoy ore processing. The total capacity of the expanded Open Pit at the proposed spillway elevation of 108.0 m is 12.276 million cubic metres (Mm^3) is sufficient to store Touquoy low grade ore processing tailings using subaqueous (i.e., in water) deposition. Considering subaqueous deposition, the exhausted Open Pit can accommodate the estimated tonnage of 6.5 million tonnes (Mt) from Touquoy ore processing.

The use of the exhausted Open Pit dispose tailings eliminates the need to expand the surface TMF and provided permanent disposal which eliminates need for long term care and maintenance of a tailings dam. As described by MEND (2015), the advantages of in-pit disposal include:

- Reduction in acid generation and metal leaching through the development of a water cover
- Permanent physical isolation of tailings mine waste
- Minimization of the need for engineered control systems and long-term monitoring
- Stabilization of pit walls
- Elimination of potential accidental release of solids
- Earlier return of the land to previous and traditional uses

3.1 TAILINGS DEPOSITION METHODOLOGY

Tailings may be chemically and physically engineered and deposited as a thickened slurry that consolidates as a relatively impervious material (relative to the pit surround). As the capacity of the Open Pit is adequate for tailings deposition and the Mill Facility is set-up to dispose of tailings in a slurry, tailings slurry alternatives such as high-density tailings and paste were not considered. The usual in-pit disposal strategy for tailings is to discharge the tailings directly into the pit as a slurry. The tailings can be discharged into the Open Pit without modifying the tailings chemical or physical properties from the current operation. The tailings will settle and consolidate, with excess water becoming a water cover. This water may be drawn off to be recycled to the mill or treated and discharged when the Open Pit fills to its overflow elevation.

As shown in Figure 3.1, the exhausted Open Pit bottom elevation of -25.0 m to the spillway elevation of 108.0 m, has a total depth below the spillway of 132 m. The exhausted Open Pit has a conical shape, reducing in area as the pit gets deeper. The eastern side of the pit has small plateaus and basins in the pit, where the western pit wall has a much steeper wall. Based on the Open Pit geometry development of a tailings beaches in the pit is not practical.

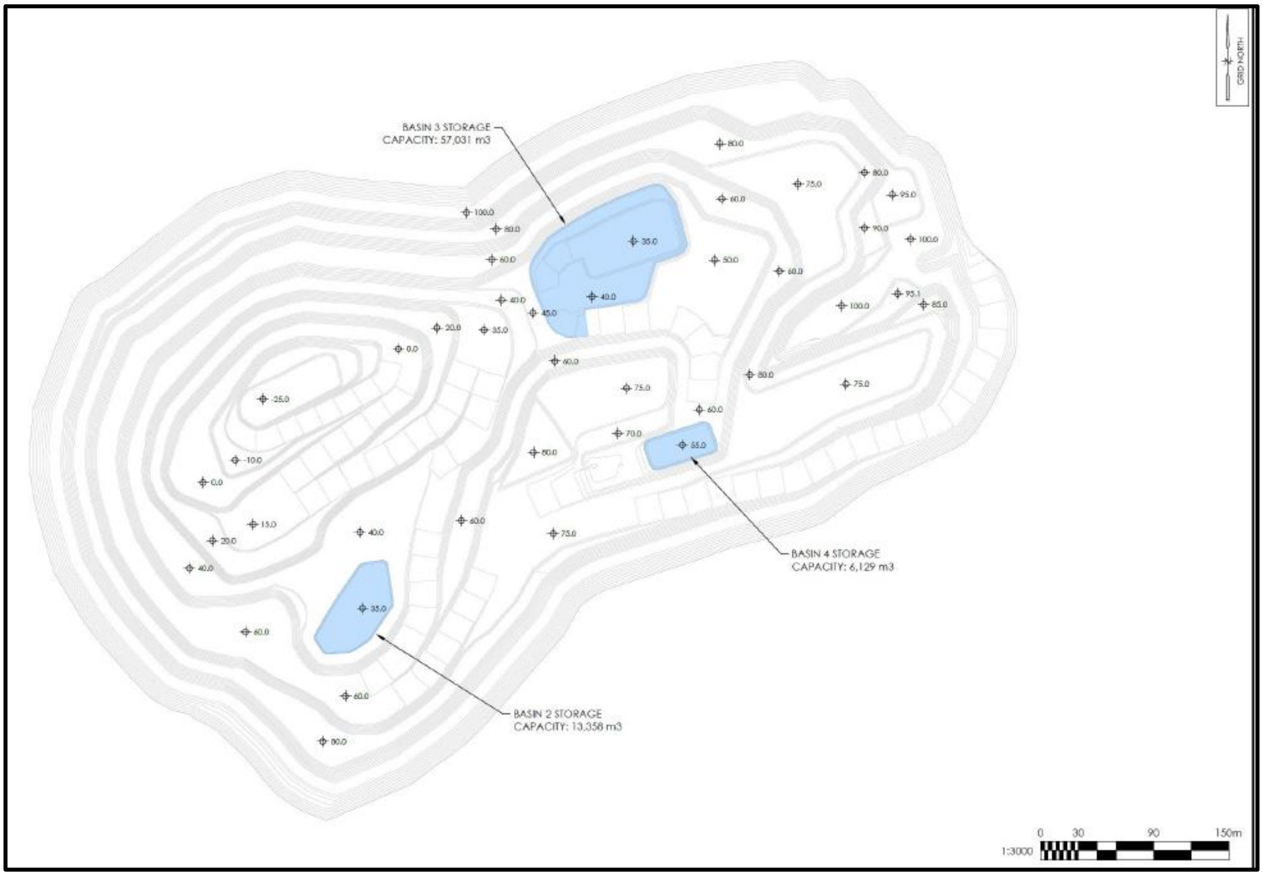


Figure 3.1 Touquoy Pit Dimensions

The tailings placement techniques considered for the Touquoy in-pit disposal included subaerial discharge and subaqueous injection of tailings below the tailings surface. Subaerial tailing deposition could be by end-of-pipe 10 meter down the pit face thus limiting the amount of infrastructure required. This method would result the formation of a turbulent zone and the resuspension of tailings particles, however, could be segregated from the reclaim line through floating baffle curtains. This method is prone to the formation of tailings piles above the water column that are susceptible to ice formation from winter exposure. The principal concern about residual ice is that it remains in the tailings thereafter and may result in the physical instability arising from ongoing consolidation due to melting and the release of tailings porewater.

Subaqueous placement reduces worker exposure to dust, provides for less tailings segregation and lower hydraulic conductivity in consolidated tailings. Depositing the tailings using a tremie pipe suspended from a floating barge will allow for tailings to be deposited under water maintaining the water cover by moving the pipe radially around the pit. Subaqueous tailings deposition under a water cover is the chosen method of disposal at Touquoy. Deposition of tailings under a water cover will limit particle segregation, maximize consolidation and prevent frost lens formation in tails that may pile above the water line or in the winter ice formation zone. A water cover inhibits further oxidation of sulphide minerals and acts as a barrier to the diffusion of atmospheric oxygen to the submerged sulphides (MEND 2015). Although the tailings are expected to be low metal leaching and non-acid generating, a water cover is expected to further improve water quality in the pit lake.

As commonly experienced with many pits when a pit lake forms, pit walls that are stable during mining can be destabilized as a result of increased phreatic pressures in the pit walls as water levels rise. Drilling programs within the pit greatly aided the monitoring and adjustment of the disposal of tailings and addressing the formation of ice lenses.

Quality of reclaim water will need to meet criteria for total suspended solids, residual reagents and other parameters to limit fouling or reduced recoveries in the Mill Facility. These criteria will be refined in subsequent phases of study to determine if additional treatment of reclaim water will be required.

In general, spring, summer and fall operation is more flexible than winter (frozen) operation, and appropriate planning and mitigation is required to prevent potential issues with respect to maintaining minimum capacities during frozen conditions.

3.2 NORMAL OPERATION (SPRING, SUMMER AND FALL)

Tailings will be transported to the Open Pit as thickened slurry via a tailings pipeline that runs from the Mill Facility to the exhausted pit. To facilitate deposition, the existing tailings slurry pipeline from the mill will be redirected from the TMF to the exhausted Touquoy pit. Secondary containment is achieved by running the main tailings pipeline in a lined ditch. The tailings will be deposited into the pit by end-of-pipe discharge, beginning in the lower areas and moving radially around the exhausted Touquoy pit. The tailings discharge pipe will be suspended in the pond by a floating barge. Initially, the pipe will likely discharge from surface at a lower bench as the bottom of the exhausted Touquoy pit has a deeper basin (i.e. Phase I). Detailed procedures will be developed for tailings line relocation and corresponding plant shutdowns to prevent plugging of the tailings pipeline.

Summer deposition will be carried out in shallower portions of the pit in preparation for the winter. Bathymetric surveys are conducted at least once a year during the ice-free period to identify areas where tailings deposition should be concentrated and to create a tailings surface. From the tailings surface, design assumptions of tailings volume and average tailings deposited density can be checked. A check that capacity is available in deeper parts of the exhausted Open Pit to prepare for winter operation is conducted through routine updates of the tailing's deposition plan. Winter deposition will occur in deeper zones to avoid beaching of deposited tails where ice lensing has been reported to occur at other Canadian open pit mines (ARCADIS, 2015).

The reclaim barge will remain in the TMF pond supplement process water to the Mill Facility. A second reclaim barge will be placed in the pit in an area with the highest water depth to avoid intake entrainment of tailings and account for winter sequestration of water in ice cover over the pit pond. A floating baffle curtain will be installed around the barge should high suspended solids/turbidity become an issue in processing. Methods to reduce icing-in of barges will be considered in detailed design such as continuous air bubbler systems.

Pertinent considerations and design criteria have been collated in Table 3.1. The assumptions presented in this water management plan should be updated with reported values when the final deposition plan is prepared. As presented in the Stantec (2018a), typical of a hard rock gold mine the grain size distribution of the tailings are silt (ASTM D2487, 2017), with specific gravity ranging between 2.76 and 2.86. An average settled tailings density of 1.3 tonnes per cubic metre (t/m³) was assumed considering subaqueous tailing deposition, thus a lower average deposited tailings density than that of the Touquoy tailings pond of 1.44 t/m³ practicing sub-aerial deposition. Based on the relationship between specific gravity over void ratio plus 1 to estimate tailings density, a density of 1.3 t/m³, specific gravity of 2.76 would assume a void ratio of 1.1. Further consolidation decreasing the void ratio is expected overtime and a lower initial tailings density of 1.2 is expected for the first 1 to 3 months.

The minimum depth required during operation to maintain a stable sediment bed within a tailings pond was assumed as 1 m (MEND 1998). However, the water depth over the tailings is dependent on the operation of the barge pump. The existing water cover at the barge pump used for the Touquoy TMF is 1.75 m and was assumed to be consistent for the Project. A water cover will reduce the potential for oxidation and leaching of metals, increase tailings consolidation, reduce the hydraulic conductivity thus reducing the potential to create a groundwater preferential pathway through the tails and eliminate dust generation from exposed tails. Once experience is gained during operation, the final depth of water cover during closure should be validated to limit entrainment of tailings particles in ice, reduce re-suspension of tailing particles by wind/waves and the upgradient flow of porewater during tailings consolidation. In addition, further modelling will be conducted to estimate the required water cover.

Table 3.1 Project Tailings Deposition Assumptions

Criteria	Value	Unit	Source
Tailings Characteristics			
Average settled tailings density	1.3	t/m³	
Specific gravity	2.83	---	Stantec 2018
Saturated water content (% of tailings production (tonnes))	36.1	%	Calculated parameter
Exhausted Touquoy Pit Characteristics			
Touquoy pit volume at spillway elev. (108.0 m)	12.275	Mm³	Ultimate Pit Design April 2021 (AMNS 2021)
Pit lake freezes over	December	month	Existing condition
Pit lake ice melts	April	month	Existing condition
Closure spillway elevation	108	m	Design condition
Minimum water depth – effective reclaim pump operation	**1.75**	**m**	Existing condition
Assumed Freeboard Requirements of Touquoy pit	**1**	**m**	
Design Storm (1:1000 year storm of 193 mm plus snow melt)	105,700	m³	

Note: Blank fields indicate an estimate or assumption as part of this study

3.3 WINTER (FROZEN) OPERATION

Based on a review of climate normal temperatures, ice cover in the pit lake may occur as early as December. Subaqueous deposition employed in cold climates require mitigation strategies to continue deposition when the water surface is frozen. Bubbler systems can be installed around the discharge/reclaim barge and its pontoons to reduce ice formation. The discharge/reclaim barge will be placed over a deep portion of the pond to provide storage of tailings deposited throughout the ice-covered portion of the winter. Another option is to submerge the tailings slurry discharge line below the ice depth to discharge tailings to a single point. Specific in-pit depositional details will be determined during detailed design considering climate factors, standing water depth, pit water surface area, reducing opportunities of differential settlement within the tails, maximizing tailings consolidation and increasing hydraulic conductivity.

4.0 WATER BALANCE MODEL

A preliminary water balance model was developed to simulate the overall operational water management of the Project in operation and reclamation. The water balance model was developed using Excel through multiple iteration and revisions simulating construction, commissioning, and operation of ore processing and tailings deposition at the Touquoy mine site to improve accuracy. Using the existing conditions water balance model at Touquoy, the model was extended to simulate the integrated water management of Project ore processing at Touquoy as part of a water and tailings management plan. Model inputs and outputs to the exhausted Open Pit accounted for groundwater inflows and seepage losses, surface runoff, direct precipitation, evaporation, process water, porewater lock up and reclaim to the Touquoy TMF and exhausted Open Pit. The objectives of the water balance model for the Project include to:

- Understand water management adjustments needed to accommodate continued ore processing and tailings deposition
- Simulate the water and tailings volume in the exhausted Open Pit over the life of the Project
- Predict when it would be necessary to withdraw reclaim water from the Open Pit, as opposed to the TMF, under climate normal conditions and
- Integrate mine water supply sources into a comprehensive and practical water management plan that increases water supply storage to account for extended dry conditions and minimum required pump intake depths, and reduces effluent treatment, FDP monitoring requirements, and energy usage.

The model was run for the climate normal conditions in addition to the 1:100 Annual Exceedance Probability (AEP) wet conditions, and 1:100 AEP dry climate conditions (assuming groundwater inflow and storage in the Touquoy pit) for the during of in-pit disposal operation, reclamation to closure.

The model was run for in-pit disposal period commencing in August 2022.

4.1 BASELINE HYDROLOGY

4.1.1 Climate

Touquoy Mine Site climatic and hydrologic conditions are required for the water balance analysis. Baseline climate and hydrology conditions at the Touquoy Mine Site and relevant data required for water balance analysis are presented in this section.

The climate for the mine site is continental with temperature extremes moderated by the ocean. The coldest temperature recorded was −41.1 °C on January 31, 1920, at Upper Stewiacke (Environment Canada 2015c). Precipitation is well distributed throughout the year. July and August are the driest months on average.

Middle Musquodoboit climate station (ECCC Station ID 8203535), was used to characterize the climatic conditions at the mine site. This station is located approximately 20 km northwest of the mine site, and reports data collected between 1961 and 2011. As presented in Table 4.1, the climate normal precipitation is approximately 1357.7 mm and the average snowfall of 172.2 cm, based on a period of record 1981-2010 (climate normal period, Environment Canada 2015a). The extreme one-day precipitation amount of 173 mm for the period of record of the selected climate station occurred in 1961. Temperatures typically drop below zero between the months of December through March each year.

Average annual lake evaporation is 515 mm for the mine site area based on average lake evaporation at the Truro climate station (Environment Canada 2015b) and corresponding monthly evaporation rates are presented in Table 4.1.

Table 4.1 Representative Climate Values for the Mine Site

Climate Normal for the 30-year period (1981-2010) at Middle Musquodoboit Climate Station													
Parameter	**Jan**	**Feb**	**Mar**	**Apr**	**May**	**Jun**	**Jul**	**Aug**	**Sep**	**Oct**	**Nov**	**Dec**	**Year**
Temperature (°C)	-6.2	-5.2	-1.3	4.4	9.9	14.8	18.5	18.4	14.2	8.5	3.5	-2.4	6.4
Rainfall (mm)	80.4	62.1	92.8	99.5	104.9	99.8	103.8	91.9	110.7	116.7	128.6	97.2	1188.3
Snowfall (cm)	49.4	41.3	31.4	9.5	0.5	0.0	0.0	0.0	0.0	0.0	8.2	31.9	172.2
Precipitation (mm)	129.8	100.5	124.2	109.0	105.4	99.8	103.8	91.9	110.7	116.7	136.8	129.1	1357.7
Snow Depth at Month End (cm)	40	67	64	22	6	1	0	0	0	0	25	28	NA
Monthly Lake Evaporation at Truro Climate Station for 30 year period (1981-2010)													
Lake Evaporation (mm/day)	0	0	0	0	89.9	102	117.8	96.1	69	40.3	0	0	515.1

4.1.1.1 Wet and Dry Years

A frequency analysis was conducted to estimate annual precipitation for various return periods using the Middle Musquodoboit climate station data from 1961 to 2011. Annual precipitation totals for various return periods are presented in Table 4.2, including climate normal, wet and dry year climate conditions. The 100 year return period (1:100) wet and dry annual precipitation amounts are estimated to be 1,831.5 mm and 967.2 mm respectively.

Table 4.2 Annual Precipitation for Range of Return Period Precipitation Events

Return Period	Annual Precipitation (mm)[1]	
	Dry Year	Wet Year
Climate Normal (1981-2010)	1357.7	
5	1179.1	1485.5
10	1111.3	1579.7
25	1043.8	1687.6

Table 4.2 Annual Precipitation for Range of Return Period Precipitation Events

Return Period	Annual Precipitation (mm)[1]	
	Dry Year	Wet Year
50	1002.6	1761.7
100	967.2	1831.5
Note: [1] Based on the average of the period of record of climate station 8203535 Middle Musquodoboit		

Maximum annual precipitation of 1,730 mm occurred in 1972 and approximately equal to the 1:40 year wet annual precipitation. Minimum annual precipitation of 1,073 mm occurred in 1992 and approximately equal to the 1:20 dry annual precipitation. Consistent with the active tailings model forecasting, monthly distributions of the 1:100 year annual precipitation used in the water balance modelling were derived using the monthly distribution trends observed in 1972 for wet years, and in 1992 for dry years.

A summary of the derived wet/dry year monthly climate conditions are presented in Table 4.3 for the 1:100 precipitation events. The mean monthly temperatures for the 1:100 wet year climate conditions are derived from monthly data observed during the wettest year on record (i.e., in 1972). Similarly, the monthly temperatures for the 1:100 dry year climate conditions are derived from monthly data observed during the driest year on record (i.e., 1992). The calculated annual precipitation was allocated by month based on the monthly distribution of the representative climate dry (1972) and wet (1992) years for the Middle Musquodoboit climate station.

Table 4.3 Wet and Dry Year Climate Values for the Mine Site

Dry Year													
	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	
Temperature (°C)	-6.9	-6.1	-4.1	2.6	9.9	15.7	16.2	18.4	14.5	7.8	1.6	-2.6	**Average** 5.6
1:100 Precipitation (mm)	122.0	134.2	126.3	27.9	54.9	41.1	71.9	56.7	59.4	101.0	96.9	74.9	**Total** 967.2
Wet Year													
Temperature (°C)	-5.3	-7.5	-3.6	3.6	13	15.6	17.1	18	14.2	7.5	2.2	-10.7	**Average** 5.3
1:100 Precipitation (mm)	130.2	136.0	237.9	114.0	152.5	140.8	145.7	103.1	69.8	231.9	217.1	152.5	**Total** 1831.5

4.1.2 Environmental Water Balance

The environmental water balance can be represented by the following relationship:

$$P = ET + R + I$$

where: P = precipitation;
ET = evapotranspiration;
R = surface runoff; and
I = infiltration and storage.

A spreadsheet-based monthly water balance model was used for the mine site based on the Thornthwaite and Mather method developed to estimate evapotranspiration, surface runoff, infiltration, and streamflow (Mather 1969, 1978 and 1979; Black 1996).

The spreadsheet model calculates monthly potential evapotranspiration (PET) using the Malstrom (1969) equation and is given by:

$$PET = 40.9 \times ea^*$$

$$ea^* = 0.611 \times \exp [(17.3T)/(T+237.3)]$$

where: PET = potential evapotranspiration (mm/month);
ea* = saturation vapour pressure (KPa); and
T = mean monthly temperature (°C).

Actual evapotranspiration (AET) is derived from potential evapotranspiration and soil-moisture. When P for a month is less than PET, then AET is equal to P plus the amount of soil moisture that can be withdrawn from storage in the soil. If P for a month is greater than PET, then AET is equal to PET. Evapotranspiration was assumed as zero for the lower 75 m elevations of the Open Pit.

Infiltration factors described by the Ontario Ministry of the Environment (OMOE 1995 and 2003) are used to determine the fraction of water surplus (excess of precipitation over evapotranspiration, P-ET) that infiltrates into the ground and the fraction that runs off to the nearby streams. The “infiltration factor” is determined from average landscape topographic slope, hydrologic soil type and vegetation cover type, and is used to determine the proportion of P-ET routed to infiltration. Infiltrated water recharges aquifers and also routes via interflow to waterbodies and watercourses. In the long term all net infiltrated water recharging aquifers is assumed to be discharged as a component of baseflow. An additional line row in the monthly water balance, estimates streamflow which integrates both overland runoff and infiltration routing back to the “stream” as groundwater discharge and interflow components of baseflow.

Although groundwater recharge and groundwater discharge may not balance within the temporal confines of a climate year, in the long-term, all water that recharges groundwater aquifers is assumed to discharge as baseflow to lakes and streams. Therefore, in this case, as all groundwater is assumed to flow in relatively localized groundwater watersheds which are highly correlated to the surface watersheds, all baseflow returns to the local watershed into which its source infiltration occurred. As a result of this convention, the water balance can be further simplified into ET and streamflow which includes all overland flow, interflow and groundwater discharge. It was assumed that runoff, evapotranspiration and infiltration are negligible in months with average monthly temperatures below 0°C.

The water balance model was applied to climate normal, wet and dry year climate conditions to estimate the existing condition environmental water balance over a temporal scale compatible with the Project life cycle.

The environmental water balance was modeled on a monthly basis using a spreadsheet-based monthly water balance model. The water balance model requires input of monthly precipitation, average monthly temperature, soil-moisture storage capacity and infiltration factor. The soil moisture storage capacity for the study area is assumed as 150 mm based on the geology near the open pit which indicated shallow glacial till overburden approximately 4 m in depth consisting of cobbly silt-sand deposits (Stantec 2015a).

The infiltration factor for the TMF area was calculated to be 0.6 based on a topographical factor of 0.5 for an average slope less than 0.6 m/km, a soil factor of 0.12 for clay loam/clay, and a vegetation factor of 0.02-0.05 representing shallow rooted vegetation as recommended by OMOE (2003). This implies that 40% of net infiltrated precipitation will be discharged to surface water via baseflow. It is important to note that all water recharging aquifers eventually cycles back to the surface as groundwater discharge providing baseflow to local streams and lakes. As a result, the water balance can be further simplified into precipitation, ET and streamflow.

Table 4.6, Table 4.7 and Table 4.8 show the water balance results under the climate normal, wet year and dry year conditions. Evapotranspiration accounts for approximately 34.7% of total annual precipitation under climate normal conditions at the Middle Musquodoboit Climate Station. Evapotranspiration accounts for approximately 25.6 % under the 1:100 Wet Year conditions and 42.5 % under the 1:100 Dry Year conditions. The mean annual lake evaporation for the Truro climate station is 514 mm (Environment Canada 2012); the Truro pan evaporation station is located approximately 50 km northeast of the site.

Table 4.4 Environmental Water Balance under Climate Normal Conditions (1981-2010)

Parameters	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	Total
Precipitation (mm)	129.8	100.5	124.2	109	105.4	99.8	103.8	91.9	110.7	116.7	136.8	129.1	1357.7
Evapo-transpiration (mm)	0	0	0	34.2	50.0	69.0	87.3	86.78	66.4	45.4	32.13	0.00	471.3
Surface Runoff (mm)	0.0	0.0	0.0	271.4	26.9	15.0	8.0	2.5	21.5	34.6	50.9	0.0	430.8
Infiltration (mm)	0.0	0.0	0.0	287.0	28.5	15.8	8.5	2.6	22.8	36.6	53.8	0.0	455.6
Streamflow (mm)	96.6	70.0	105.5	150.7	79.8	43.4	25.7	25.7	25.7	52.3	105.5	105.5	886.4

Table 4.5 Environmental Water Balance under 1:100 Wet Year Conditions

Parameters	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	Total
Precipitation (mm)	130.2	136.0	237.9	114.0	152.5	140.8	145.7	103.1	69.8	231.9	217.1	152.5	1831.5
Evapo-transpiration (mm)	0.00	0.0	0.0	32.4	61.4	72.6	79.9	84.6	66.4	42.5	29.3	0.0	469.1
Surface Runoff (mm)	0.0	0.0	0.0	358.8	44.3	33.1	31.9	9.0	1.6	92.1	91.3	0.0	662.1
Infiltration (mm)	0.0	0.0	0.0	379.4	46.8	35.0	33.8	9.5	1.7	97.4	96.6	0.0	700.3
Streamflow (mm)	148.5	107.6	162.1	231.6	122.6	66.8	39.5	39.5	39.5	80.4	162.1	162.1	1362.4

Table 4.6 Environmental Water Balance under 1:100 Dry Year Conditions

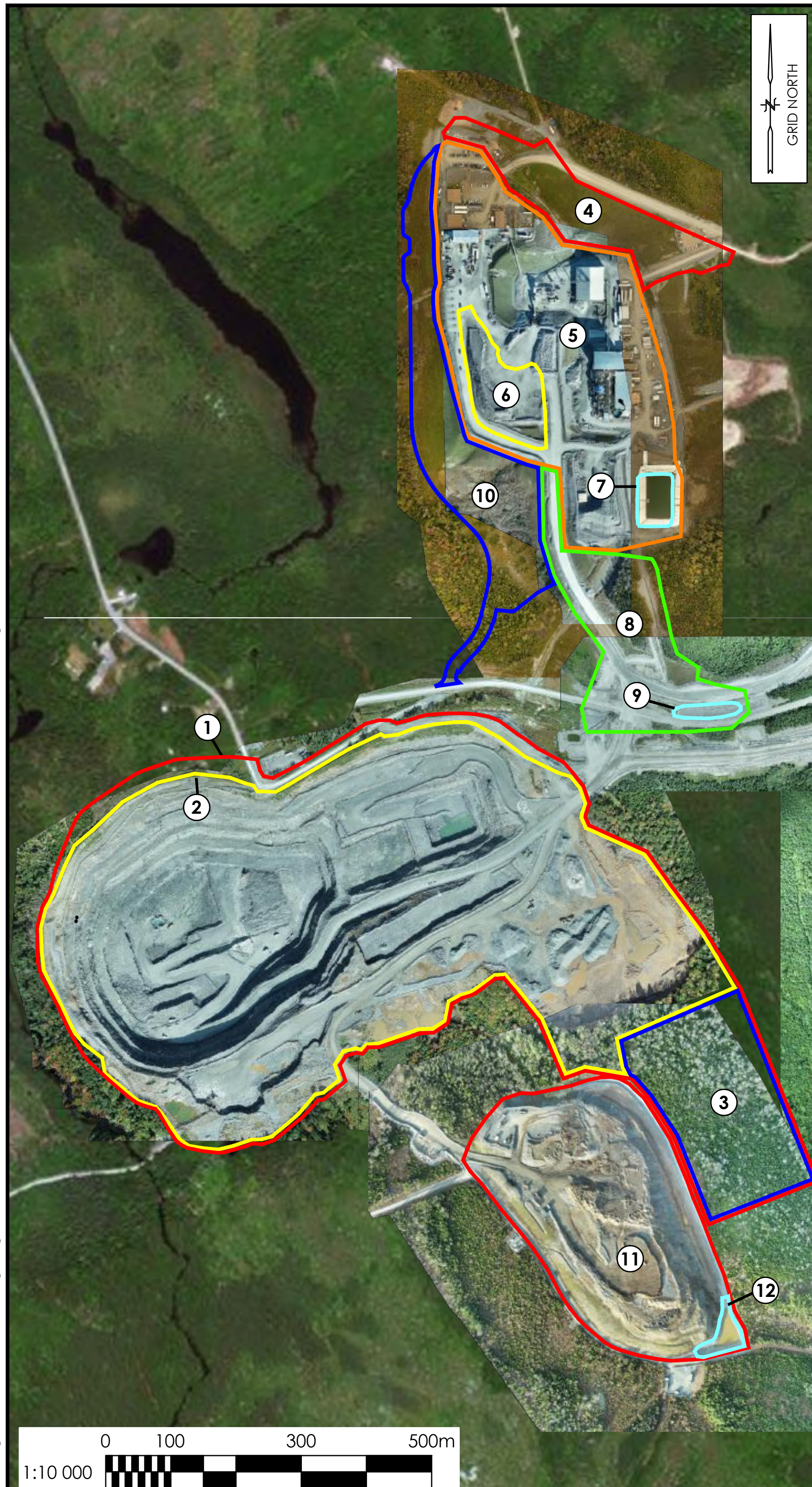
Parameters	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	Total
Precipitation (mm)	122.0	134.2	126.3	27.9	54.9	41.1	71.9	56.7	59.4	101.0	96.9	74.9	967.2
Evapo-transpiration (mm)	0.0	0.0	0.0	30.1	50.0	71.1	71.9	56.7	59.4	43.3	28.1	0.0	410.6
Surface Runoff (mm)	0.0	0.0	0.0	222.3	2.4	0.0	0.0	0.0	0.0	13.4	33.5	0.0	271.6
Infiltration (mm)	0.0	0.0	0.0	235.1	2.5	0.0	0.0	0.0	0.0	14.2	35.4	0.0	287.2
Streamflow (mm)	60.9	44.1	66.5	95.0	50.3	27.4	16.2	16.2	16.2	33.0	66.5	66.5	558.8

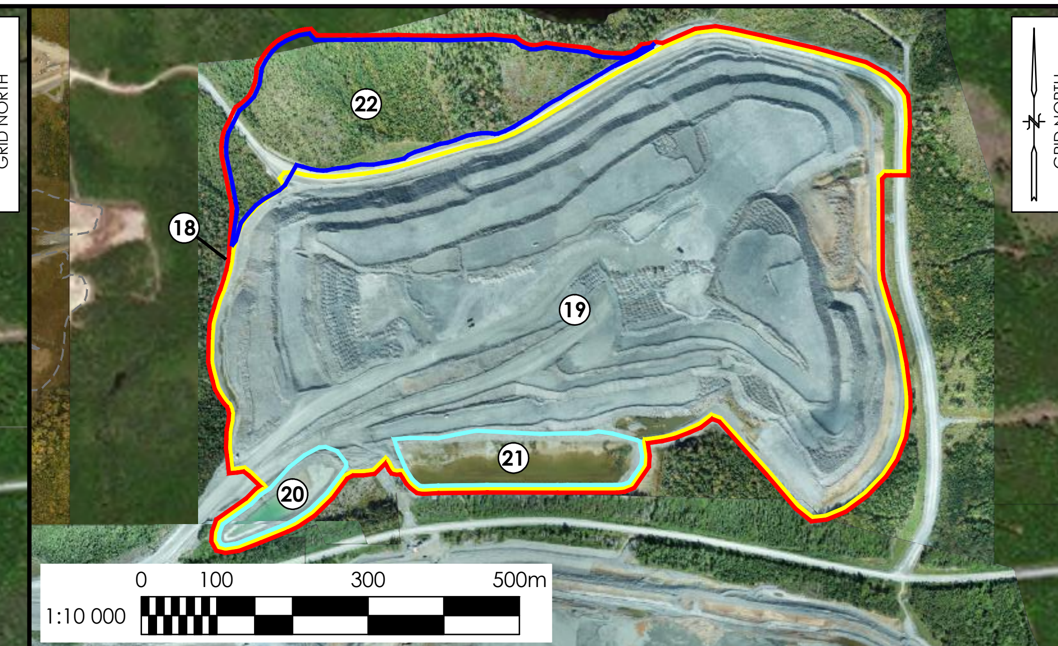
4.2 TOUQUOY TMF WATER BALANCE

The Touquoy TMF water balance was used as the basis for the water balance for the Open Pit water balance for the deposition of Touquoy tailings. The following model inputs were used, based on the calibration of the Touquoy TMF water balance.

4.2.1 Drainage Areas

The Touquoy Mine Site was delineated into five watersheds using the available Light Detection and Ranging (LiDAR) topography data (CRA 2010) and future mine site operational drainage conditions, as shown in Figure 4.1, and identified by area number and facility name. Drainage and sub-drainage areas may change as the mine develops.





OPEN PIT			
1	red	EXPANDED DRAINAGE AREA	51.3 ha
2	yellow	EXISTING INFRASTRUCTURE AREA	42.1 ha
3	blue	PROPOSED CLAY BORROW EXPANSION AREA	5.9 ha
MILL			
4	red	SEPTEMBER 2020 COLLECTOR POND DRAINAGE AREA	4.0 ha
5	orange	MILL POND DRAINAGE AREA	14.5 ha
6	yellow	EXISTING ROM PAD AREA	1.58 ha
7	cyan	MILL POND	0.42 ha
8	green	EMERGENCY PUMP POND DRAINAGE AREA	4.9 ha
9	cyan	EMERGENCY PUMP POND	0.19 ha
10	blue	PROPOSED ADMIN ROAD DRAINAGE AREA	5.3 ha
SCRAGGY OVERBURDEN STOCKPILE			
11	red	EXISTING DRAINAGE AREA	11.3 ha
12	cyan	EXISTING WET AREA	0.3 ha
TAILINGS MANAGEMENT FACILITY			
13	red	YEAR 5 DRAINAGE AREA	95.7 ha
14	yellow	WASTE ROCK PUMPED DRAINAGE AREA	8.1
15	cyan	EXISTING WET AREA	29.4 ha
POLISHING POND			
16	yellow	EXISTING AND YEAR 5 DRAINAGE AREA	15.5 ha
17	cyan	EXISTING WET AREA	9.0 ha
WASTE ROCK AREA			
18	red	EXPANDED DRAINAGE AREA	52.4 ha
19	yellow	EXISTING DRAINAGE AREA	43.6 ha
20	cyan	WEST POND	1.07 ha
21	cyan	EAST POND	2.46 ha
22	blue	PROPOSED EXPANSION	7.1 ha

THIS DRAWING ILLUSTRATES SUPPORTING INFORMATION SPECIFIC TO A STANTEC CONSULTING LTD. REPORT AND MUST NOT BE USED FOR OTHER PURPOSES.

Reference:
DATA SOURCE: DRONE IMAGERY (2019-09-06, 2019-09-28, 2019-09-30, 2019-10-01 & 2019-10-22) PROVIDED BY ATLANTIC MINING NOVA SCOTIA, BING IMAGERY © 2019 DIGITALGLOBE.

WATER BALANCE DRAINAGE AREAS
TOUQUOY MINE TAILINGS MANAGEMENT FACILITY
HALIFAX COUNTY, NOVA SCOTIA

Client: ATLANTIC MINING NS INC.

Job No.: 121619250
Scale: AS SHOWN
Date: 2021 05 25
Dwn. By: JL
App'd By: RJ
Fig. No.: 4.1

Stantec

V:\1216\active\121619250\1_geotechnical\3_drawings_logs\600_TOUQUOY\SHEET FILES\16_WBAL\2_WBAL-RV15\C19250-WBAL-RV15-FIG-4.2.dwg

4.2.2 Runoff Coefficients

Model input runoff coefficients were adjusted based on the operational responses for the Touquoy TMF to match measured parameters for natural ground, prepared ground and pile/pit/dam or beach surfaces. As summarized in Table 4.9, the runoff coefficient of the waste rock pile was derived using measured pump volumes to the TMF and assigned at 0.43. A higher value of 0.5 was forecasted for the 1:100 year climate condition to account for an increase in runoff potential under wetter conditions and toe drainage below the pile overtime from wetting. The WRSA runoff coefficient increases incrementally to 0.7, assuming that toe seepage increases over 15 years as the waste rock storage area starts to wet and the transmission of infiltration and recharge through the pile improves overtime (Smith, et al., 1995; Williams, 2006; Trinchero et al., 2011)

Table 4.7 Run-off Coefficients at the Touquoy Mine Site

Facility	Natural Ground	Prepared Ground	Pile/Pit/Dam/ Beach
Mill Site	0.67	0.85	0.9
Open Pit	0.67	0.85	0.5
Waste Rock and Overburden Piles	-	-	0.43 (dry/climate normal), 0.5 (wet)
TMF	0.67	0.85	0.92
Polishing Pond	0.67	0.85	0.9
Scraggy Overburden Stockpile	-	0.85	0.9

Note: * Run-off Coefficient of TMF Dam and wet tailings beach/dry tailings beach

4.2.3 TMF WATER INFLOW AND OUTFLOW VOLUME FORECAST

The TMF is currently the receptor for process flows from the mill in addition to pit dewatering, waste rock area drainage, seepage collection ponds, and miscellaneous inputs. Figure 4.2 presents the forecasted water volumes pumped/piped in and out of the TMF based on the 2021 calibrated water balance (Stantec 2021). Inflows include water pumped from the pit sump pond, waste rock area collection pond(s), and seepage collection ponds. Outflows include the volume treated in the existing effluent treatment plant and process water from the TMF, presented as the net of tailings slurry water volume in the TMF, and the water lock-up and process water demand. For comparison to direct flows in the TMF, the net precipitation (i.e., precipitation less evaporation and seepage losses) is presented in the Figure 4.2. As noted in the figure, the sum of inflows from the Open Pit and waste rock collection pond dewatering is comparable to the net precipitation on the TMF.

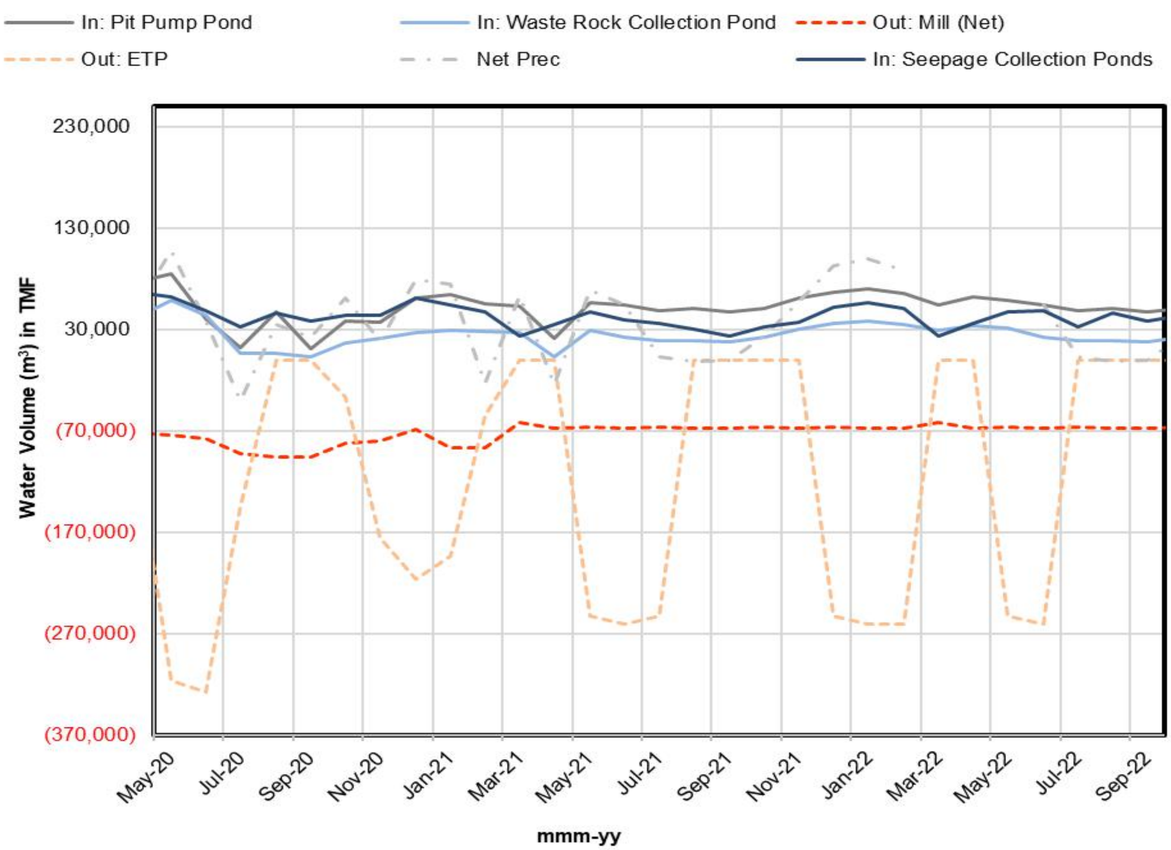


Figure 4.2 Water Volume In and Out of TMF – Climate Normal Condition

4.3 EXISTING CONDITIONS AND ASSUMPTIONS FOR TOUQUOY OPEN PIT TAILINGS MODEL

The water balance was run for the Open Pit based on the water management plan presented in Section 2.0. In-pit disposal in the model commenced in August 2022, commensurate with the completion of the open pit mining and when the TMF is simulated to reach the ultimate tailings volume. Water will be reclaimed from the TMF for most of the first month of in-pit disposal, allowing for additional water to accumulate in the pit and contribute to process water supply. Mining of the lowest section (Phase I pit) was assumed to be completed in September 2021, providing more than adequate time for the Phase I pit to naturally fill with water to the maximum estimated volume of 188,000 m^3. At this time, higher elevations in the pit will continue to be mined until the time of in-pit disposal. Pit dewatering was assumed to cease in November 2021 at which time pit inflows and seepage will begin to accumulate in the phase 1 pit, and with the potential for augmentation by redirecting TMF treated effluent from the polishing pond to the open pit. Thus in-pit tailings deposition start-up water will be supplied by drawing down the standing water volume in the TMF Pond and Polishing Pond after the tailings slurry line has been relocated to the

Open Pit as well as prefilling the phase 1 portion of the pit with seepage and runoff inflows and treated TMF effluent prior to in pit tailings deposition

Tailings was assumed to be deposited by end of pipe from the surface of the Open Pit. This would result in tailings running down the pit face and depositing sub- aqueously at the bottom of the pit from CDGV 2013 Elev. -25.0 up to 74.3 m. There would be an additional 34 m of water cover over the top of the tailings to the pit spillway elev. of 108 m. Figure 4.3 and Table 4.10 is the Open Pit stage-storage relationship to the spillway elevation at 108.0 m.

The daily groundwater inflow rate was interpolated from a relationship between groundwater inflow and elevation of the pit lake, based on the groundwater model results conducted by Stantec (2021) elevation of the pit lake. This relationship is presented in Figure 4.4.

Water balance assumptions for the deposition of tailings in the Open Pit are summarized in Table 4.11.

Table 4.8 Pit Stage Storage Relationship

Water Elevation (m)	Total Open Pit Storage Volume (1000 m^3)	Water Elevation (m)	Total Open Pit Storage Volume (1000 m^3)
-25.00	8	45.00	1,714
-20.00	21	50.00	2,114
-15.00	41	55.00	2,570
-10.00	67	60.00	3,082
-5.00	104	65.00	3,691
0.00	157	70.00	4,361
5.00	236	75.00	5,101
10.00	328	80.00	5,964
15.00	432	85.00	6,944
20.00	561	90.00	7,985
25.00	722	95.00	9,074
30.00	902	100.00	10,225
35.00	1,111	105.00	11,486
40.00	1,373	108.00	12,276

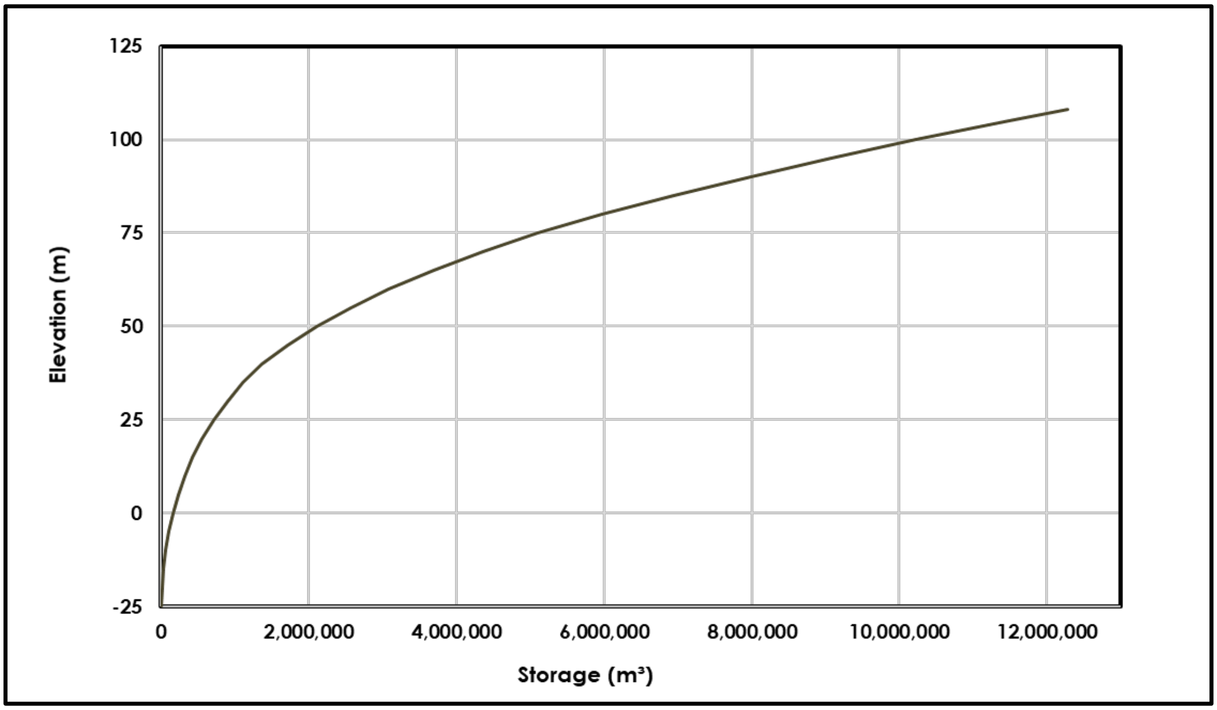


Figure 4.3 Elevation Storage Relationship in the Exhausted Touquoy Pit

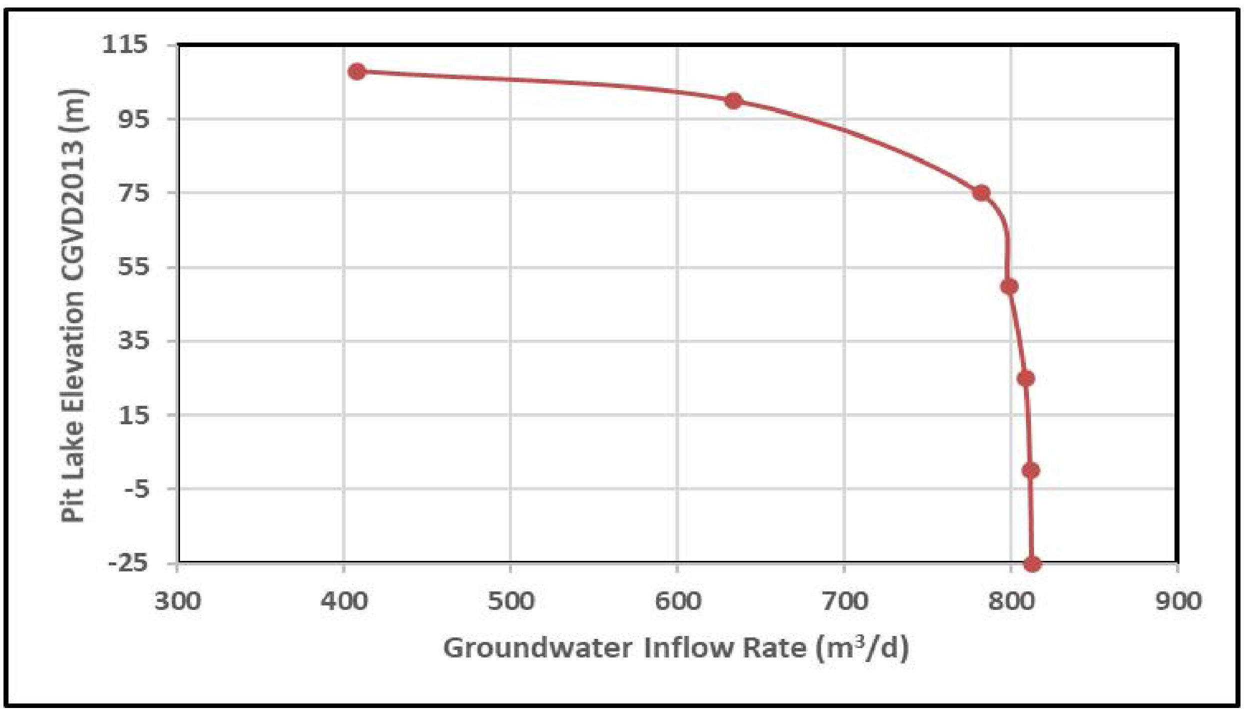


Figure 4.4 Groundwater Inflow Rates to Pit Based on Water Elevation in Touquoy Pit

Table 4.9 In-Pit Tailings Deposition Assumptions

Parameter	Value
Tailings Production	8,900 tpd
Deposited Tailings Volume (at 1.3 t/m^3 density)	6,846 m^3/d
Slurry Density (of dry mass tailings)	41%
Moisture Content of Ore Going into Mill Facility	222.5 m^3/d (2.5% of tailings production (t))
Fresh Make-up Water to Mill Facility	720 m^3/d
Saturated Water Content of Deposited Tailings	41.6%
Water Lost to Evaporation and Spillage at Mill Facility	384.2 m^3/d (3% of tailings production (t))
Water in Tailings Discharged in Tailings Slurry	12,807 m^3
Recycled Water to the Mill Facility	12,249 m^3
Water Retained in the Consolidated Tailings Mass	3,701 m^3
In-Pit Tailings Deposition Method	Sub-aqueous – End of Pipe
Pit Groundwater Inflow*	Based on elevation of pit lake, ranges between 813 m^3/d - 408 m^3/d
Collected TMF and PP seepage	1,336 m^3/d
Initial Pond Volume in the TMF at time of In-Pit disposal	350,000 m^3
Phase I Pit Volume	188,000 m^3 available Nov 2021- Aug 2022
Note: Based on 2021 groundwater flow model of the expanded pit (Stantec 2021a)	

4.4 MODEL RESULTS

The water balance model predicted the amount of water and tailings stored in the Open Pit over the simulation period for Touquoy ore processing. The model considered the evaporation losses in the Open Pit, groundwater inflows, and surface water runoff from direct precipitation to the pit catchment (including the Clay Borrow Area) and water directed from the WRSA and Scraggy stockpile. The average monthly runoff and ground water inflows to the open pit are summarized on Table 4.12, once in-pit disposal is complete. During closure, the surface runoff diverted to the pit would be returned to pre-development drainage patterns and the overflow from the Open Pit would include only runoff from the pit catchment and waste rock storage area until the pit fills. Pit overflow during closure is diverted to Moose river via a spillway and conveyance channel.

Table 4.10 Average Monthly Pit Inflows (m^3/month) During Pit Filling and at Pit Overflow

Month	Runoff to Open Pit	Evaporation from Pit Lake	Pit GW Inflow	Water Diverted from WRSA	Sum of Inflows to Pit During Pit Filling	Pit Overflow During Closure
January	20,822	0	24,817	3,359	58,291	38,871
February	26,383	0	22,589	4,289	64,994	32,383
March	33,094	0	24,749	10,841	81,854	35,537
April	145,991	0	23,881	18,689	204,953	75,409
May	45,168	(36,251)	24,579	6,788	50,112	119,303
June	42,768	(41,130)	23,730	6,345	40,967	19,223
July	44,482	(47,502)	24,482	6,556	37,609	11,660
August	39,383	(38,751)	24,450	5,901	39,498	7,322
September	47,439	(27,824)	23,633	7,493	61,140	11,236
October	50,010	(16,251)	24,385	8,182	77,386	29,397
November	58,624	0	23,558	9,991	105,278	43,816
December	27,662	0	24,298	4,740	62,884	67,829

The monthly Open Pit process water balance is summarized in Table 4.13 for the period of in-pit disposal. The process flows include the water in the tailings slurry, water retained in the consolidated tailings mass, recycled water to the Mill Facility from the Open Pit, and recycled water to the Mill Facility from the TMF. The cumulative Open Pit tailings volume and water volumes and the associated elevations are also summarized in the table.

Tailings are proposed to be deposited in the exhausted Open Pit for a total of 24 months and assuming a start up prefilling volume of water in the pit of 188,000 m^3. As originally planned in the approved Touquoy Gold Mine Project Reclamation Plan (Stantec 2017b), the inflow of groundwater, surface runoff and precipitation into the Open Pit will naturally create a lake upon closure of the site. The water balance model simulated that it would take an additional 79 months or a total of 103 months from commencement of tailings deposition in the exhausted Open Pit to fill the pit to the spillway invert elevation, considering the accelerated pit filling and climate normal precipitation. Figure 4.5 and Figure 4.6 illustrate the predicted water and tailings elevation and storage volume in the exhausted Open Pit, respectively.

Based on results of the water balance model, process water can be reclaimed from the TMF just under one month, prior to relocating the reclaim to the exhausted Open Pit. As the Open Pit will accumulate a start-up water supply in the pre-processing period and additional water from the TMF and Polishing Pond is directed to the Open Pit to supplement process supply throughout operation, the Open Pit has adequate water supply (≥ 538,000 m^3) for continued processing of the Touquoy ore for the modelled climatic conditions.

Based on the proposed pit prefilling and start up water, under climate normal conditions it is anticipated that a water cover will be maintained over the deposited tails ranging from a minimum of 1.75 m at the start of tailings deposition to 4 m at the end of tailings deposition. The proposed water cover will enable continued operation during winter when ice cover will temporarily sequester some of the available water, and will provide sufficient water depth to avoid entraining tailings in the pump intake.

Table 4.11 Open Pit Process Water Balance

Year	Month of Year	Sum of Inflows to Pit	Supplemental Flow from Polishing Pond	Discharged with thickened tailings	Water Retained in the Consolidated Tailings Mass	Recycled Water to Mill from the Pit	Recycled Water to Mill/Pit from TMF	Touquoy Pit Tailings Volume (m³)	Touquoy Pit Tailings Elevation (m³)	Touquoy Pit Water Volume (m³)	Touquoy Pit Water and Tailings Elevation (m)
Year 1	Initial	247,286	0	0	0	0	385,910	0	(25.0)	247,286	5.6
	Aug	48,291	0	397,027	(114,740)	(85,930)	293,790	212,231	3.5	483,398	24.2
	Sep	71,729	0	384,220	(111,038)	(309,529)	57,942	417,615	14.3	508,294	30.6
	Oct	88,668	0	397,027	(114,740)	(306,733)	72,988	629,846	22.1	561,295	36.5
	Nov	118,524	75,000	384,220	(111,038)	(261,970)	105,501	835,231	28.1	752,696	43.2
	Dec	69,661	0	397,027	(114,740)	(344,875)	34,845	1,047,462	33.5	753,236	46.1
	Jan	67,583	0	397,027	(114,740)	(358,866)	20,854	1,259,692	37.8	734,949	48.5
	Feb	76,727	0	361,807	(104,561)	(311,878)	34,157	1,453,096	41.2	745,310	50.9
	Mar	95,024	0	397,027	(114,740)	(266,082)	113,638	1,665,327	44.3	843,370	54.3
	Apr	221,344	0	384,220	(111,038)	(213,254)	154,217	1,870,712	47.0	1,108,250	59.0
Year 2	May	59,939	109,427	397,027	(114,740)	(334,380)	45,340	2,082,942	49.6	1,215,696	61.8
	June	50,222	0	384,220	(111,038)	(297,906)	69,565	2,288,327	51.9	1,231,938	63.6
	July	47,200	0	397,027	(114,740)	(345,762)	33,958	2,500,558	54.2	1,206,073	65.1
	Aug	48,014	0	397,027	(114,740)	(348,430)	31,290	2,712,788	56.4	1,179,428	66.5
	Sep	71,538	0	384,220	(111,038)	(309,529)	57,942	2,918,173	58.4	1,204,221	68.2
	Oct	88,445	0	397,027	(114,740)	(306,733)	72,988	3,130,404	60.4	1,257,161	70.2
	Nov	118,384	0	384,220	(111,038)	(261,970)	105,501	3,335,788	62.1	1,373,651	72.4
	Dec	69,068	0	397,027	(114,740)	(344,875)	34,845	3,548,019	63.8	1,373,947	73.8
	Jan	67,268	0	397,027	(114,740)	(358,866)	20,854	3,760,250	65.5	1,355,344	75.1
	Feb	76,513	0	361,807	(104,561)	(311,878)	34,157	3,953,654	67.0	1,365,493	76.3
	Mar	128,125	0	397,027	(114,740)	(266,082)	113,638	4,165,885	68.5	1,496,653	78.3
	Apr	188,525	0	384,220	(111,038)	(213,254)	154,217	4,371,269	70.1	1,728,715	80.7
Year 3	May	59,066	0	397,027	(114,740)	(334,380)	45,340	4,583,500	71.5	1,725,861	81.8
	June	49,233	0	384,220	(111,038)	(297,906)	69,565	4,788,885	72.9	1,741,116	82.9
	July	46,013	0	397,027	(114,740)	(345,762)	33,958	5,001,115	74.3	1,714,064	83.8

Note: (Red) indicates a water loss

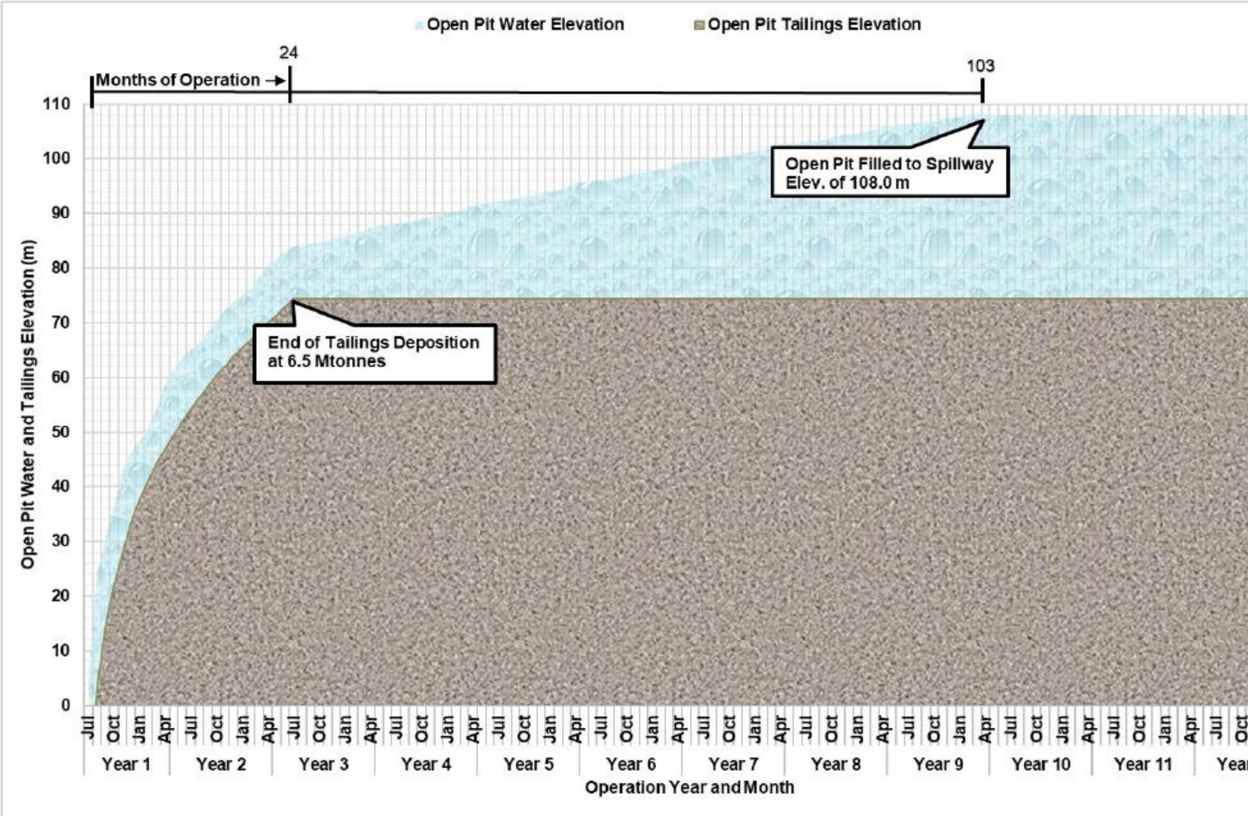


Figure 4.5 Tailings and Water Elevation in the Exhausted Touquoy Pit

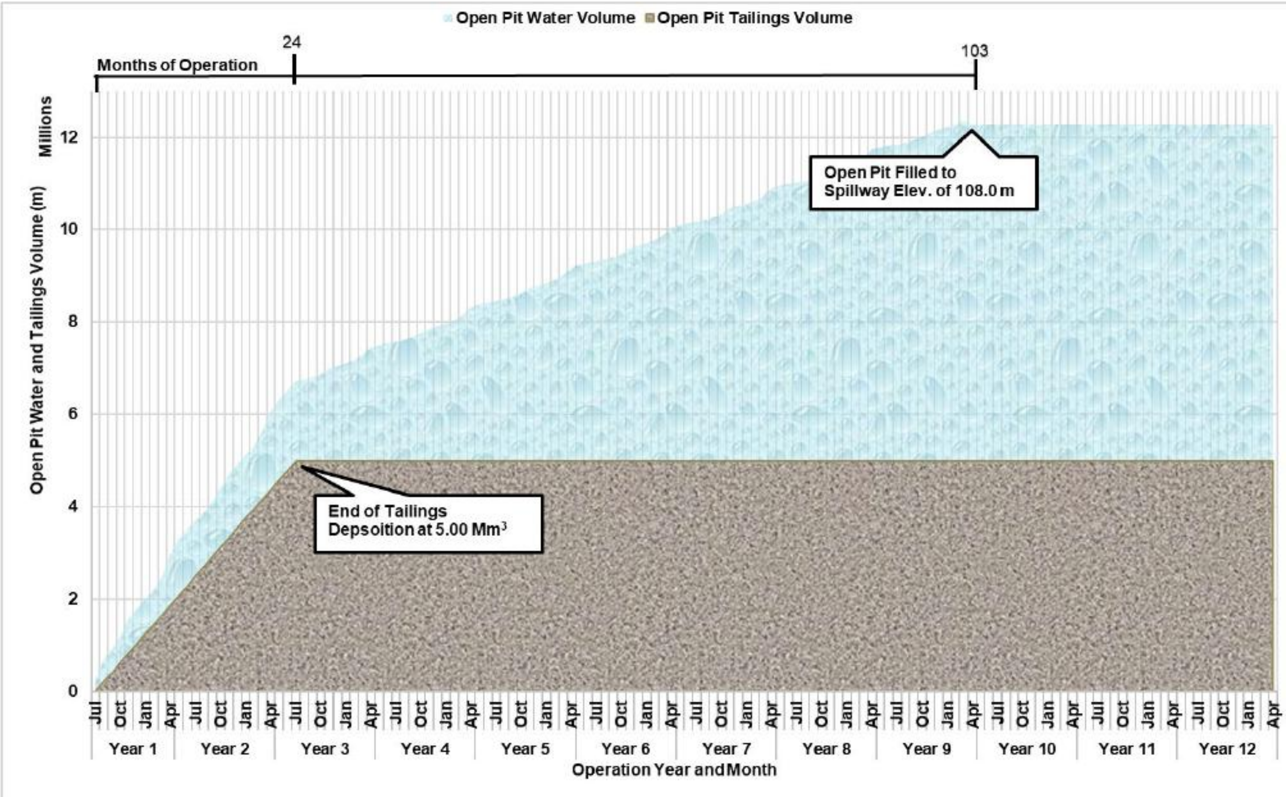


Figure 4.6 Tailings and Water Storage Volume in the Exhausted Touquoy Pit

5.0 WATER QUALITY MODEL

Deposition of tailings in the exhausted Open Pit will alter water quality in the Open Pit compared to filling of the Open Pit as per the Touquoy reclamation plan (Stantec 2017b). The monthly water quality model for the exhausted Open Pit was developed using Excel to simulate the overall water quality of metal parameters, cyanide, and nitrogen species (including ammonia, nitrate, and nitrite) during operation, reclamation, and closure of the Project. This model was developed from the water balance spreadsheet for the Open Pit, using inputs from the existing and calibrated water balance for the Touquoy TMF. The objectives of the Touquoy water quality model are to predict future water quality and inform water treatment required prior to the pit lake effluent discharge to Moose River, and the water quality of effluent discharge to Moose River at aquatic monitoring stations. The environmental effects of predicted discharge water quality in Moose River are assessed.

5.1 GEOCHEMICAL SOURCE TERMS

Water quality modelling considered the pore water quality in the tailings and the pit floor/ walls, the dilution from surface runoff, direct precipitation in the exhausted Open Pit and the water quality of the mixture based on the geochemistry of the individual water quality parameters. As discussed in the source terms memo (Lorax 2020), geochemical source term predictions of pore water quality of pit walls/floor were derived from upscaling of kinetic tests and Touquoy monitoring data. The geochemical model used to develop the source terms (Lorax 2018) simulated the oxidation and reduction reactions to understand the water quality of the mixed pit lake quality based on the geochemistry of the individual water quality parameters during operation and reclamation. The kinetic testing and Touquoy monitoring data were considered representative for the Project as the ore bodies are from the same geologic formation as the Touquoy ore with similar marker parameter content.

As presented by Lorax (2020) and Minnow (2021), geochemical source term predictions of pore water quality of pit walls/floor had elevated metal (e.g., arsenic, cobalt, copper), ammonia, nitrate and cyanide concentrations thus reducing pit lake water quality at the time of discharge.

Using the Touquoy TMF as a site analogue for saturation indices (Lorax 2020), solubility caps were predicted for iron (0.10 mg/L at end of mine and 0.039 mg/L at closure) and aluminum (0.178 mg/L at end of mine and 0.057 mg/L at closure). As recommended by Lorax (2020), a degradation rate for ammonia of $y = -0.0134x^2 + 0.4915x + 0.0676$ was applied, where x is the ammonia concentration in a given year. The degradation rate for ammonia was capped at 4.57 mg/L/yr for ammonia concentrations of 18.35 mg/L or above. Degraded ammonia was converted to nitrate and nitrite in operation and reclamation, at ratios provided by Lorax. During operation, a higher proportion of nitrite was predicted due to competing oxygen-consuming mechanisms where 25% as NO_3 and 75% as NO_2 (Lorax 2020). Within approximately 3 years following completion of tailings deposition, most of the nitrite was estimated to oxidize to nitrate with 98% as NO_3 and 2% as NO_2.

5.2 MODEL INPUTS AND ASSUMPTIONS

The water quality model combined the quality of the source terms with the water balance model flows and groundwater interaction to predict monthly discharge water quality over 50 years beginning at the start of discharge into the exhausted Open Pit, simulating steady state conditions for all source terms provided by Lorax.

As described in the previous water balance section, inputs to the Open Pit include the tailings and water slurry deposition, direct precipitation, and surface water runoff. These flows have been modelled for a climate normal scenario over 50 years.

Based on results of the groundwater flow model (Stantec 2021a), the Open Pit acts as a sink (i.e., gaining groundwater to the Open Pit) until the groundwater level reaches the shallow weathered bedrock layer. The interaction between the Open Pit lake and Moose River is limited to groundwater flow from Moose River to the pit during this period. Therefore, no water quality effects to Moose River are predicted during this period. When the pit lake level rises to the spillway elevation of 108 m, the groundwater flow gradients allow for seepage from the Open Pit to migrate towards the Moose River as baseflow at a rate of approximately 258 m^3/d. The flow rate in Moose River of 2,160 m^3/d in April is 125 times this rate, and therefore represents a dilution ratio of approximately 125.

The water quality model predicts the effluent discharge quality from the Open Pit during reclamation and closure. Effluent discharge water quality from the pit lake to Moose River is required to meet MDMER discharge limits. Therefore, it was assumed that any effluent quality for any parameter that exceeds the MDMER limits will be treated to meet the MDMER limits. Discharge from the Open Pit is not anticipated until after 2021, therefore the MDMER discharge limits for an existing mine after June 1, 2021 (MDMER Schedule 4, Table 2) were used as minimum treatment criteria for effluent discharges to Moose River. An assimilative capacity study of Moose River (Stantec 2021b) was completed to simulate the mixed water quality at the future MDMER biological monitoring stations located at 100 m, 250 m, and 1000 m downstream of the effluent discharge point.

5.2.1 Water Treatment

Similar to Touquoy ore processing, the tailings slurry from the processed ore will be subject to cyanide destruction at the process plant before pumping to the exhausted Open Pit. Based on water quality monitoring results at Touquoy for existing operation, cyanide destruction to cyanate is 99.5% effective (Lorax 2018; Minnow 2021). Cyanate readily complexes with metals and can precipitate under increased pH conditions. The majority of the residual cyanide reagent introduced to the tailings during ore processing will be degraded and hydrolyzed to carbon dioxide and ammonium during storage in the tailings pond. Similarly, this will be expected to occur for the Touquoy tailings being stored in the Open Pit. Potential failures related to cyanide recovery and proposed Touquoy pit disposal will be addressed in updates to the existing Touquoy groundwater contingency plan (Stantec 2019), as required in the Industrial Approval for the Touquoy Gold Project.

An effluent treatment plant is planned to be located at the Touquoy Open Pit engineered spillway to treat the pit lake water until MDMER discharge limits are met. The water quality of the pit lake will be monitored during the pit filling and as the pit level approaches the spillway elevation. The water quality will be compared to the MDMER discharge limits and will be treated as required to meet these limits and additional regulatory closure criteria or site-specific guidelines, presented in Table 5.1.

Table 5.1 MDMER Schedule 4 Table 2 Limits of the Metal and Diamond Mining Effluent Regulations

Deleterious Substance	Maximum Authorized Monthly Mean Concentration	Maximum Authorized Concentration in a Composite Sample	Maximum Authorized Concentration in a Grab Sample
Arsenic	0.30 mg/L	0.45 mg/L	0.6 mg/L
Copper	0.30 mg/L	0.45 mg/L	0.60 mg/L
Cyanide	0.5 mg/L	0.75 mg/L	1.00 mg/L
Lead	0.10 mg/L	0.15 mg/L	0.20 mg/L
Nickel	0.50 mg/L	0.75 mg/L	1.00 mg/L
Zinc	0.50 mg/L	0.75 mg/L	1.00 mg/L
Total Suspended Solids	15.00 mg/L	22.50 mg/L	30.00 mg/L
Radium 226	0.37 Bq/L	0.74 Bq/L	1.11 Bq/L
Un-Ionized Ammonia	0.50 mg/L (as nitrogen)	Not applicable	1.00 mg/L (as nitrogen)

Proposed water treatment strategies include:

- Initial treatment of the pit as a batch reactor with the objective of adjusting the pH to precipitate metals to improve water quality in the pit lake as the pit is filling. As an additional benefit of the slow filling of the pit over time, the residence time and exposure to sunlight will increase, thus enhancing the natural UV degradation of cyanide and improving water quality in the pit lake.
- Should water treatment still be necessary, effluent from the Open Pit will be pumped for treatment to an effluent treatment plant and discharged to the Moose River receiving environment. Once water quality meets discharge criteria (i.e., representing closure conditions), surplus water in the Open Pit will spill to a channel and discharge to Moose River. Discharge water quality will continue to be monitored against discharge criteria to identify if the pit should continue to be pumped and treated at the effluent treatment plant prior to discharge to the Moose River.

5.3 MODEL RESULTS

The water quality model predicted an exceedance of the MDMER discharge limits. In February of Year 9 at the commencement of discharge from the Open Pit when the pit lake is simulated to reach the spillway elevation, the water quality model predicted elevated concentrations of arsenic and nitrite pot lake water as summarized in Table 5.2. Results of the water quality model in the exhausted Open Pit lake over time for metals, ammonia, and cyanide parameters are presented in figures included in the Appendix A. These figures show the water quality trend over time and the outflow to Moose River.

Table 5.2 Predicted Water Quality Concentrations in the Touquoy Pit Lake

Parameter	Effluent Discharge Concentration (mg/L) in Year 9	Groundwater Seepage Concentration (mg/L) in Year 50	Schedule 4 Table 2 Limits MDMER Monthly Mean Concentration (mg/L)
(SO_4) Sulphate	384	1.3×10^{-3}	
(Al) Aluminum	0.095	6.6×10^{-8}	
(As) Arsenic	**1.05**	4.3×10^{-6}	0.30
(Ca) Calcium	109	1.2×10^{-4}	
(Cd) Cadmium	0.000016	2.8×10^{-11}	
(Co) Cobalt	0.079	3.7×10^{-8}	
(Cr) Chromium	0.00057	2.8×10^{-10}	
(Cu) Copper	0.044	1.3×10^{-8}	0.30
(Fe) Iron	0.058	4.6×10^{-8}	
(Hg) Mercury	0.000021	7.1×10^{-12}	
(Mg) Magnesium	13.9	2.1×10^{-5}	
(Mn) Manganese	0.173	5.2×10^{-7}	
(Mo) Molybdenum	0.010	8.5×10^{-8}	
(Ni) Nickel	0.020	9.7×10^{-9}	0.50
(Pb) Lead	0.00033	3.5×10^{-11}	0.10
(Se) Selenium	0.0011	2.7×10^{-10}	
(Ag) Silver	0.00005	1.6×10^{-11}	
(U) Uranium	0.0048	4×10^{-9}	
(Zn) Zinc	0.0039	3.2×10^{-9}	0.5
(WAD CN) Weak Acid Dissociable Cyanide	0.150	1.5×10^{-8}	0.5
(Total CN) Total Cyanide	0.458	8.0×10^{-9}	
(NO_3) Nitrate (as N)	10.4	1.4×10^{-7}	
(NO_2) Nitrite (as N)	4.3	8.5×10^{-8}	
(NH_3) Unionized Ammonia	**0.64**	1.8×10^{-7}	0.50 (Unionized)

Note: **Bold numbers** indicates an exceedance of MDMER discharge limit

The pit lake is simulated to take approximately 9 years to fill from commencement of depositing tailings in the exhausted Open Pit. Effluent from the pit lake will be discharged to meet MDMER limits. The water quality in the pit is predicted to initially exceed the MDMER discharge limits prior to discharge, not considering the water quality treatment. The water quality is predicted to improve with time and is predicted to no longer require treatment to meet MDMER discharge limits after approximately 28 years from commencement of depositing tailings in the exhausted Open Pit.

6.0 MODEL SENSITIVITY AND LIMITATIONS

Results of the water balance and quality model are based on information available at the time of the study, as sections above. It is recommended that the existing conditions and assumptions be updated as information becomes available, such as further developed reclamation plan, updates of the water balance/water management plan, updates to the mine plan, testing to predict settled tailings density, and the results of operational monitoring.

The 1:100 AEP wet and the 1:100 AEP dry climate statistics are used to provide an upper and lower bound of predicted climate normal conditions. Assuming the model boundary conditions account for potential future conditions, water levels in the TMF and exhausted Touquoy pit during the 24 months of processing of Touquoy ore, should fall within these bounds.

Model sensitivity to predicted Open Pit groundwater inflows were conducted by adjusting the groundwater contribution of 813 m^3/d associated to a pit water elevation of -25.0 m (CGVD2013) to the groundwater contribution filled with water to elevation 108.0 m (CGVD2013) of 408 m^3/d. This change would delay the timing of when the process water reclaim is relocated from the TMF to the exhausted Open Pit by 1 day.

The variation in the initial pond water volume between low and high operating levels in the TMF on the available water reclaim at time of commencement of tailings deposition in the exhausted Open Pit was modelled. Should the pond at the time of start-up be at a low operating level opposed to a high operating level, reclaim supply from the TMF and Polishing Pond will be less than a month and conversely may be more than a month in TMF pond water levels are higher than normal.

Sensitivity on the deposited tailings density in the exhausted Open Pit was simulated. The average deposited tailings density of 1.3 t/m^3 is expected, with a lower tailings density at start-up and a higher density as tailings are deposited in the exhausted Open Pit due to the consolidation of the tailings from the accumulating tailings and water mass. Lower initial tailings density would result in additional initial tailing porewater lock up which is why prefilling and drawing down the TMF and Polishing ponds at start up are critical elements of this Project. As further tails and water are deposited in the pit the lower tails will consolidate, liberating porewater until an equilibrium is reached balancing the additional porewater demand in fresh tails deposition vs porewater liberation from deeper consolidated tails.

7.0 SUMMARY & RECOMMENDATIONS

7.1 WATER MANAGEMENT

The water management plan at the Touquoy Mine Site for Touquoy ore processing considered the existing process water requirements, existing water management infrastructure, the water inventory at the mine site, the available freshwater sources, and effluent water quality. Consistent with existing water management at the site, the TMF continues to collect direct precipitation and runoff from the tailings pond, the WRSA and perimeter toe seepage collection. Tailings slurry is discharged to the exhausted Open Pit upon commencement of processing of the Touquoy ore. Initially, process water is reclaimed from the TMF until pond volumes are depleted and inadequate to meet process water requirements. A second reclaim line will be installed from the exhausted Open Pit to the Mill Facility as a closed loop.

Surplus water in the TMF during operation will be pumped to the Mill Facility to supplement process water supply. During climate normal conditions, there will not be a discharge from the TMF to the existing FDP.

During closure, the TMF will follow the closure plan for a vegetated cover that limits infiltration and drains surface runoff toward predevelopment catchments. The exhausted Open Pit will be allowed to fill. Surplus water in the exhausted Open Pit is managed through a water treatment plant (if required) and proposed spillway/conveyance channel draining to Moose River.

The water management plan should be updated to reflect the next stage of design of in-pit disposal.

7.2 TAILINGS DEPOSITION

It is assumed that tailings deposition will be performed using subaqueous deposition of a conventional tailings slurry through a barge. Deposition strategies will require routine modification based on the season. An approximate volume of deposited tailings of 6.03 Mt will be deposited. The capacity of the exhausted Open Pit can manage both the tailings and water volume, and thus accommodating flood storage and freeboard.

The tailings management plan should be updated to reflect the next stage of design. A detailed tailings deposition plan should be developed to support operation to define the monthly deposition areas.

7.3 WATER BALANCE MODEL

The water balance model provides an understanding of the water and tailings management for processing of the Touquoy ore.

The exhausted Open Pit in combination with the TMF is predicted to have sufficient process water for the Touquoy mine life. Reclaim from pit as a process water supply will be less than 1 month under a climate normal condition to allow for time to build up a start-up water supply. For example, initially process water will be reclaimed to the Mill Facility from the TMF through the existing reclaim barge and related water piping infrastructure until pond volumes are no longer adequate for process water reclaim.

Reclaiming process water initially from the TMF will reduce the required capacity of booster pumps in the exhausted Touquoy pit, as a greater capacity is required with depth. The existing reclaim water lines and decant pump could be retrofitted to accommodate the change to the source of the process water reclaim supply.

The water balance should be updated throughout operation to allow for model calibration and to reflect changed in detailed design.

7.4 WATER QUALITY MODEL

Water quality modelling considered the pore water quality in the tailings and the pit floor and walls, dilution from surface runoff, direct precipitation in the pit, and the water quality of the mixture based on the geochemistry of the individual water quality parameters. Water quality is simulated to include elevated metals (e.g., arsenic, cobalt, copper), ammonia, nitrate and cyanide concentrations thus reducing pit lake water quality at the time of pit overflow discharge. The pit lake will be treated to meet applicable MDMER discharge limits for an existing mine prior to discharge to Moose River. As the pit lake was simulated to take approximately 9 years to fill from commencement of Touquoy ore, the water treatment design will be fully developed during operation and pit filling.

Water quality predictions and assimilative capacity in Moose River will be updated following an update of source terms as a result of the on-going Touquoy geochemistry assessment. Following this study, a water treatment plan will be further developed for implementation in operation and reclamation of the Touquoy Mine Site.

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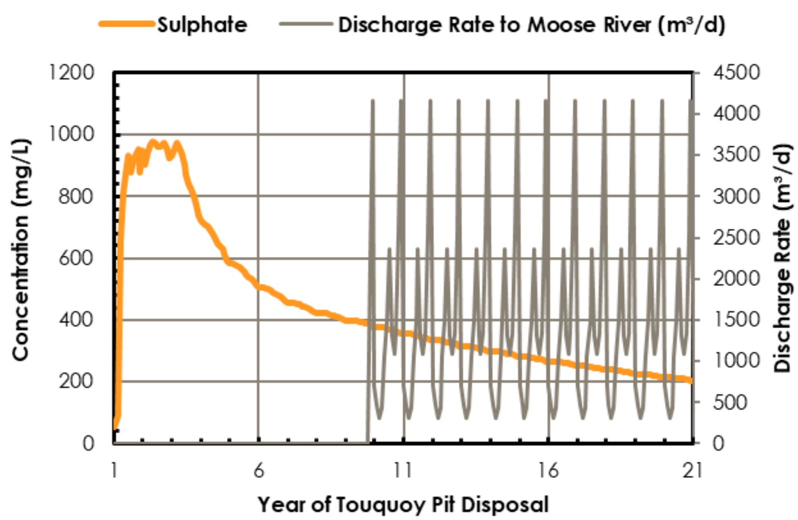
Groundwater Flow and Solute Transport Modelling to Evaluate Disposal of Tailings in Touquoy Open Pit

APPENDIX A
Water Quality Predictions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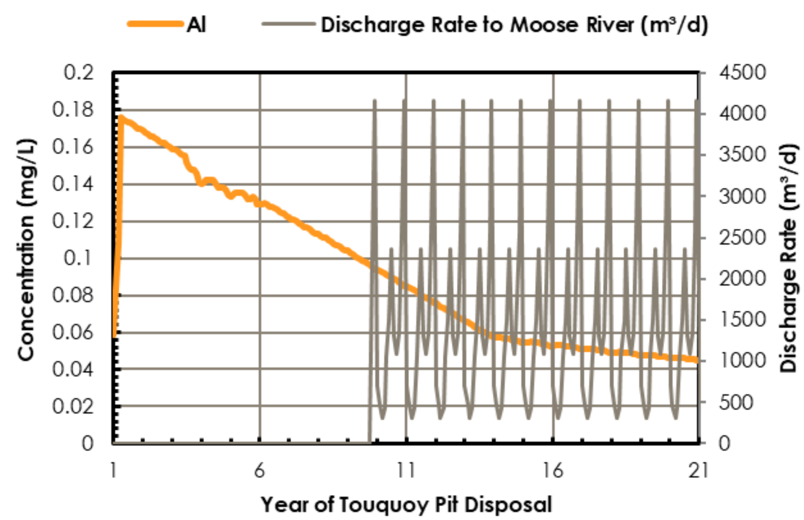
Discharge Rate to Moose River (m³/d)
Concentration (mg/L)
Discharge Rate (m³/d)
Year of Touquoy Pit Disposal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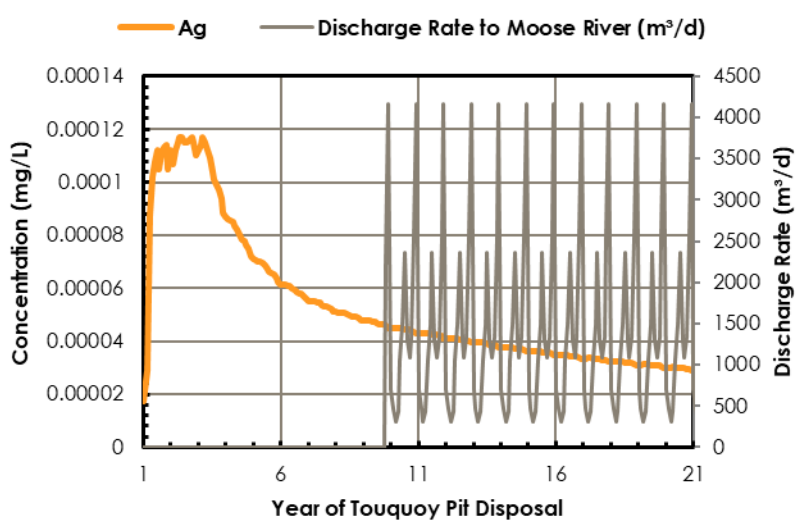
Concentration (mg/L)
Discharge Rate (m³/d)
Year of Touquoy Pit Disposal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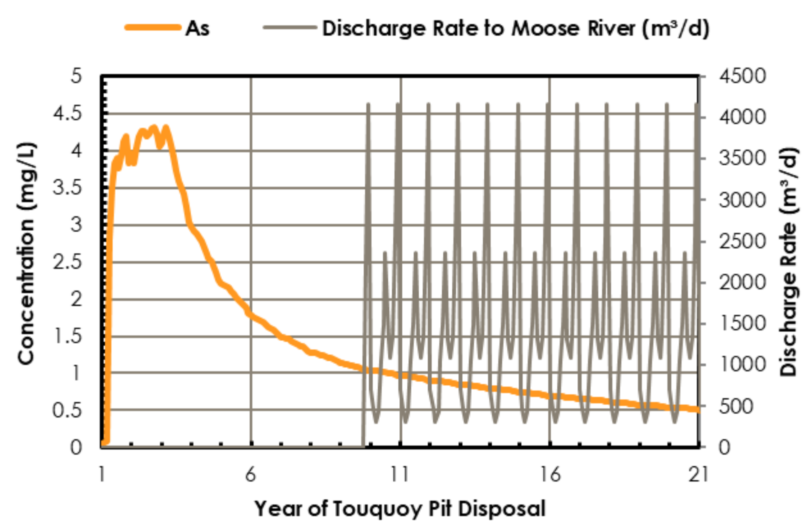
Discharge Rate to Moose River (m³/d)
Concentration (mg/L)
Discharge Rate (m³/d)
Year of Touquoy Pit Disposal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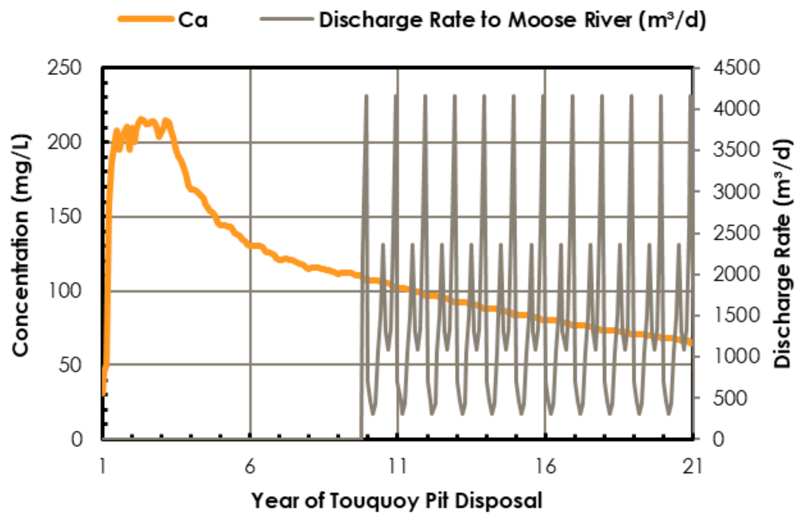
Discharge Rate to Moose River (m³/d)
Concentration (mg/L)
Discharge Rate (m³/d)
Year of Touquoy Pit Disposal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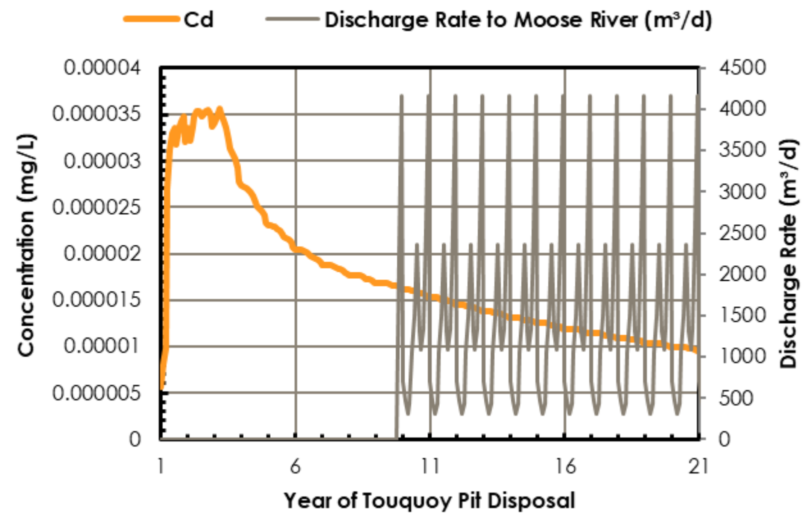
Concentration (mg/L)
Discharge Rate (m³/d)
Year of Touquoy Pit Disposal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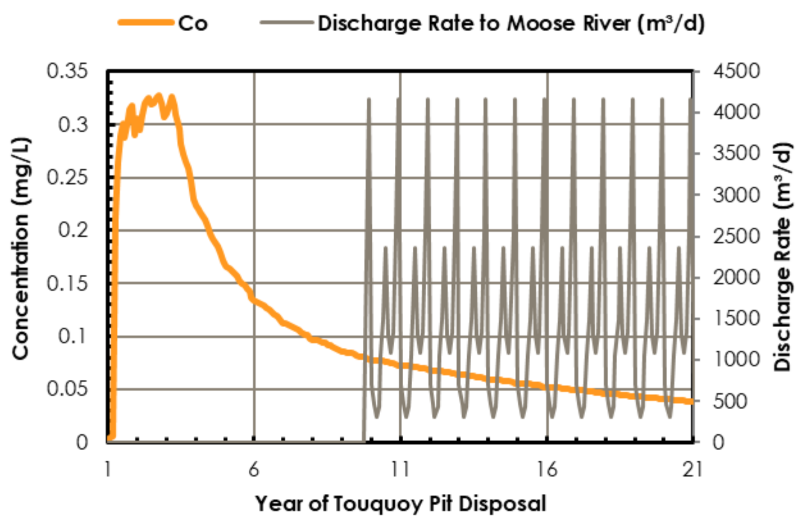
Concentration (mg/L)
Discharge Rate (m³/d)
Year of Touquoy Pit Disposal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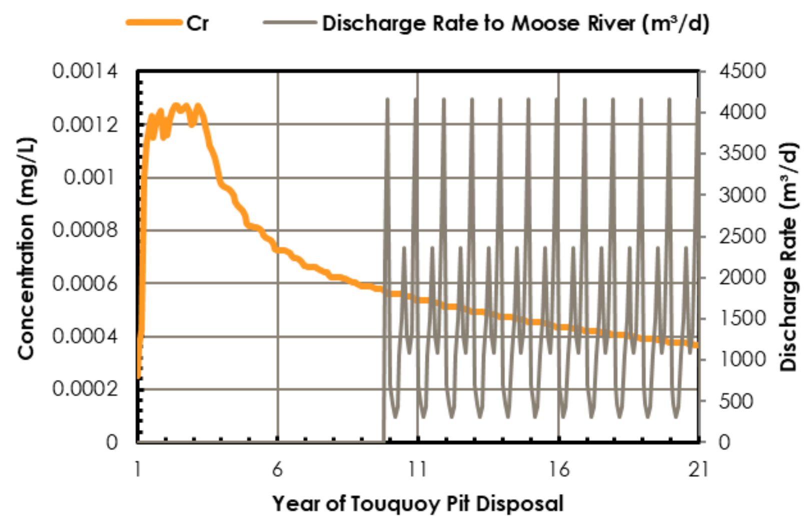
Concentration (mg/L)
Discharge Rate (m³/d)
Year of Touquoy Pit Disposal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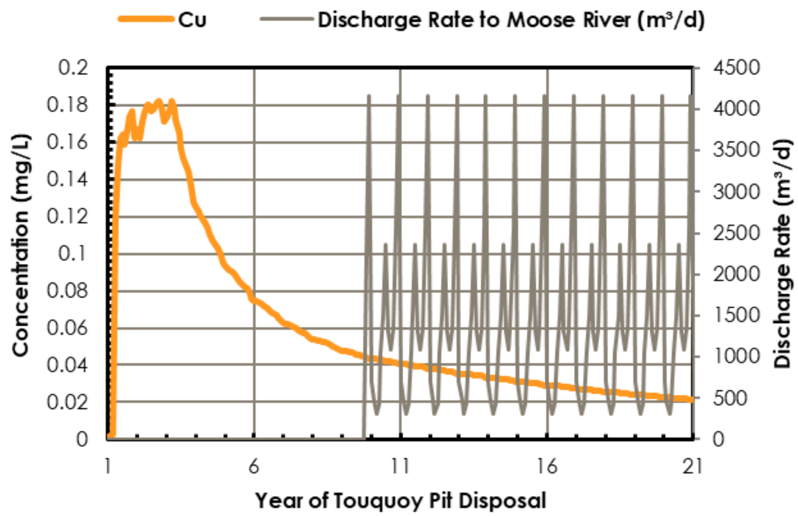
Discharge Rate to Moose River (m³/d)
Concentration (mg/L)
Discharge Rate (m³/d)
Year of Touquoy Pit Disposal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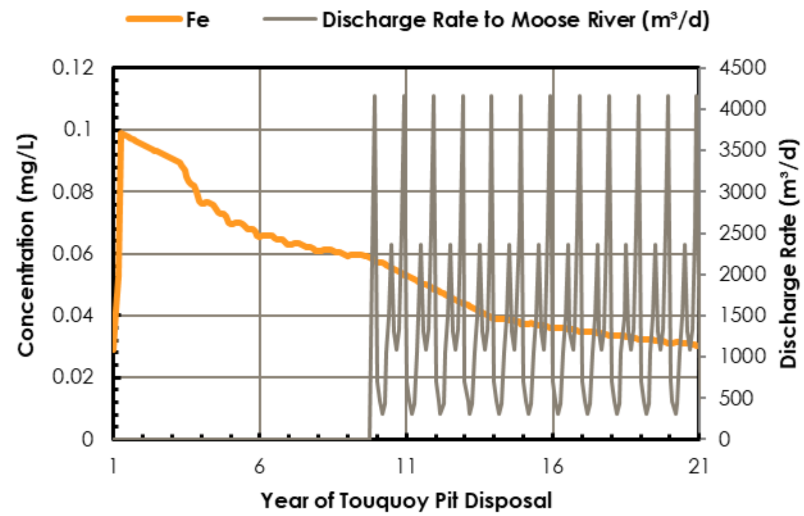
Concentration (mg/L)
Discharge Rate (m³/d)
Year of Touquoy Pit Disposal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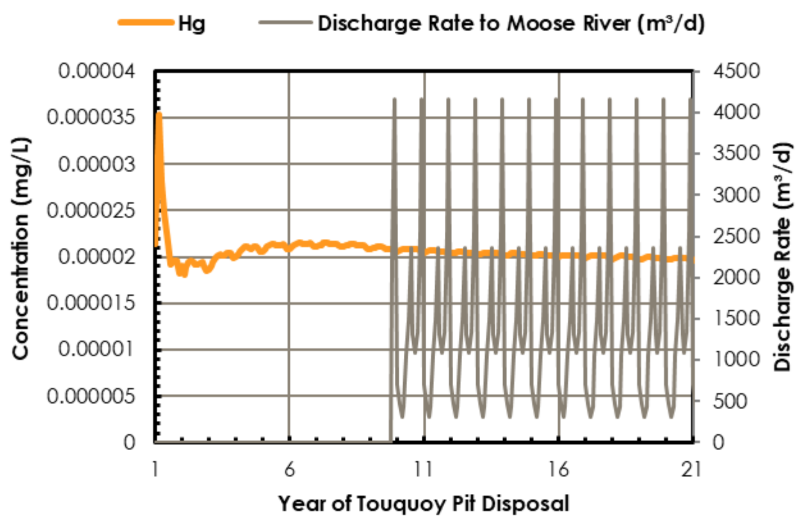
Concentration (mg/L)
Discharge Rate (m³/d)
Year of Touquoy Pit Disposal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Discharge Rate to Moose River (m³/d)
Concentration (mg/L)
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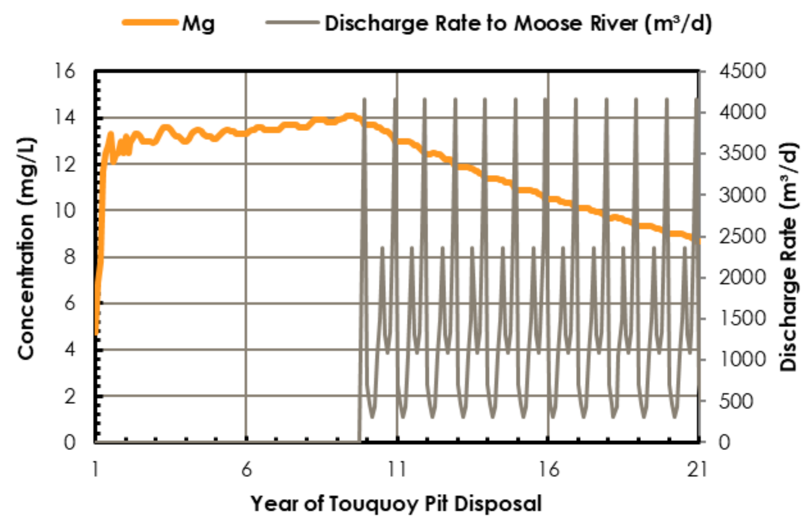
Year of Touquoy Pit Disposal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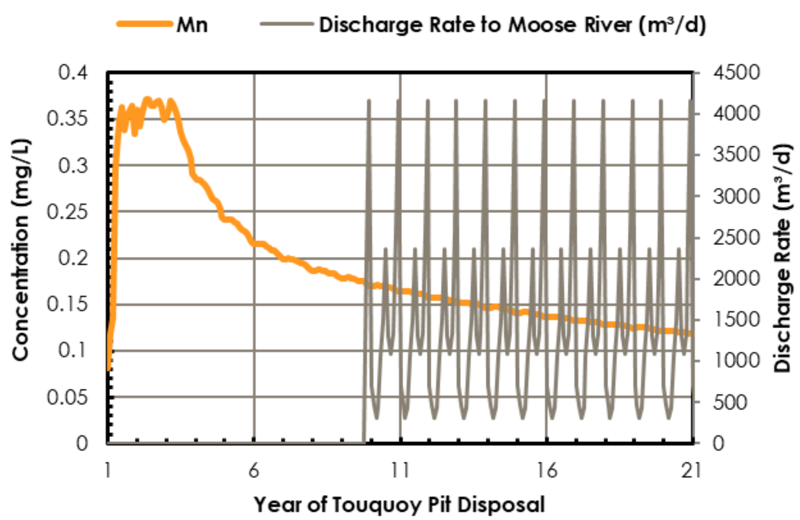
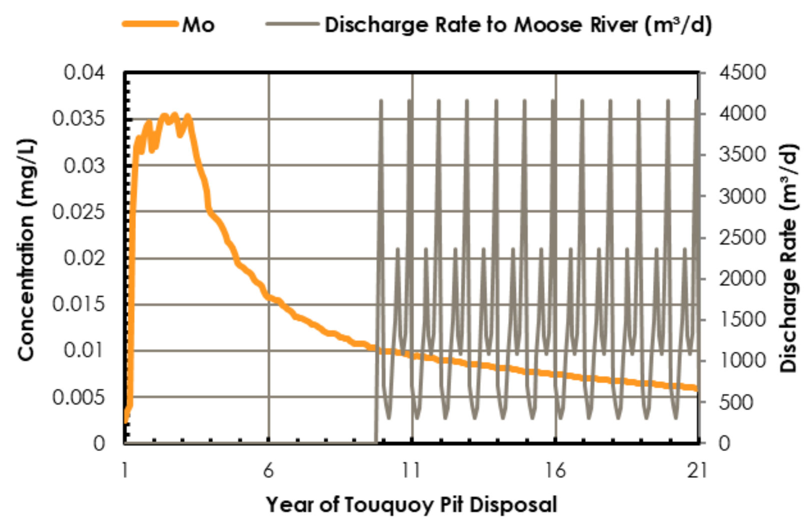
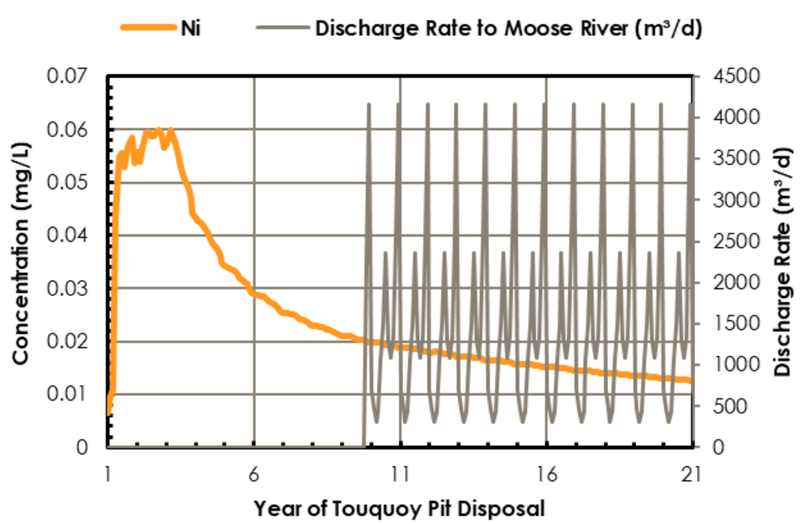
Discharge Rate to Moose River (m³/d)
Concentration (mg/L)
Discharge Rate (m³/d)
Year of Touquoy Pit Disposal



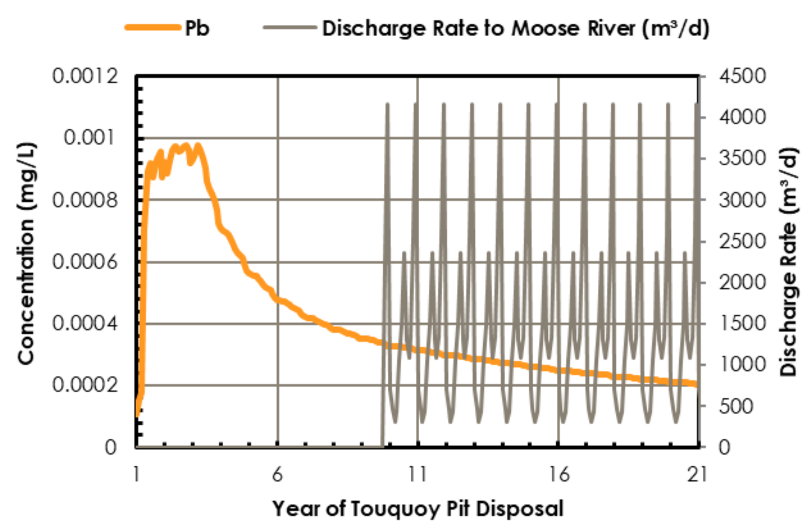
Mn
Discharge Rate to Moose River (m³/d)
Concentration (mg/L)
Discharge Rate (m³/d)
Year of Touquoy Pit Disposal



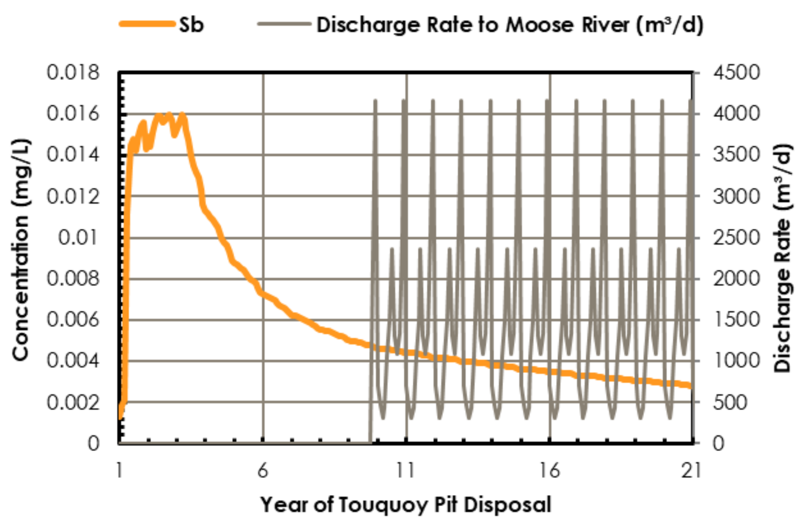
Mo
Discharge Rate to Moose River (m³/d)
Concentration (mg/L)
Discharge Rate (m³/d)
Year of Touquoy Pit Disposal

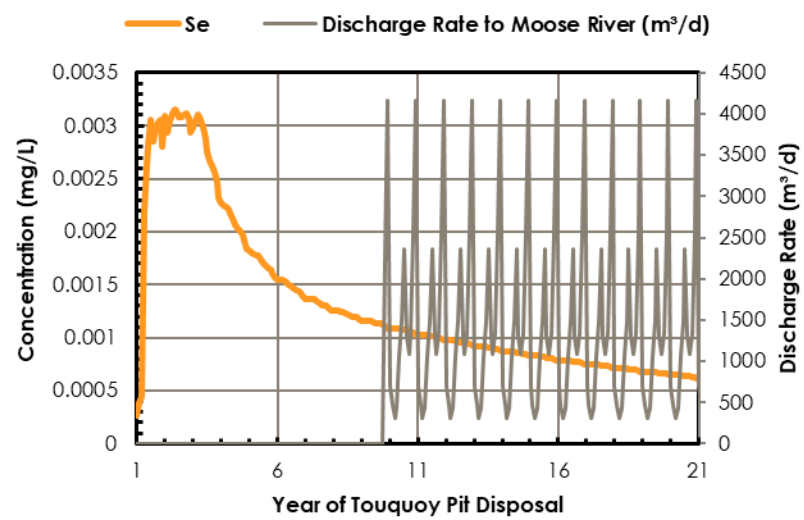


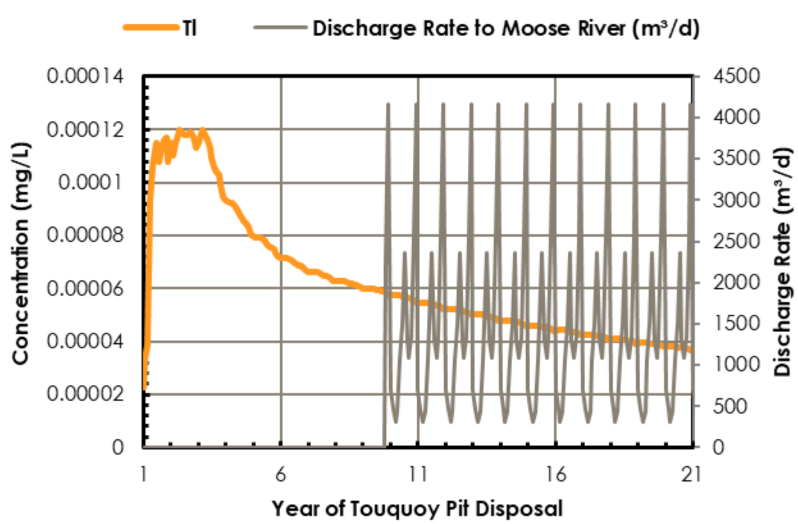
Ni
Discharge Rate to Moose River (m³/d)
Concentration (mg/L)
Discharge Rate (m³/d)
Year of Touquoy Pit Disposal

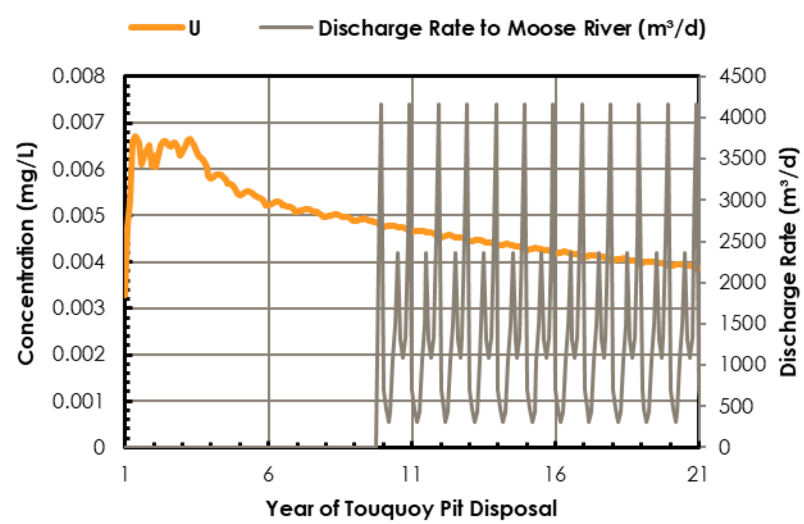


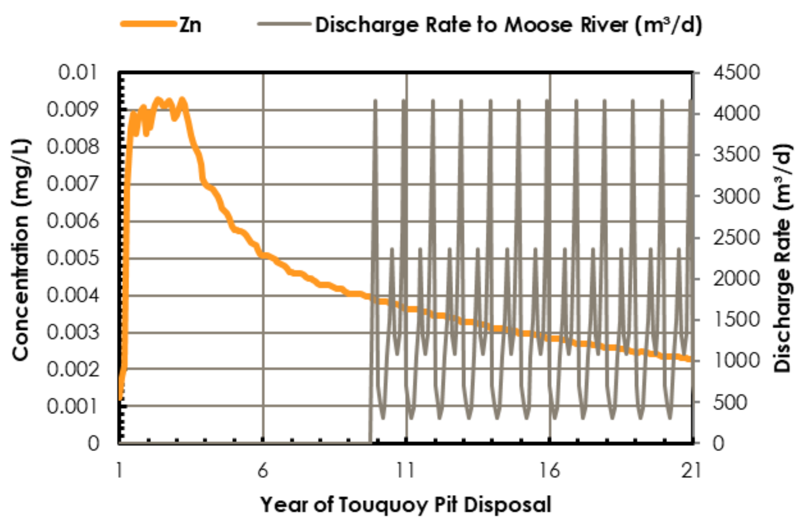
Pb
Discharge Rate to Moose River (m³/d)
Concentration (mg/L)
Discharge Rate (m³/d)
Year of Touquoy Pit Disposal

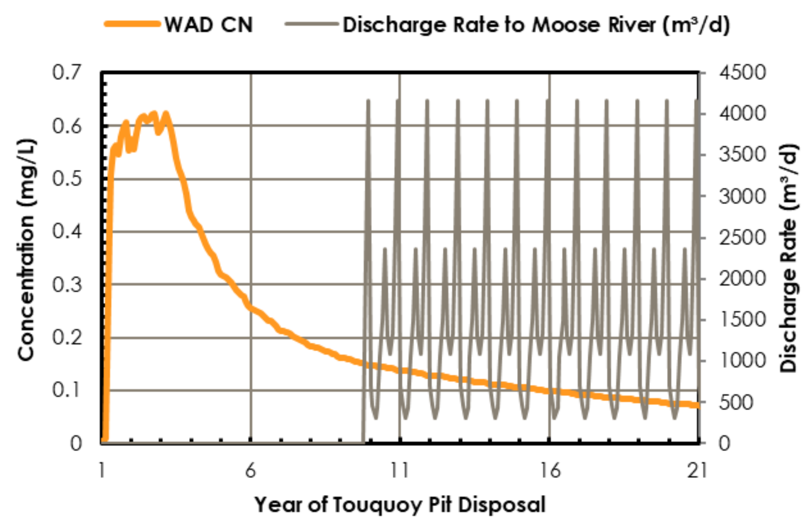

Sb
Discharge Rate to Moose River (m³/d)
Concentration (mg/L)
Discharge Rate (m³/d)
Year of Touquoy Pit Disposal

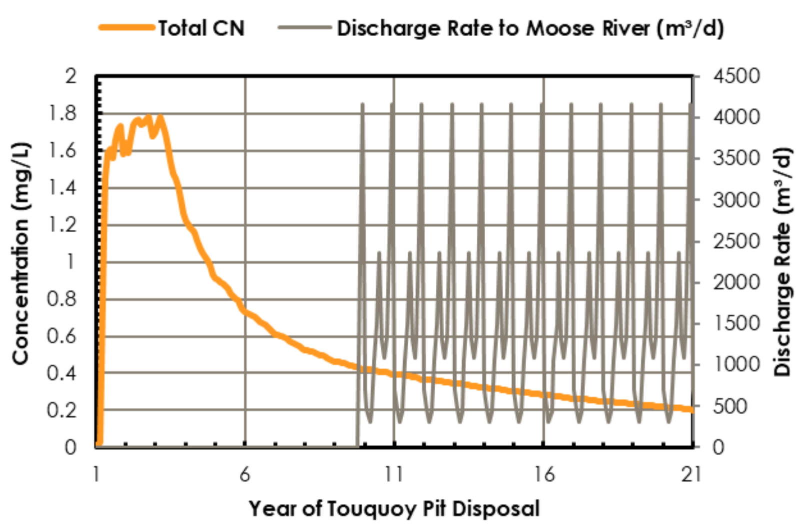

Se
Discharge Rate to Moose River (m³/d)
Concentration (mg/L)
Discharge Rate (m³/d)
Year of Touquoy Pit Disposal

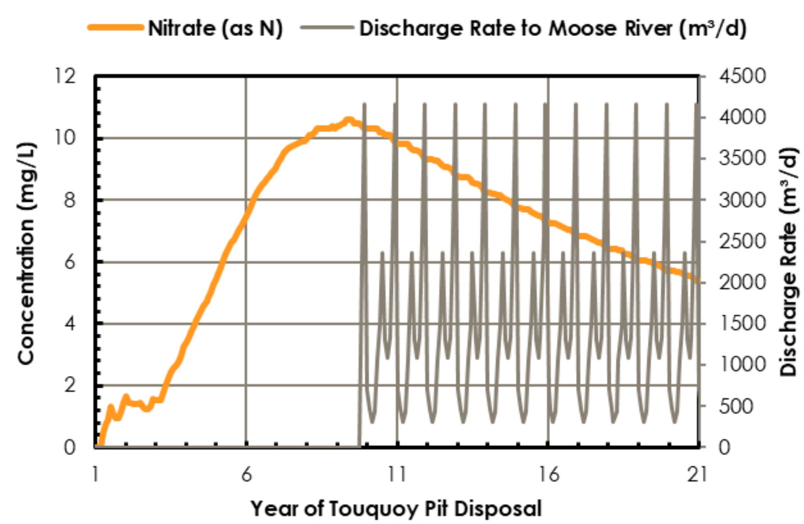

Tl
Discharge Rate to Moose River (m³/d)
Concentration (mg/L)
Discharge Rate (m³/d)
Year of Touquoy Pit Disposal

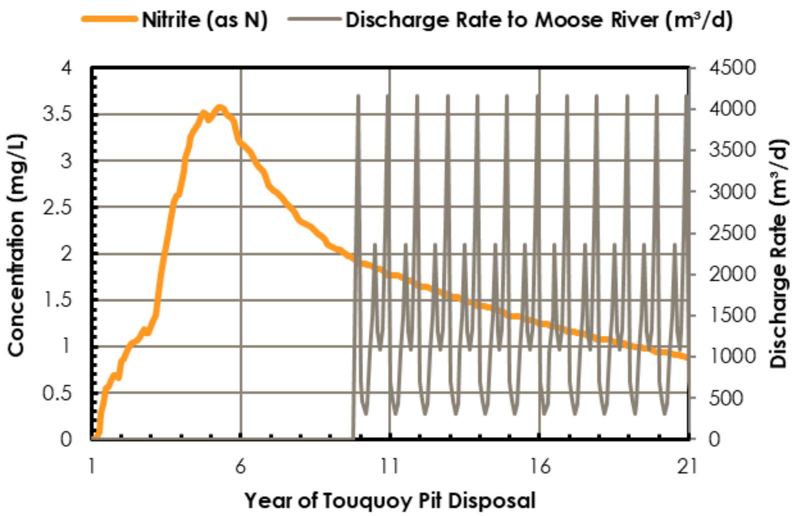

U
Discharge Rate to Moose River (m³/d)
Concentration (mg/L)
Discharge Rate (m³/d)
Year of Touquoy Pit Disposal

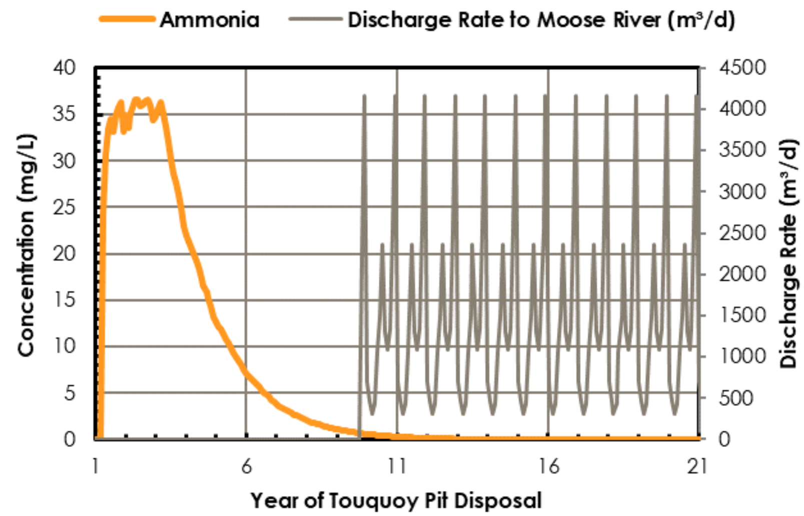

Zn
Discharge Rate to Moose River (m³/d)
Concentration (mg/L)
Discharge Rate (m³/d)
Year of Touquoy Pit Disposal


WAD CN
Discharge Rate to Moose River (m³/d)
Concentration (mg/L)
Discharge Rate (m³/d)
Year of Touquoy Pit Disposal


Total CN
Discharge Rate to Moose River (m³/d)
Concentration (mg/L)
Discharge Rate (m³/d)
Year of Touquoy Pit Disposal


Nitrate (as N)
Discharge Rate to Moose River (m³/d)
Concentration (mg/L)
Discharge Rate (m³/d)
Year of Touquoy Pit Disposal


Nitrite (as N)
Discharge Rate to Moose River (m³/d)
Concentration (mg/L)
Discharge Rate (m³/d)
Year of Touquoy Pit Disposal
0
0.5
1
1.5
2
2.5
3
3.5
4
0
500
1000
1500
2000
2500
3000
3500
4000
4500
1
6
11
16
21


Ammonia
Discharge Rate to Moose River (m³/d)
Concentration (mg/L)
Discharge Rate (m³/d)
Year of Touquoy Pit Disposal
0
5
10
15
20
25
30
35
40
0
500
1000
1500
2000
2500
3000
3500
4000
4500
1
6
11
16
21

APPENDIX/DIVIDER TITLE

APPENDIX A.2
UPDATE ON THE SLOPE STABILITY ASSESSMENT AND DESIGN OF THE WASTE ROCK STORAGE FACILITY OF TOUQUOY MINE

CONFIDENTIAL

TECHNICAL MEMORANDUM

DATE November 27, 2020 **Project No.** GAL0013-1781033-Rev0

TO Diana Sellier-Gomez, EIT, M.A.Sc., Geotechnical Engineer
Atlantic Gold Corporation

FROM Mohammad Kermani, Yves Boulianne,
Marc Rougier **EMAIL mkermani@golder.com**

CC

UPDATE ON THE SLOPE STABILITY ASSESSMENT AND DESIGN OF THE WASTE ROCK STORAGE FACILITY OF TOUQUOY MINE

1.0 INTRODUCTION

This technical memorandum summarizes the re-evaluation of the geotechnical stability of the waste rock storage facility (WRSF) of Touquoy Mine following Atlantic Gold Corporation's (AMNS) request on October 9, 2020. The review addresses minimal changes to the configuration of the slopes based on the drawing provided by AMNS.

The original design of the WRSF (Golder 2020) considered constant slopes of 3H:1V up to an elevation of 170 m. According to the update received from AMNS, in order to comply with commitments to the regulators, the final slope configuration requires 10 m high lifts with 2.7H:1V face slopes and catch benches of 3 m or 4 m, depending on the zone.

Figure 1 at the end of the text (Attachment A) shows the zones with different final configurations and the location of the cross-sections re-evaluated.

2.0 SCOPE

The three cross sections studied in the original WRSF stability assessment report (Golder 2020) were reanalyzed for 10 m high lifts with 2.7H:1V slopes and 3 m wide catch benches between lifts. The cross section 1-1' in the northeastern side of the WRSF falls in the zone with 4 m wide catch benches. However, for the cross section to be generic and representative of all of the northeastern zone, catch benches of 3 m were considered in the analyses as a conservative approach for purpose of factor of safety calculation. In other words, a steeper global slope was considered so that the studied cross section covers the whole northeastern part of the WRSF.

For the work described above, no change was made to the site conditions and the design criteria, sections 3.0 and 5.0 of the original design report (Golder, 2020) respectively. Similarly, no change was applied to the section 4 of Golder (2020), WRSF's conditions, except for the final profile (geometry).

3.0 RESULTS AND RECOMMENDATIONS

Assuming that the undrained strength analyses (USA) control the design, the USA case analyses were rerun with the update on the geometry. Table 1 presents the updated results of the stability analyses. There are no significant changes in the obtained factors of safety. Obtained factors of safety, the same as the required factors of safety, are presented to one decimal in this table. The potential slip surfaces as considered by the slope stability software are presented in attachment B of this memorandum.

As stated in the original design report (Golder, 2020, section 6.2), the assumption of pore pressure coefficient $\bar{B} = 1$ (the ratio of the generated pore water pressure over the applied vertical total stress) remains very conservative, and the FoS<1 for the 170m final crest elevation does not mean that the slope will be unstable, because that high degree of pore pressure is unlikely. In the absence of in-situ data to monitor the pore water pressure and as best practice, we recommend observational method stability monitoring during construction.

In the original design with continued 3H:1V slopes, the cross section 2-2' in the south-eastern side of the WRSF included a 15 m bench at elevation 150 m. With 2.7H:1V face slopes, this bench should be 20 m wide in to satisfy the stability criteria. This zone is shown with blue colour in the south-eastern side of the WRSF.

The construction must be supervised by an engineer. If foundation soil or embankment deformations occur, the construction activities must cease, and the situation reassessed by a geotechnical engineer (see section 7.0 in Golder 2020 for all recommendations).

Table 1: Factors of Safety against Sliding for Maximum Attainable Elevation, 170 m (USA analysis)

Cross-Section	Condition	Required Factor of Safety	2.7H : 1V Slope with 3 m wide bench	2.7H : 1V Slope with 3 m wide bench ; $\bar{B}$ =1	20 m Bench at 150 m Elevation	20 m Bench at 150 m Elevation; $\bar{B}$ =1
1-1'	Static	1.3	1.3	1.2	-	-
	Pseudo-static	1.1	1.3	-	-	-
	Post-liquefaction (Seismic/Static)	1.1	1.1	-	-	-
2-2'	Static – Circular	1.3	-	-	1.4	1.4
	Static – Non-circular	1.3	1.2	-	1.3	<1
	Pseudo-static	1.1	-	-	1.2	-
	Post-liquefaction (Seismic/Static)	1.1	-	-	1.1	-

Cross-Section	Condition	Required Factor of Safety	2.7H : 1V Slope with 3 m wide bench	2.7H : 1V Slope with 3 m wide bench ; $\overline{B}$ =1	20 m Bench at 150 m Elevation	20 m Bench at 150 m Elevation; $\overline{B}$ =1
3-3'	Static – Circular	1.3	1.5	1.5	-	-
	Static – Non-circular	1.3	1.3	<1	-	-
	Pseudo-static	1.1	1.2	-	-	-
	Post-liquefaction (Seismic/Static)	1.1	1.1	-	-	-

4.0 CONCLUSION

The geotechnical stability of the WRSF was reassessed by considering 10 m high lifts with 2.7H:1V face slopes and 3 m wide catch benches between lifts as the final configuration. Given that this geometry is overall 3:1 for 3 m catch benches and 3.1: 1 for 4m catch benches, the factors of safety results are the same as those for 3:1 unbenched slopes from Golder 2020. In the south-eastern side of the WRSF, a 20 m wide catch bench at elevation 150 m must be included in order to satisfy the stability criteria. During construction, the slope and foundation performance should be monitored and documented by the on-site geotechnical engineer. Should signs of deformation (as elaborated in section 7 of Golder 2020) be observed, construction activities should cease, and the conditions reassessed.

5.0 REFERENCES

Golder 2020. *Waste Rock Storage Facility of Touquoy Mine, Slope Stability Assessment and Design*. Ref. No. 005-18108591_RA_Rev0. April 8th, 2020.

6.0 CLOSURE

We trust that this report meets your current needs. Please contact us should you have any questions or need additional information.

Respectfully Submitted,

Golder Associates Ltd.

Mohammad Kermani, Ph.D., P.Eng (Qc)
Geotechnical Engineer

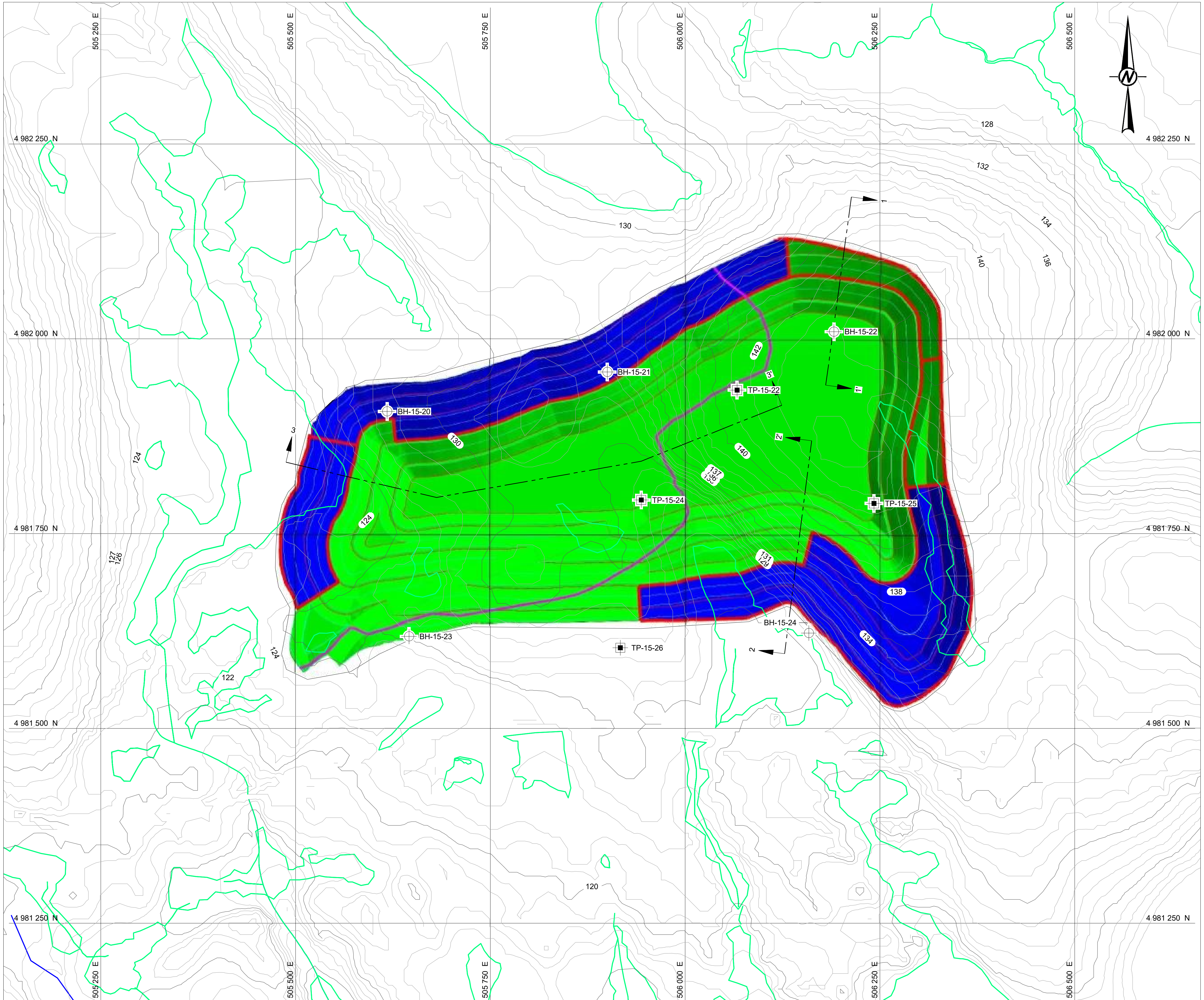
Yves Boulianne, P.Eng.(Qc)
Principal, Senior Geotechnical Engineer

Marc Rougier, P.Eng (NS)
Principal, Senior Geotechnical Engineer

MK/YB/MR/cd

Attachments:

- Attachment A: Figure 1: Studied cross sections
- Attachment B: Stability analyses results



LEGEND

- BOREHOLE LOCATIONS
- TEST PIT LOCATIONS
- TOPOGRAPHIC CONTOUR (INTERVAL 1 m)
- WATERCOURSE
- WETLAND
- 2.7:1 WITH 3 m BENCH
- 2.7:1 WITH 4 m BENCH

NOT FOR CONSTRUCTION

NOTE

COORDINATES SYSTEM: UTM NAD83, ZONE 20

REFERENCES

- DRAWING "1m LIDAR surface.dxf"
- FILE "Touquoy mine.jpg"

0 100 200

1:5 000 METRES

CLIENT

ATLANTIC MINING NS CORP.
TOUQUOY GOLD MINE

PROJECT

WASTE ROCK STORAGE FACILITY
GEOTECHNICAL STABILITY REVIEW

TITLE

STUDIED CROSS-SECTIONS

CONSULTANT

GOLDER

YYYY-MM-DD	2020-11-06
DESIGNED	R. Gravel
PREPARED	E. Nkamegue
REVIEWED	M. Kermani
APPROVED	Y. Boulianne

PROJECT NO.	PHASE	REV.	FIGURE
1781033	7000	0	1

STABILITY ANALYSES
TOUQUOY MINE WASTE ROCK STORAGE FACILITY

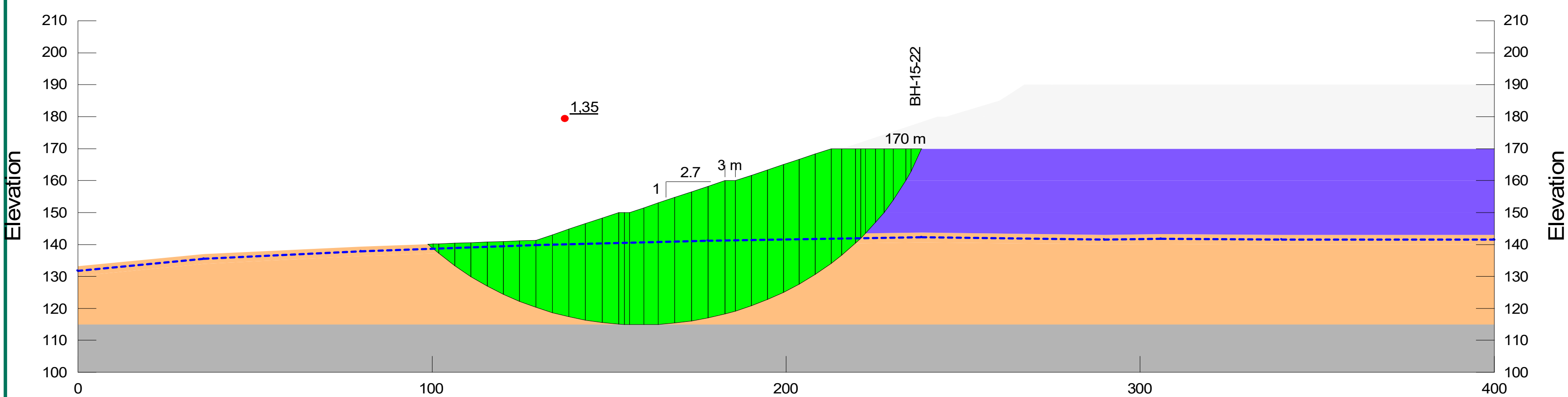
FIGURE B-1

Section 1-1' - 170 m
Static condition
USA peak

Color	Name	Model	Unit Weight (kN/m³)	Phi' (°)	Minimum Strength (kPa)	Tau/Sigma Ratio
	01c - Glacial till - USA peak_sumin=120kPa	SHANSEP	21		120	0,22
	02 - Bedrock	Bedrock (Impenetrable)				
	03a - Waste rock	Mohr-Coulomb	21	35		

Type 3 failure

FS = 1.3



Date : November 2020
Project: 1781033-7000

Prepared by: EN/MK
Reviewed by: SB

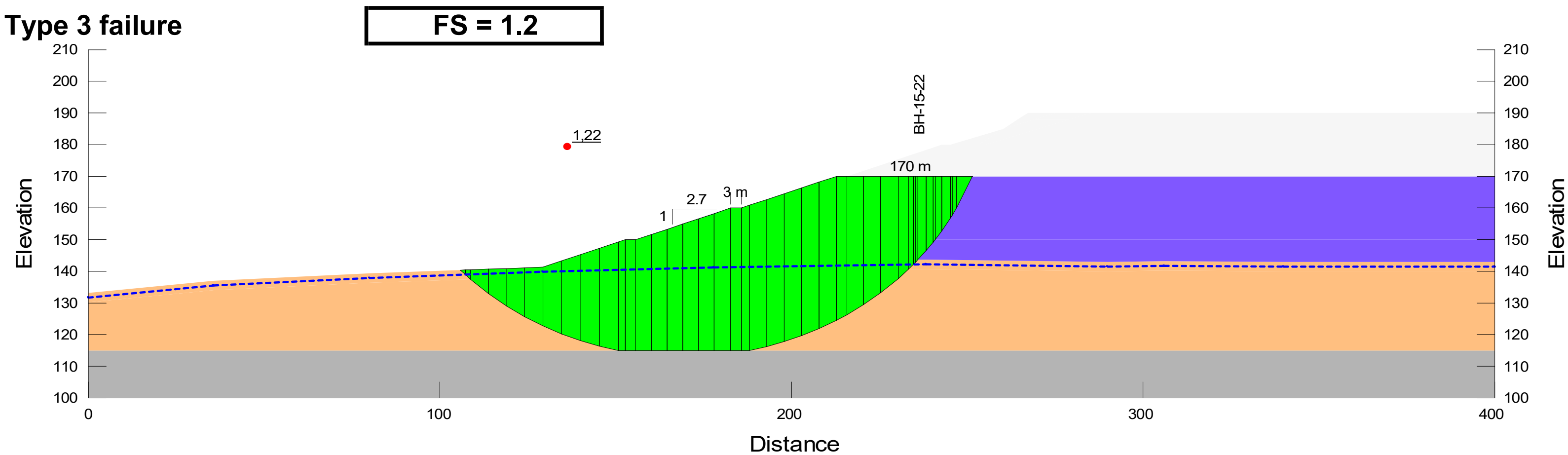
Golder Associés Ltée

STABILITY ANALYSES
TOUQUOY MINE WASTE ROCK STORAGE FACILITY

FIGURE B-2

Section 1-1' - 170 m
Static condition - $\bar{B}$= 1
USA peak

Color	Name	Model	Unit Weight (kN/m³)	Phi' (°)	Minimum Strength (kPa)	Tau/Sigma Ratio
	01c - Glacial till - USA peak_sumin=120kPa	SHANSEP	21		120	0,22
	02 - Bedrock	Bedrock (Impenetrable)				
	03a - Waste rock	Mohr-Coulomb	21	35		



Date : November 2020
Project: 1781033-7000

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Reviewed by: SB

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STABILITY ANALYSES
TOUQUOY MINE WASTE ROCK STORAGE FACILITY

FIGURE B-3

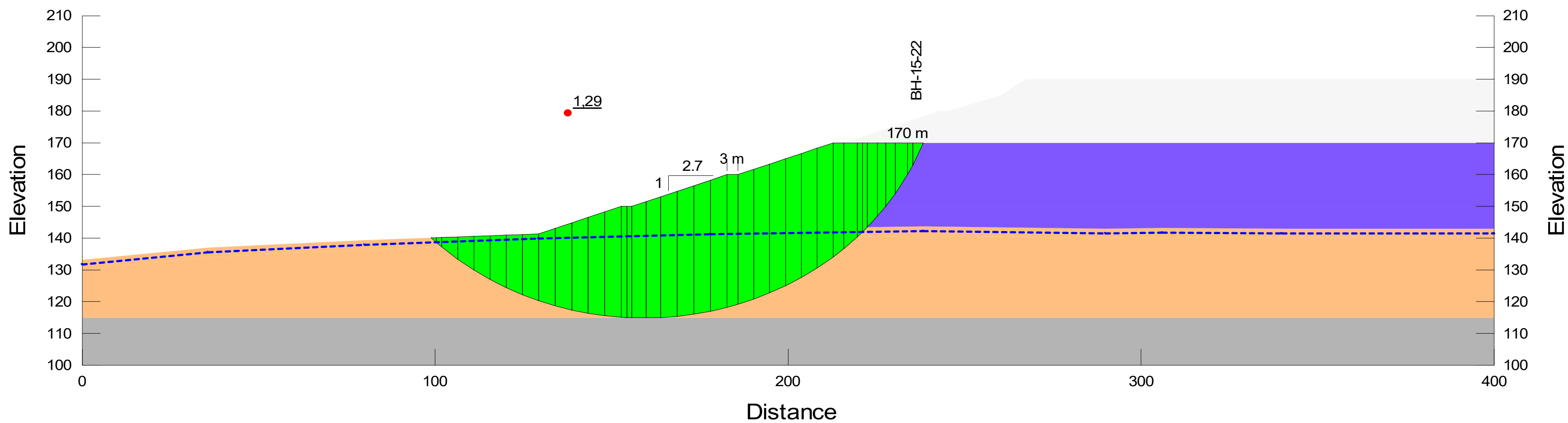
Section 1-1' - 170 m
Pseudo-static condition
USA peak

Color	Name	Model	Unit Weight (kN/m³)	Phi' (°)	Minimum Strength (kPa)	Tau/Sigma Ratio
	01c - Glacial till - USA peak_sumin=120kPa	SHANSEP	21		120	0,22
	02 - Bedrock	Bedrock (Impenetrable)				
	03a - Waste rock	Mohr-Coulomb	21	35		

kh = 0.115

Type 3 failure

FS = 1.3



Date : November 2020
Project: 1781033-7000

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Reviewed by: MK

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STABILITY ANALYSES
TOUQUOY MINE WASTE ROCK STORAGE FACILITY

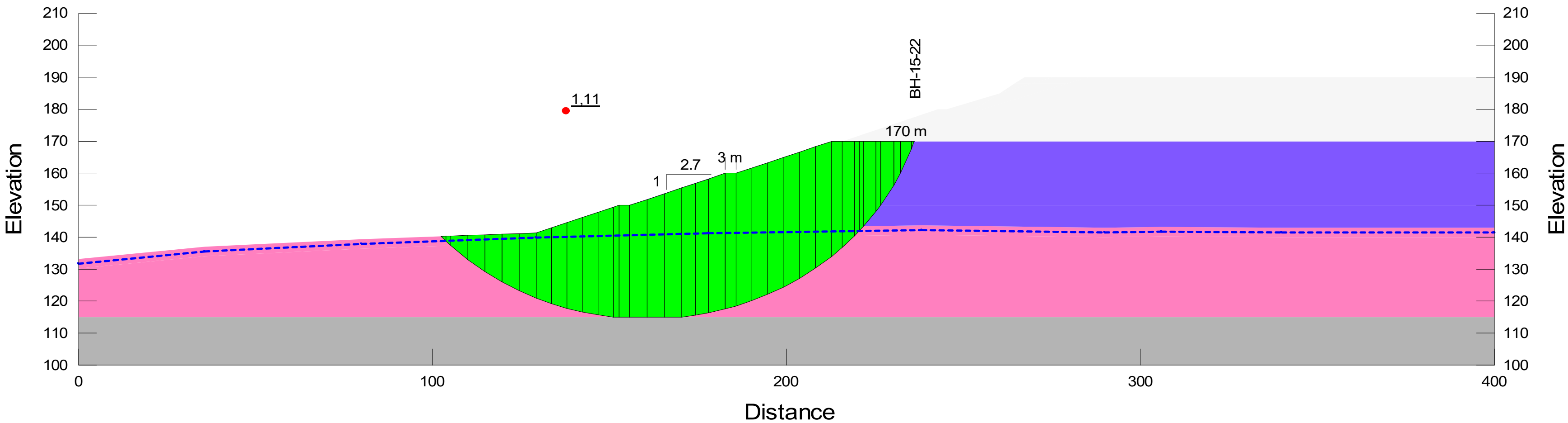
FIGURE B-4

Section 1-1' - 170 m
Post-seismic condition
USA post-peak

Color	Name	Model	Unit Weight (kN/m³)	Phi' (°)	Minimum Strength (kPa)	Tau/Sigma Ratio
	01g - Glacial till - USA post-peak	SHANSEP	21		96	0,176
	02 - Bedrock	Bedrock (Impenetrable)				
	03a - Waste rock	Mohr-Coulomb	21	35		

Type 3 failure

FS = 1.1



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Project: 1781033-7000

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Reviewed by: MK

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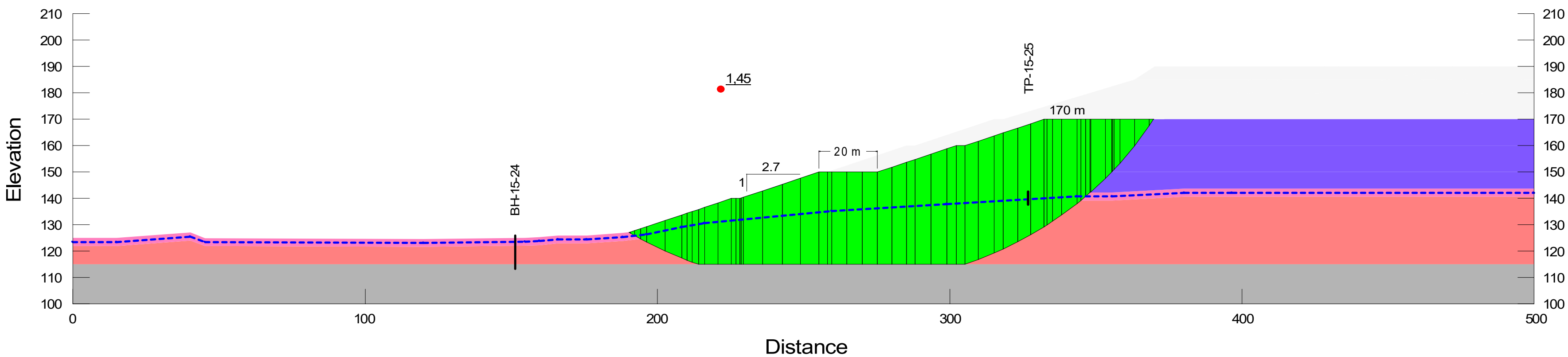
STABILITY ANALYSES
TOUQUOY MINE WASTE ROCK STORAGE FACILITY

FIGURE B-5

Section 2-2' - 170 m - 15 m bench at 150 m
Static condition
USA peak

Color	Name	Model	Unit Weight (kN/m³)	Phi' (°)	Minimum Strength (kPa)	Tau/Sigma Ratio
	01b - Glacial till - USA peak_sumin=50 kPa	SHANSEP	21		50	0.22
	01e - Glacial till - USA post-peak_sumin=170 kPa	SHANSEP	21		170	0.22
	02 - Bedrock	Bedrock (Impenetrable)				
	03a - Waste rock	Mohr-Coulomb	21	35		

Type 3 failure **1.4**



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Project: 1781033-7000

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Reviewed by: MK

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STABILITY ANALYSES
TOUQUOY MINE WASTE ROCK STORAGE FACILITY

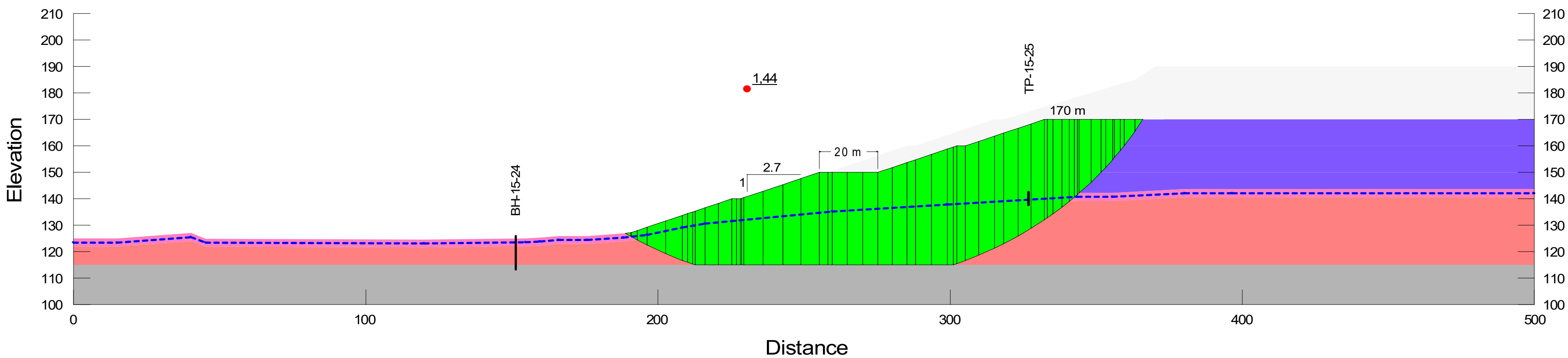
FIGURE B-6

Section 2-2' - 170 m - 15 m bench at 150 m
Static condition - $\bar{B}$= 1
USA peak

Color	Name	Model	Unit Weight (kN/m³)	Phi' (°)	Minimum Strength (kPa)	Tau/Sigma Ratio
	01b - Glacial till - USA peak_sumin=50 kPa	SHANSEP	21		50	0.22
	01e - Glacial till - USA post-peak_sumin=170 kPa	SHANSEP	21		170	0.22
	02 - Bedrock	Bedrock (Impenetrable)				
	03a - Waste rock	Mohr-Coulomb	21	35		

Type 3 failure

FS = 1.4



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Project: 1781033-7000

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Reviewed by: MK

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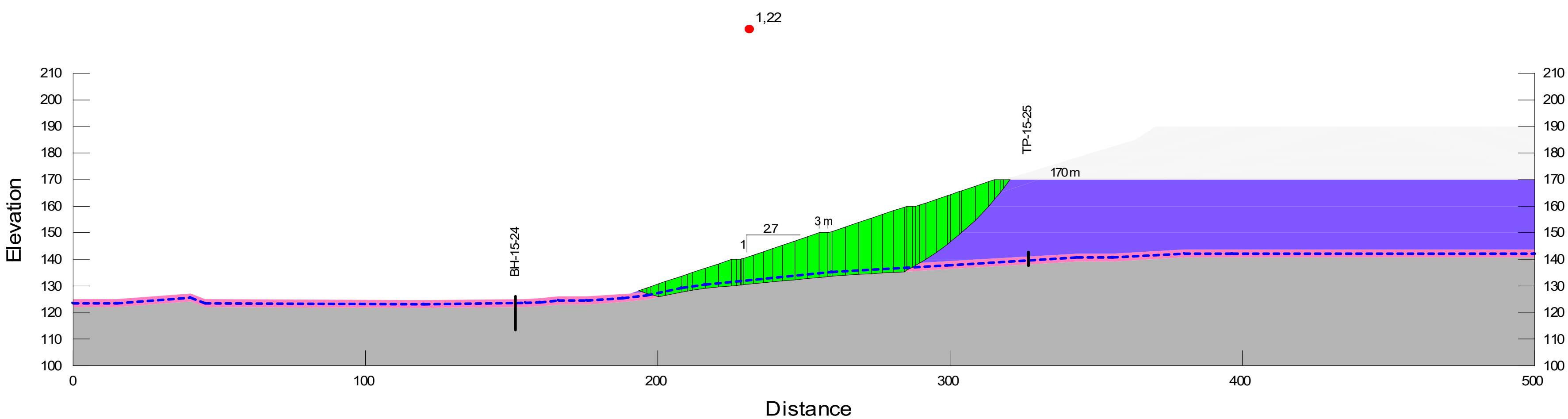
STABILITY ANALYSES
TOUQUOY MINE WASTE ROCK STORAGE FACILITY

FIGURE B-7

Section 2-2' - 170 m
Static condition
USA peak

Color	Name	Model	Unit Weight (kN/m³)	Phi' (°)	Minimum Strength (kPa)	Tau/Sigma Ratio
	01b - Glacial till - USA peak_sumin=50 kPa	SHANSEP	21		50	0,22
	02 - Bedrock	Bedrock (Impenetrable)				
	03a - Waste rock	Mohr-Coulomb	21	35		

Non circular failure **FS = 1.2**



Date : November 2020
Project: 1781033-7000

Prepared by: EN/MK
Reviewed by: MK

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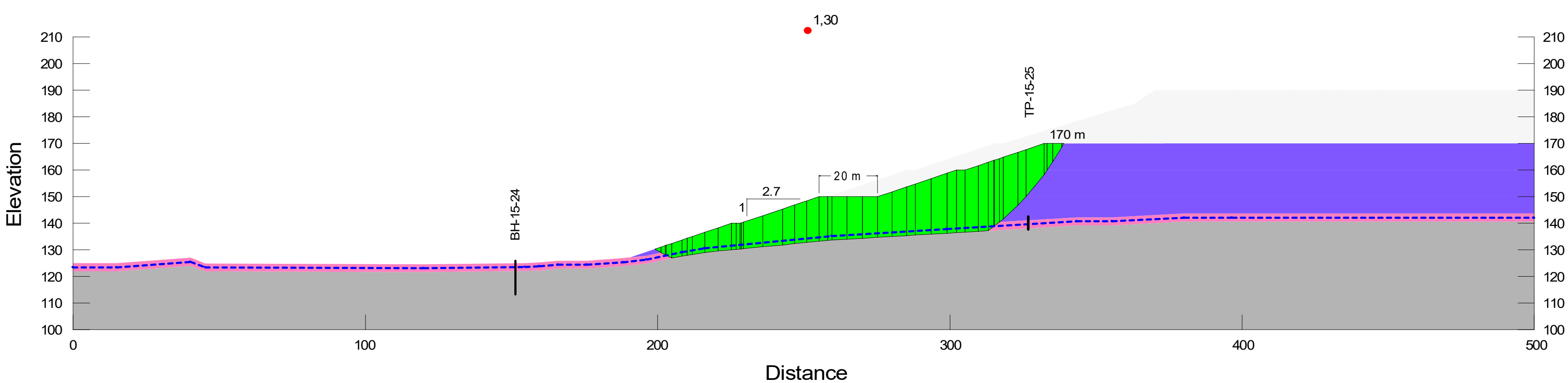
STABILITY ANALYSES
TOUQUOY MINE WASTE ROCK STORAGE FACILITY

FIGURE B-8

Section 2-2' - 170 m - 15 m bench at 150 m
Static condition
USA peak

Color	Name	Model	Unit Weight (kN/m³)	Phi' (°)	Minimum Strength (kPa)	Tau/Sigma Ratio
	01b - Glacial till - USA peak_sumin=50 kPa	SHANSEP	21		50	0,22
	02 - Bedrock	Bedrock (Impenetrable)				
	03a - Waste rock	Mohr-Coulomb	21	35		

Non circular failure **FS = 1.3**



Date : November 2020
Project: 1781033-7000

Prepared by: EN/MK
Reviewed by: MK

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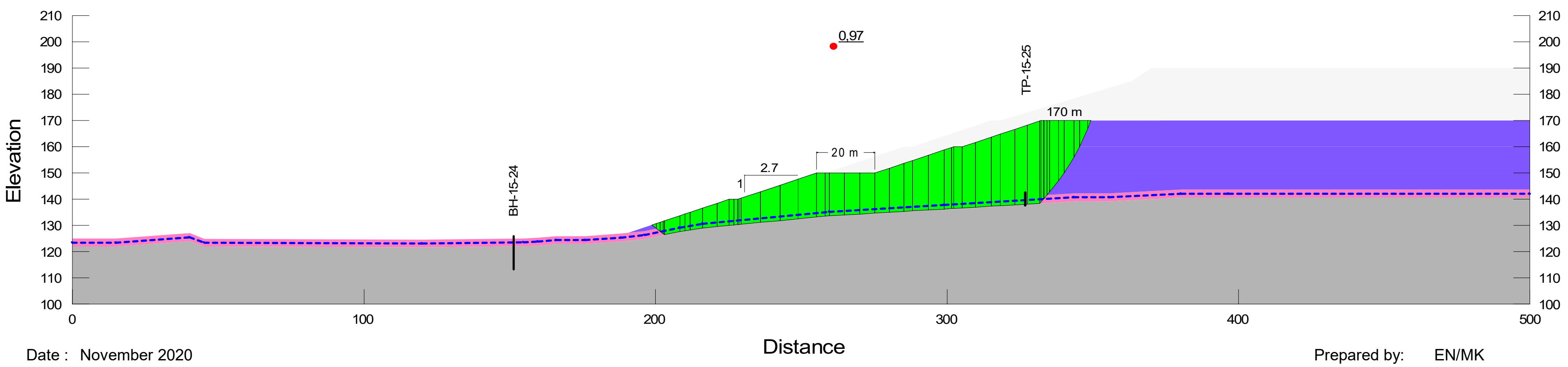
STABILITY ANALYSES
TOUQUOY MINE WASTE ROCK STORAGE FACILITY

FIGURE B-9

Section 2-2' - 170 m - 15 m bench at 150 m
Static condition - $\bar{B}$= 1
USA peak

Color	Name	Model	Unit Weight (kN/m³)	Phi' (°)	Minimum Strength (kPa)	Tau/Sigma Ratio
	01b - Glacial till - USA peak_sumin=50 kPa	SHANSEP	21		50	0,22
	02 - Bedrock	Bedrock (Impenetrable)				
	03a - Waste rock	Mohr-Coulomb	21	35		

Non circular failure **FS < 1**



Date : November 2020
Project: 1781033-7000

Prepared by: EN/MK
Reviewed by: MK

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STABILITY ANALYSES
TOUQUOY MINE WASTE ROCK STORAGE FACILITY

FIGURE B-10

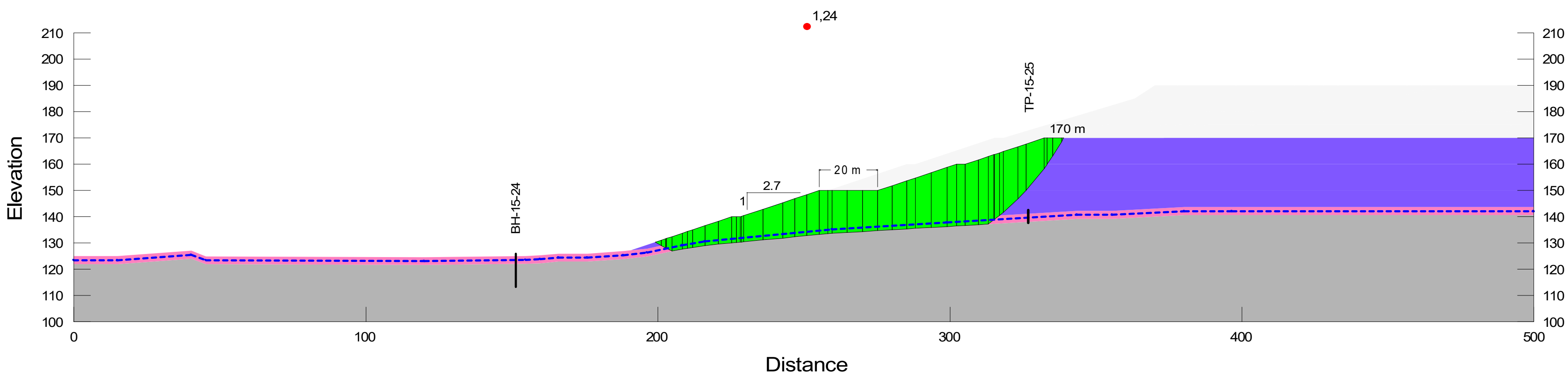
Section 2-2' - 170 m
Pseudo-static condition
USA peak

Color	Name	Model	Unit Weight (kN/m³)	Phi' (°)	Minimum Strength (kPa)	Tau/Sigma Ratio
	01b - Glacial till - USA peak_sumin=50 kPa	SHANSEP	21		50	0,22
	02 - Bedrock	Bedrock (Impenetrable)				
	03a - Waste rock	Mohr-Coulomb	21	35		

kh = 0.0115

Non circular failure

FS = 1.2



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Project: 1781033-7000

Prepared by: EN/MK
Reviewed by: MK

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STABILITY ANALYSES
TOUQUOY MINE WASTE ROCK STORAGE FACILITY

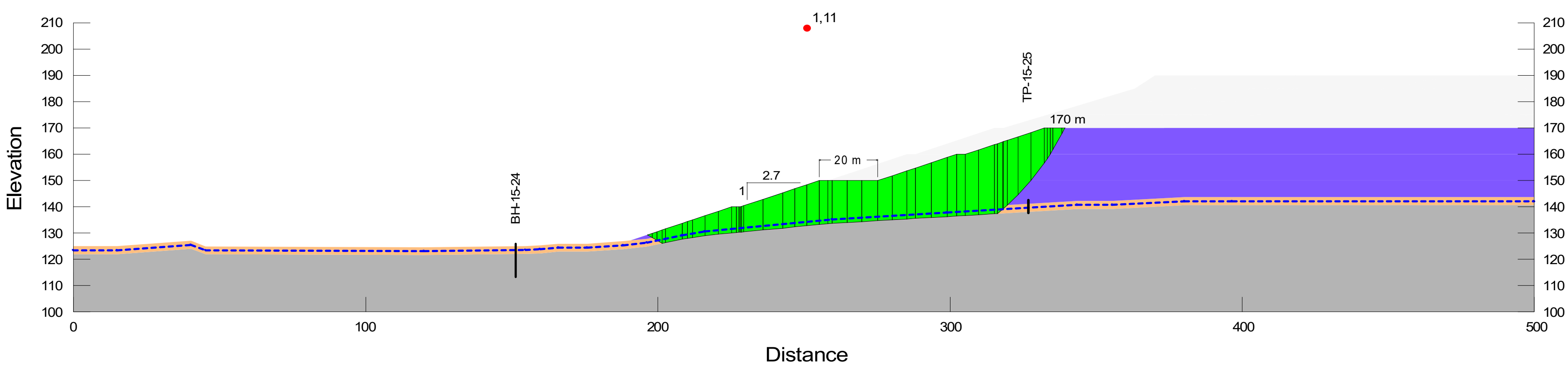
FIGURE B-11

Section 2-2' - 170 m
Post-seismic condition
USA post-peak

Color	Name	Model	Unit Weight (kN/m³)	Phi' (°)	Minimum Strength (kPa)	Tau/Sigma Ratio
	01g - Glacial till - USA post-peak_sumin=40 kPa	SHANSEP	21		40	0,176
	02 - Bedrock	Bedrock (Impenetrable)				
	03a - Waste rock	Mohr-Coulomb	21	35		

Non circular failure

FS = 1.1



Date : November 2020
Project: 1781033-7000

Prepared by: EN/MK
Reviewed by: MK

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STABILITY ANALYSES
TOUQUOY MINE WASTE ROCK STORAGE FACILITY

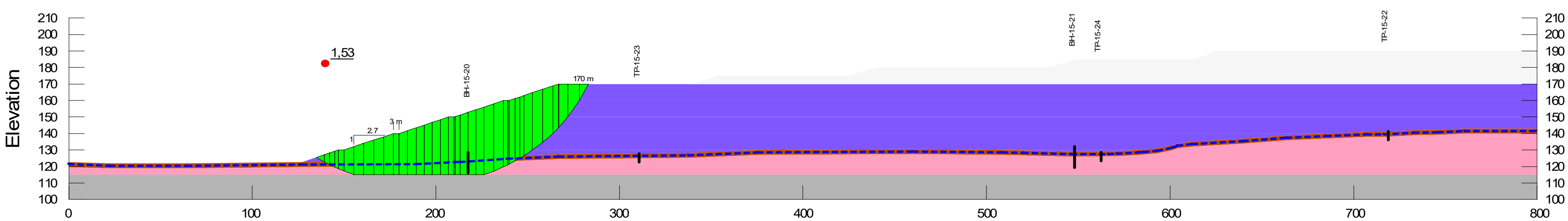
FIGURE B-12

Section 3-3' - 170 m
Static condition
USA peak

Color	Name	Model	Unit Weight (kN/m³)	Phi' (°)	Minimum Strength (kPa)	Tau/Sigma Ratio
	01b - Glacial Till USA - sumin=50 kPa	SHANSEP	21		50	0.22
	01c - Glacial Till USA - sumin=170 kPa	SHANSEP	21		170	0.22
	02 - Bedrock	Bedrock (Impenetrable)				
	03a - Waste rock	Mohr-Coulomb	21	35		

Type 3 failure

FS = 1.53



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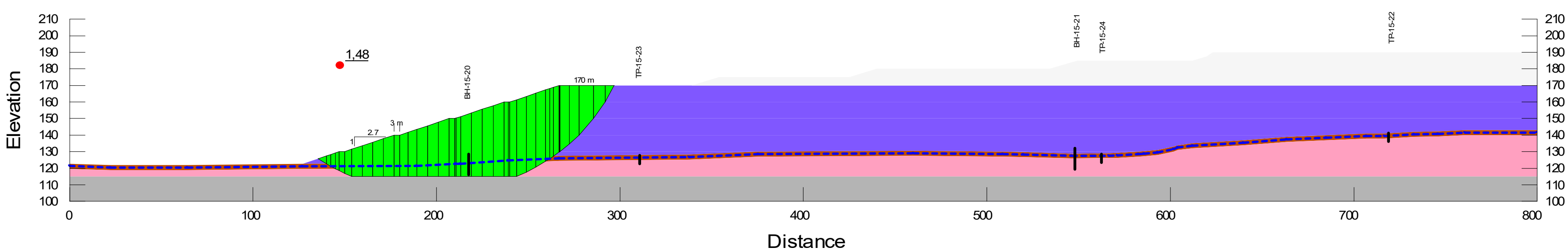
TOUQUOY MINE WASTE ROCK STORAGE FACILITY

FIGURE B-13

Section 3-3' - 170 m
Static condition - $\bar{B}$= 1
USA peak

Color	Name	Model	Unit Weight (kN/m³)	Phi' (°)	Minimum Strength (kPa)	Tau/Sigma Ratio
	01b - Glacial Till USA - sumin=50 kPa	SHANSEP	21		50	0.22
	01c - Glacial Till USA - sumin=170 kPa	SHANSEP	21		170	0.22
	02 - Bedrock	Bedrock (Impenetrable)				
	03a - Waste rock	Mohr-Coulomb	21	35		

Type 3 failure **FS = 1.48**



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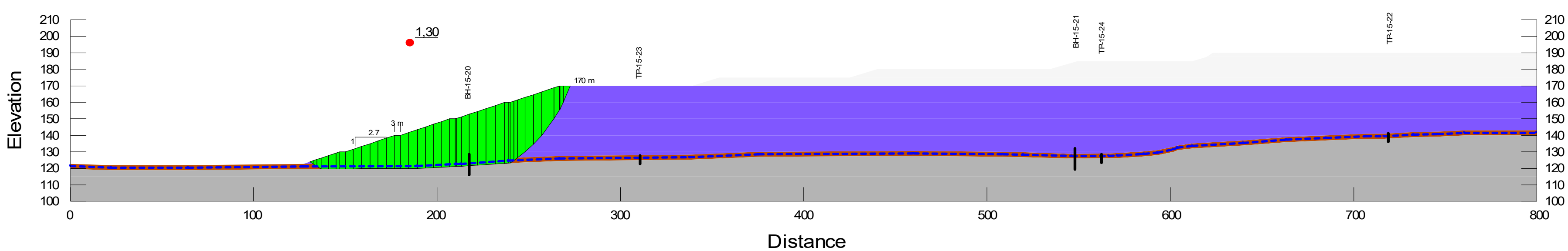
STABILITY ANALYSES
TOUQUOY MINE WASTE ROCK STORAGE FACILITY

FIGURE B-14

Section 3-3' - 170 m
Static condition
USA peak

Color	Name	Model	Unit Weight (kN/m³)	Phi' (°)	Minimum Strength (kPa)	Tau/Sigma Ratio
	01b - Glacial Till USA - sumin=50 kPa	SHANSEP	21		50	0,22
	02 - Bedrock	Bedrock (Impenetrable)				
	03a - Waste rock	Mohr-Coulomb	21	35		

Non circular failure **FS = 1.30**



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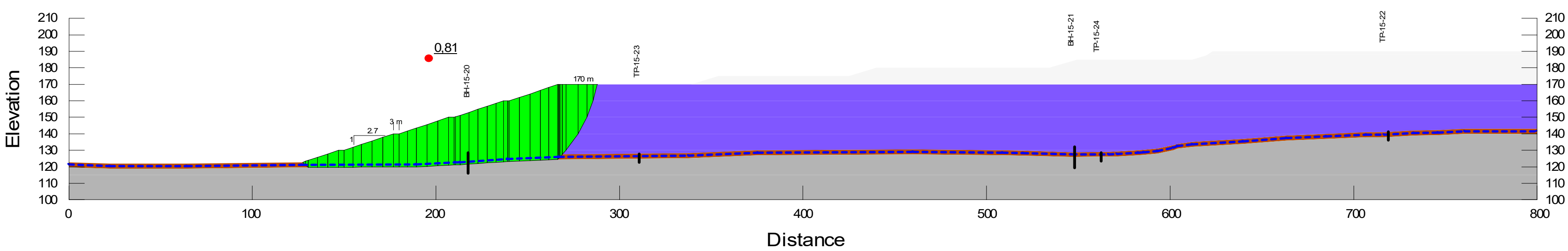
STABILITY ANALYSES
TOUQUOY MINE WASTE ROCK STORAGE FACILITY

FIGURE B-15

Section 3-3' - 170 m
Static condition - $\bar{B}$= 1
USA peak

Color	Name	Model	Unit Weight (kN/m³)	Phi' (°)	Minimum Strength (kPa)	Tau/Sigma Ratio
	01b - Glacial Till USA - sumin=50 kPa	SHANSEP	21		50	0,22
	02 - Bedrock	Bedrock (Impenetrable)				
	03a - Waste rock	Mohr-Coulomb	21	35		

Non circular failure **FS < 1**



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STABILITY ANALYSES
TOUQUOY MINE WASTE ROCK STORAGE FACILITY

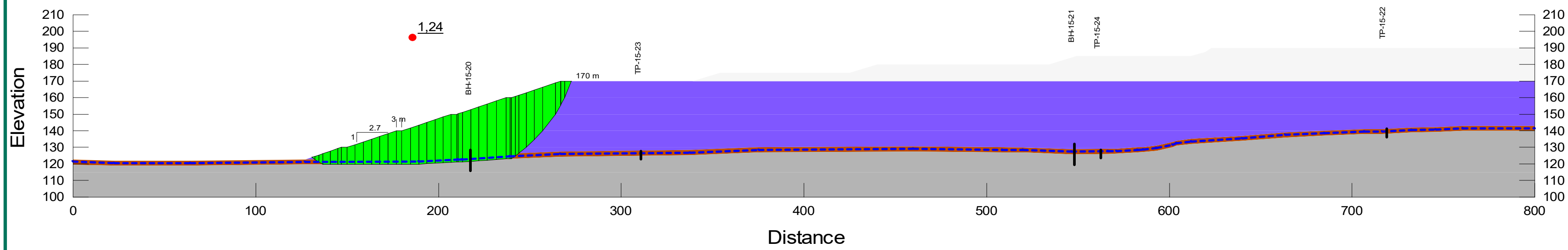
FIGURE B-16

Section 3-3' - 170 m
Pseudo-static condition
USA peak

Color	Name	Model	Unit Weight (kN/m³)	Phi' (°)	Minimum Strength (kPa)	Tau/Sigma Ratio
	01b - Glacial Till USA - sumin=50 kPa	SHANSEP	21		50	0,22
	02 - Bedrock	Bedrock (Impenetrable)				
	03a - Waste rock	Mohr-Coulomb	21	35		

kh = 0.0115

Non circular failure **FS = 1.2**



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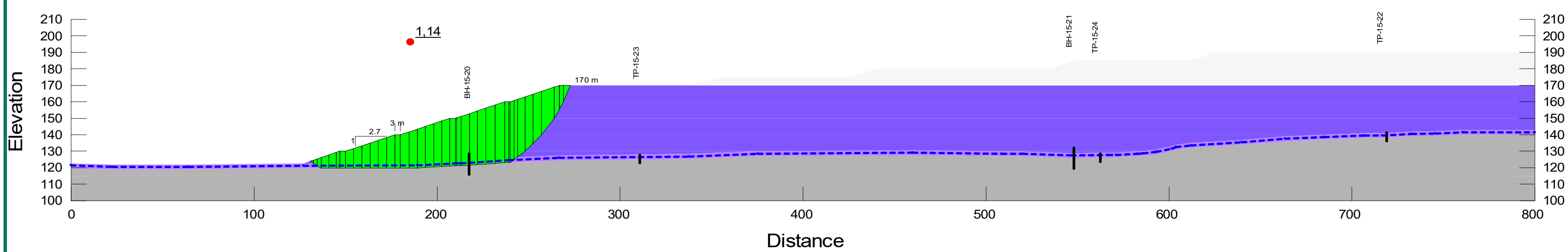
STABILITY ANALYSES
TOUQUOY MINE WASTE ROCK STORAGE FACILITY

FIGURE B-17

Section 3-3' - 170 m
Post-seismic condition
USA post-peak

Color	Name	Model	Unit Weight (kN/m³)	Phi' (°)	Minimum Strength (kPa)	Tau/Sigma Ratio
	01e - Glacial Till USA - post-peak - sumin=40 kPa	SHANSEP	21		40	0,176
	02 - Bedrock	Bedrock (Impenetrable)				
	03a - Waste rock	Mohr-Coulomb	21	35		

Non circular failure **FS = 1.1**



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GOLDER

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REPORT

Waste Rock Storage Facility of Touquoy Mine

Slope Stability Assessment and Design

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Since the composition and geometry of soil and bedrock horizons are always highly variable, the borehole and test pit descriptions can only provide an approximate estimation of their actual characteristics and profiles. The contacts between different soil and/or bedrock horizons are often gradual, and their location in borehole and test pit logs is to some extent the result of interpretation. Furthermore, the accuracy of the collected data and their interpretation depend on different factors, including the investigation method, spacing between soundings, depth of investigation, sampling method and frequency, as well as the uniformity of the underground conditions. Some of those factors, such as the investigation method, spacing between soundings, depth of investigation, sampling method and frequency can themselves be subordinated to the physical, financial or time constraints stipulated by Atlantic Mining NS Corp.

In all cases, the results obtained and presented in this report should be considered as applying only to the locations of the boreholes and test pits, at the indicated sampling depths and at the time of the investigation. The interpreted underground conditions, physical as well as quantitative or qualitative, can vary significantly between and beyond the drilled boreholes and test pits and the indicated sampling depths.

The groundwater measurements and characteristics given in this report are valid only for the specified locations and dates. These conditions can vary from one season or one year to another, or because of activities or occurrences on the investigated site or adjacent land.

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APPENDICES

1.0 INTRODUCTION

Atlantic Mining NS Corp. (AMNS) has retained Golder Associates Ltd. (Golder) to carry out the geotechnical stability analysis of the WRSF of Touquoy Mine. The objective is to improve the initial design carried out by Stantec (Stantec 2016b) by considering the latest layout and configuration of the facility, latest updates regarding the closure plan, to include the information gathered during placement of the waste rock to this point and the choice of appropriate criteria regarding seismic aspects. This report summarizes the design criteria, the assumptions and the results of the stability analyses. The study evaluates the feasibility of the design provided by AMNS from a geotechnical point of view, defines a maximum attainable elevation for the pile considering the existing data, and proposes the required amendment to meet the design criteria.

2.0 OBJECTIVES OF THE STUDY

The present report aims to evaluate the feasibility of the WRSF layout proposed by AMNS from a geotechnical point of view. The evaluation includes:

1) Classification of the WRSF according to the, “Guidelines for Mine Waste Dump and Stockpile Design,” Hawley and Cunning (2017) and determination of risks and consequences of failure.
2) Slope stability analyses to determine the factor of safety against failure of slopes.
3) Determine the maximum attainable height considering the available information.
4) Determine the path forward for achieving the maximum height proposed by AMNS as the initial design.
5) Provide geotechnical recommendations for the construction of the amended WRSF configuration.

This report presents the methodology and results of the stability analysis. It also presents the proposed approach to achieve the desired geotechnical performance of the WRSF.

3.0 SITE CONDITIONS

General Site Description

The Touquoy Gold Mine is located approximately 60 km northeast of Halifax, Nova Scotia, Canada, in the Moose River Gold Mines District. For the current production phase of Touquoy Mine, it is planned to stockpile waste rock at the WRSF, ore at the Run of Mine Stockpile, and topsoil at the Mill Topsoil Stockpile as well as at the general Topsoil Stockpile (Figure 1). The current study looks at the WRSF, located at the northeast side of the mine site, to the north of the existing tailings storage facility, between coordinates 505,458 m E and 506,357 m E, and between coordinates 4,981,464 N and 4,982,133 m N.

The zone is currently surrounded by the existing access roads. A drainage ditch is excavated and lined in the periphery of the WRSF. There are wetlands on the southeastern and northwestern sides of the site. The current design has limited the boundary of the WRSF to avoid the wetlands. The tailings storage facility is located approximately 60 m to 130 m south of the WRSF.

3.1 Geology

Stantec (2016a), has geologically classified the bedrock as interlayered argillite and greywacke, and grey in colour. Argillite and greywacke are types of sedimentary bedrock.

3.2 Topography

Topography in the area of the WRSF generally consists of a low-lying, relatively flat area in the western section rising to a drumlin feature to the east (Stantec, 2016b). At this point, the initial topography of the area is altered by the ore and waste rock placed on the site. The natural topography is interpreted from the site LiDAR surface file provided by AMNS, and the latest geometry of the WRSF was taken from the LiDAR file, "1m LIDAR surface", provided by AMNS. The organic topsoil (0.1 to 0.3 m in thickness) has been stripped prior to dumping the waste/ore materials. However, at the southeastern side of the site, the waste rock has been dumped either on cut trees or directly on standing trees.

3.3 Groundwater Conditions and Porewater Pressure

According to the geotechnical investigation report (Stantec, 2016a), during the field investigations the groundwater was encountered in depths from approximately 1.5 m to 4.3 m below ground surface in the test pits. However, groundwater was not encountered/observed in any of the boreholes.

The groundwater table at 4.3 m depth was encountered in TP-15-22 which is in the drumlin (elevation 141.1 m). TP-15-25 is also in the drumlin, but the groundwater table is at 1.5 m depth at this location. The rest of the test pits and the monitoring wells around the WRSF are at elevations 122 m to 133 m and phreatic surface is at 1 m to 2 m depth.

3.4 Hydrology

The Environment Canada Greenwood station is located approximately 15 km north of the site. The average annual rainfall from 1981 to 2010 was 903 mm, and the average annual snowfall was 271 mm (https://weather.gc.ca/canada_e.html). Water is collected by a clay-lined drainage ditch that surrounds the WRSF and carried towards the collection ponds at the southern and southwestern side of the WRSF.

3.5 Hydrogeology

As per the geotechnical investigation data, a low plasticity clayey till layer of 2 to 12 m in thickness is overlain by a relatively thin layer of organic soil (0.1 m to 0.3 m) and silt and sand layer (0.2 m to 1.4 m), and is underlain by bedrock. The silt and sand layer is described as loose to compact, and can be considered as having a relatively high permeability.

The clayey till is described as firm to hard with water contents between 8.8% and 13.5%, plastic limits between 15.3% and 17.7% and percentage of fines as high as 73%. This material could have a relatively low hydraulic conductivity and is expected to be susceptible to porewater pressure build-up during construction under the weight of the waste rock.

3.6 Stratigraphy

A geotechnical site investigation was carried out by Stantec in 2015 (Stantec 2016b) in the WRSF area, which included five boreholes and five test pits. The borehole and test pit records are presented in Appendix E of this report.

The subsurface conditions encountered generally consisted of a surficial layer of topsoil and peat with a thickness of 0.1 to 0.3 m overlying a sand and/or silt layer (up to 1.4 m thick, not present in all locations) on top of the clayey till (2 to 12 m thick) and/or bedrock. Bedrock was encountered at depths of 2.3 m (southwest) to 10.6 m (southeast).

In the 2015 geotechnical site investigation, index properties and the number of blow counts in Standard Penetration Test (SPT) were measured. In the present report, in order to determine the stability analysis parameters, the blow count values were corrected for the hammer energy efficiency (N_{60}). Additionally, in order to eliminate the effect of presence of coarse grains in the soil matrix, only the N_{60} values associated with recoveries greater than 60% are considered. Then the 20th percentile of the N_{60} values were used to determine the shear strength parameters of soil using empirical correlations (see Appendix C). The choice of the 20th percentile of the N_{60} values for parameter definition is a relatively conservative assumption. This assumption was adopted because of the limited number of boreholes over the site, and the fact that the N_{60} values were used to determine undrained shear strength (Su) of the fine-grained soil, whereas, using SPT test results for the determination of Su is not considered to be common practice.

The clayey till is classified as low plasticity clay with plasticity indexes of 6.6% to 11.3%. The soil's plasticity (even low) suggests that this material could behave as a cohesive material. In the boreholes located in the southeastern (BH15-24), northern (BH15-21) and northwestern (BH15-20) portion of the site, the first 2 m to 3 m of the clayey till layer, compared to greater depths, has lower Standard Penetration Test blow counts (20th percentile of N_{60} between 8 and 12). For greater depths and in other locations, the 20th percentile of N_{60} values for the clayey till falls between 19 and 35, therefore the clayey till is rather stiff. These two clayey till layers are distinguished in the stability analyses carried out in undrained strength analysis (USA), as presented in Section 6.2. For the effective stress analysis (ESA), a single clayey till layer was defined and relatively conservative shear strength parameters were chosen.

4.0 WRSF'S CONDITIONS

4.1 Geometry

The target WRSF's outline was based on the profile provided by AMNS, which incorporates the maximum approved storage capacity. The final proposed configuration of the WRSF includes a global constant slope of 3H:1V, which will be covered by a topsoil layer of 300 mm thick. The waste rock will be placed in 10 m high lifts at the angle of repose with 16 m wide benches. Then, the benches will be profiled with the deposited waste rock to reach a constant slope. The cover details are not included in the stability model, as considered of minor impact on the geotechnical stability of the WRSF. However, the stability of the slope during construction before profiling to constant slope was evaluated, by considering a 10 m high slope with angle of repose and by applying a load representing a CAT 775 truck. As discussed in Section 5.7, the final original elevation (190 m) cannot be achieved and an alternative final elevation (and consequently pile height) that meets the design criteria was established.

4.2 Waste Rock Materials

The dumped waste rock is composed of argillite and greywacke rock with angular particles of 0 to 600 mm in diameter. The waste rock is typically free dumped on a flat surface overall the placement surface directly out from the haul truck (no additional placement) and the above lift is dumped and pushed with a dozer so that a new flat surface allows the haul truck circulation, thus building 2 lifts of approximately 2 m thick. This placement method is followed to make up a 10 m high bank. The material is described as well-graded. The material's deterioration is observed since the start of the construction of the WRSF. The deterioration process occurs due to freeze-thaw and dry-wet cycles and is expected for these types of rock. In order to establish the material properties of waste rock for the stability analysis, it was assumed that the waste particles will break down to smaller sand/gravel size particles in time.

4.3 Current Conditions

Presently, diverse materials are placed in the footprint of the pile presented in Figure 1. Waste rock is placed on the central and southeastern side of the footprint, whereas low-grade and medium-grade ore are placed on the western side. As of March 7, 2020, the maximum elevation was approximately at 154 m in the central part and at approximately 164.5 m in the southeastern part. Additionally, the maximum elevation for the low-grade and medium-grade ore placed on the western side of the WRSF footprint is approximately at 155 m. According to the data provided by AMNS, the maximum rate of loading for the waste rock materials has been in the order of 25 m per year. Presently, no sign of instability has been observed in the existing piles.

5.0 DESIGN CRITERIA

5.1 The Documents and Guidelines of Reference

The mine waste dump and stockpile design guidelines presented by Hawley and Cunning (2017) is used for risk assessment and classification of the WRSF. Table 1 presents the guidelines used for the engineering design.

Table 1: Reference Documents and Guidelines

Guideline	Comments
Editors: Mark Hawley and John Cunning, “Guidelines for Mine Waste Dump and Stockpile Design,” 2017	Practical guide to the investigation, design, operation, and monitoring of mine waste dumps, dragline spoils, and major stockpiles associated with large open pit mines. Developed by industry experts with extensive knowledge and experience, it summarizes the current state of practice and provides guidance.
Canadian Dam Association (CDA 2014)	Guideline used for assessment of risks and consequences.
The National Building Code of Canada 2015 (NBC)	Code used for the determination of the peak ground acceleration for pseudo-static analyses.

5.2 Targeted Outline and Quantity of Waste Rock Storage

The current configuration of the WRSF (3H:1V slopes) is proposed by the mine in accordance with the local legislative requirements. Using the before mining ground surface LiDAR file, a volume of 8 million m^3 approximately can be attained by the original proposed profile.

5.3 Infrastructure near the WRSF

The tailings storage facility is located within a 60 m distance approximately, to the south of the projected WRSF. Additionally, the access roads and a drainage ditch surround the projected WRSF.

A preliminary runout distance analysis considering a maximum original height of approximately 65 m (crest and toe elevation difference) and the distance of 60 m from the closest tow of the WRSF to the Tailings Storage Facility

(TMF) showed that, for the potential ruptures in the pile, the probability of the debris reaching the TMS is very low. Therefore, the peripheral roads and ditches are likely to be impacted but not the TMF.

5.4 Chemical Stability of the Material

As per AMNS, there is an insignificant volume of rock that demonstrates a potential acid rock drainage. According to the 2018 review of operational metal leaching/acid rock drainage monitoring data, a small percentage of all collected waste rock samples were potentially acid generating (Lorax, 2019). According to the information provided by AMNS, prior to commencing mining, 90 samples have been tested for potential acid rock drainage with 85 of them being acid consuming samples or neutral samples. The AMNS's current procedure is that once acid-generating rock has been identified, it will be kept together and encapsulated within neutral or acid-consuming rock.

5.5 Risks and Consequences Assessment for the WRSF

The risks and consequences for the WRSF were assessed according to the, "Guidelines for Mine Waste Dump and Stockpile Design," Hawley and Cunning (2017) guides. The WSRHC system is used as a guide to assess the level of effort required to investigate, design, and construct waste dump. The classification also helps determine the earthquake return period and targeted factors of safety formulated specifically for the Touquoy operation WRSF. This system requires evaluating 22 key factors that are thought to affect stability of the waste dump. Ratings are assigned to each factor. The sum of these ratings defines the Waste Dump and Stockpile Rating (WSR). The higher rating indicates a more stable configuration. The waste dump hazard class (WHC) and the qualitative instability hazard are defined according to the WSR (Hawley and Cunning 2017). Given that the geometry of the WRSF and the foundation conditions vary in different locations, two different cross-sections were considered for the characterization of the pile. For each cross-section, two slopes were considered (see Appendix A).

The WSRHC considers acceptance criteria based on "consequence" and "confidence" levels. "Consequence" concerns the impact of potential instability and the service life of the structure. "Confidence" relates to the reliability in the key input parameters and analytical technique used. Based on the Guidelines for Mine Waste Dump and Stockpile Design (Hawley and Cunning 2017), the level of consequence of the waste rock dump failure on the site is regarded as low, and the level of confidence on material properties and failure mechanisms is considered low.

The consequence of failure was rated as low because of the following:

- The slope less than 25^{o} and height of less than 100 m;
- Annual precipitation between 1,000 mm and 2,000 mm;
- Proximity to the tailing's storage facility, but not within the runout distance;
- Limited potential for environmental impact, but manageable.

The level of confidence on material properties and failure mechanisms is considered low because:

- The uncertainties related to the waste pile;
- Foundations were characterized in limited areas and only standard penetration tests and index properties are available, which are not considered adequate for determination of the shear strength of fine-grained soils;
- The foundation is only partially prepared prior to dumping waste rock (organic matter remains in some areas).

Note that the waste rock appears to degrade overtime and limited information is available on its mechanical behaviour for a long period of time. As this aspect is considered in the choice of the shear strength properties, it does not reduce the level of confidence on the material properties.

Table 2 presents the minimum required factors of safety (FS) for each consequence of failure and parameter confidence. For these categories, the stability acceptance criteria suggest a minimum factor of safety of 1.3 to 1.4 in static analysis and 1.05 to 1.1 in pseudo-static and post-liquefaction analysis.

Table 2. Waste Dump Design Acceptance Criteria (adapted from Hawley M. & Cunning J., 2017)

Consequences of Failure[1]	Parameter Confidence[1]	Acceptance Criteria	
		Minimum FS	Minimum Dynamic FS
Low	Low	1.3 – 1.4	1.05 – 1.1
	Moderate	1.2 – 1.3	1.0 – 1.05
	High	1.1 – 1.2	1.0
Moderate	Low	1.4 – 1.5	1.1 – 1.15
	Moderate	1.3 – 1.4	1.05 – 1.1
	High	1.2 – 1.3	1.0 – 1.05
High	Low	>= 1.5	1.15
	Moderate	1.4 – 1.5	1.1 – 1.15
	High	1.3 – 1.4	1.05 – 1.1

1) Semi-quantitatively evaluated.

According to the WSRHC system, with the currently available data, the dump rating obtained for the WRSF is between 52.5 and 56.5. This rating is subject to change if additional filed and laboratory data become available. This dump stability rating corresponds to Class II (Low hazard) to Class III (Moderate hazard) waste dump and stockpile hazard. A summary of the results of the WSRHC system, and a detailed description of considered factors for the four cross-sections are presented in Appendix A.

For a moderate hazard classification, Hawley and Cunning (2017) suggest the following level of effort:

- **Investigation and characterization**: Comprehensive desktop studies to establish initial stability rating and hazard classification; detailed site reconnaissance to confirm assumptions from desktop studies; detailed mapping and subsurface investigation likely including test pitting-trenching and limited drilling and sampling; in situ instrumentation and testing and laboratory testing to verify foundation and fill material properties; initiate comprehensive baseline environmental monitoring; condemnation drilling.
- **Analysis and design**: Comprehensive stability analyses, including consideration of runout potential; qualitative risk assessment; design moderately constrained by stability and potential impacts; design optimization and impact mitigation studies; design conducted by experienced geotechnical specialist with peer review.
- **Construction and operation**: Moderate site preparation, may include diversions and underdrainage; limited foundation instrumentation to verify performance; runout/rollout mitigation measures, if required; moderately constrained construction sequence; control of fill quality and placement as necessary; loading/advance rate

restrictions; standard instrumentation and visual monitoring with well-defined TARPs; periodic (minimum annual) inspections by experienced geotechnical specialist.

5.6 Water Management

There are water ponds at the southern and southwestern sides of the WRSF. It is assumed that the WRSF is composed of mining waste rock materials sufficiently coarse and permeable to allow the flow of runoff water to the drainage ditches and finally collected in the ponds.

5.7 Stability Analyses

AMNS has provided Golder with the proposed WRSF layout based on the closure plan that accounts for the maximum approval limit. This report presents the results of the stability assessment for the layout proposed by AMNS, and provides a maximum achievable height considering the proposed slopes, the pile's location relative to the site topography and the existing data on both waste rock and foundation soil.

5.7.1 Cross-Sections

Three cross-sections considered to be the most critical were modelled in order to reflect different fill heights, and various stratigraphy of the site. A plan view of the site showing the location of the studied cross-sections is presented in Figure 2. A global slope of 3H:1V was considered as the final closure profile. Also, an intermediate section with 10 m high banks with slopes equal to the angle of repose and 16 m wide benches was considered. In order to determine the maximum attainable height of the WRSF, stability analyses were carried out for different final elevations (160 m, 170 m, 180 m, 190 m). As presented in Section 6.2 below, the stability analyses for the cross-section 2-2' for a final elevation of 170 m were carried out with a 15 m wide bench at an elevation of 150 m in order to meet the stability criteria.

It was assumed that the topsoil had been removed prior to depositing the waste material within the WRSF, and the waste rock was deposited directly on the clayey till layer. For cross-sections 2-2' and 3-3', a 3 m thick layer of softer material (surface clayey till) was considered below the ground surface. Therefore, the stratigraphy in the slope stability analyses cross-sections includes the following:

- An embankment of waste rock;
- A layer of surface clayey till below the ground surface (for cross-sections 2-2' and 3-3');
- A layer of clayey till;
- The weathered bedrock.

5.7.2 Loading

Seismic Conditions

The earthquake return period is determined from the WRSF's risk and consequences assessment and the WRSF classification (Hawley and Cunning 2017). The return period of the earthquake is defined as 1:475 years. Using the disintegration of seismic hazard data provided by the earthquakescanada website, based on the National Building Code of Canada 2015 (NBC) for soil class C at the location of the site, the Peak Ground Acceleration (PGA) equals 0.023 (see Appendix B). Fifty percent of the PGA for a soil class C was used as the horizontal pseudostatic acceleration (K_h = 0.0115).

External Loads

No long-term external load is planned to be applied to the WRSF. However, a truck load representing CAT 775 was applied 5 m from the crest of the slope of the WRSF. The design of the WRSF should not be controlled by this load case, given the temporary nature of the load and the conservative assumptions regarding the calculation of a line load in a 2D limit equilibrium model. The factor of safety against slope failure was evaluated for this case only for future references.

5.7.3 Phreatic Surface Level

Based on the soil investigation report (Stantec, 2016a) and the data provided by AMNS on the monitoring wells in the zones adjacent to the WRSF, the groundwater table was assumed to be 1.5 m below the natural ground surface. It was assumed that the particle size distribution of the waste rock will allow free drainage of water, and the groundwater table will remain at its present level. This assumption can be further evaluated by installing vibrating wire piezometers in the WRSF (see Section 0).

5.7.4 Factors of Safety

Targeted minimum safety factors for stability of the structure was determined based on Hawley and Cunning (2017) recommendations for hazard and stability assessment of waste rock piles and stacks (Appendix A), as well as consequence of failure and parameter confidence. Stability analyses under static, pseudo-static and post-liquefaction conditions will be performed with SLOPE / W software (GeoStudio software suite). The targeted factors of safety are presented in Table 3 below.

Table 3: Targeted Factors of Safety

Condition	Factor of Safety
Static	1.3
Pseudo-static	1.1
Post-liquefaction (seismic and static)	1.1

5.7.5 Types of Limit Equilibrium Stability According to the Drainage Conditions at Failure

Two types of stability analysis were carried out.

Effective Stress Analysis (ESA)

This analysis represents the case where the excess pore pressures caused by the placement of waste rock dissipates due to a relatively slow rate of construction or sufficient time after construction, and the shear-induced pore pressures are also zero due to a slow rate of shearing and dilative behaviour of soil during failure.

Undrained Strength Analysis (USA)

This case applies to staged construction where loading rates could produce positive excess pore pressures, where the soil behaviour is contractive, and the shearing is quick enough that shear-induced porewater pressure can be generated.

Three different modes of failure were considered for the USA analysis:

- Circular slip surfaces;
- Non-circular slip surfaces;
- Block specified slip surfaces.

The circular failure mode represents deep slip surfaces, whereas, non-circular and block specified modes consider slip surfaces that pass through the surface clayey till which has lower shear strength compared to lower elevations. In the results section, only the factors of safety for the most critical slip surfaces are presented.

5.7.6 Porewater Pressure Generation During Construction

Given that the clayey till might have a relatively low hydraulic conductivity and a thickness of up to 12 m, and that the foundation's consolidation rate relative to the rate of loading is unknown, the excess porewater pressure generated during construction was taken into consideration in the analysis. The porewater pressure buildup due to construction was incorporated in the analysis by the use of the pore pressure coefficient $\bar{B}$:

Equation 1 $\bar{B} = \Delta u / \Delta \sigma_1$

where, Δu is the change in the porewater pressure due to a change in the principal stress, σ_1. As in most of the cases, the principal stress is close to vertical, σ_1 can be estimated as the vertical stress.

In this report, analyses in USA are performed with $\bar{B} = 0$, i.e., no porewater pressure will be built up due to construction, and with $\bar{B} = 1$. A $\bar{B} = 1$ implies that any addition of overburden load will be transferred to the porewater pressure in the foundation. As an example, a $\bar{B} = 1$ for the final configuration means that the entire height of the pile is placed quickly in one lift to the foundation. It is to be noted that this assumption is very conservative and will be discussed in geotechnical recommendations provided in Section 0.

5.7.7 Material Properties

Table 4 below presents the material properties used and defined in the stability analysis. The material properties were chosen based on the Standard Penetration Test (SPT) blow counts values, as well as soil descriptions and index properties as reported in Stantec, 2016. A synthesis of the data on the soil layers is presented in Section 3.6.

For the USA analyses, two separate clayey till layers were determined to distinguish between the surface and the deeper clayey till. For both layers the minimum undrained shear strength, Su (for overconsolidated clay) was determined based on empirical correlations presented in Appendix C using the 20th percentile of N60 values. It is assumed that the soil shear strength increases as the overburden load increases and consolidation takes place (for normally consolidated clay). The following equation (Mesri, 1975) was used to estimate the shear strength of normally consolidated clay.

Equation 2 $S_u = 0.22\, \acute{\sigma}_v$

In order to consider the potential loss of shear strength due to the generation of porewater pressure (static or cyclic liquefaction), the USA analyses were also carried out applying a 20% reduction is the undrained shear strength. The associated material properties are presented in the column USA post-liquefaction of Table 4.

For the effective stress analysis (ESA), a single clayey till layer was defined, and the associated friction angle was chosen based on the soil descriptions and index properties, as well as the data in the literature on low plasticity clays.

Table 4: Material Geotechnical Properties

Material	Unit Weight, γ (kN/m³)	ESA		USA Peak	USA Post-Liquefaction (seismic/static)	Source
		Cohesion, c' (kPa)	Angle of Internal Friction, ϕ' (°)	s_u or $\frac{s_{u,peak}}{\sigma'_v}$ (kPa)	s_r or $\frac{s_r}{\sigma'_v}$ (kPa)	
Waste rock	21	0	35 Sensitivity analysis: 30 to 42	n/a	n/a	Leps (1970) and client information
Clayey till foundation (Cross Section 1-1')	21	0	27	USA_{NC}: $\frac{s_{u,peak}}{\sigma'_v}$ = 0.22 $USA_{NC\text{-}min}$: 120	USA_{NC}: $\frac{s_{u,peak}}{\sigma'_v}$ = 0.18 $USA_{NC\text{-}min}$: 96	SPT data analysis
Surface clayey till[1] foundation (Cross Sections 2-2' and 3-3')[1]	21	0	27	USA_{NC}: $\frac{s_{u,peak}}{\sigma'_v}$ = 0.22 $USA_{NC\text{-}min}$: 50	USA_{NC}: $\frac{s_{u,peak}}{\sigma'_v}$ = 0.18 $USA_{NC\text{-}min}$: 40	SPT data analysis
Clayey till foundation (Cross Sections 2-2' and 3-3')	21	0	27	USA_{NC}: $\frac{s_{u,peak}}{\sigma'_v}$ = 0.22 $USA_{NC\text{-}min}$: 170	USA_{NC}: $\frac{s_{u,peak}}{\sigma'_v}$ = 0.18 $USA_{NC\text{-}min}$: 136	SPT data analysis
Bedrock	Impenetrable					

Note:

[1] Surface clayey till extends from the ground surface to a depth of 3 m.

6.0 STABILITY ANALYSES RESULTS

As explained in Section 5.7, given the nature of the clayey till, presence of sand, gravel and cobbles, as well as high fines content and the groundwater table which is close to the surface, two series of analyses were carried out: Effective Stress Analyses (ESA) and Undrained Strength Analyses (USA). Undrained strength analyses were carried out for different pile heights in order to determine the maximum achievable height considering available data on the foundation soil.

6.1 Effective Stress Analyses (ESA)

The studied cross-sections as named in Section 5.7.1 are presented in Figure 2 at the end of the text. Select potential slip surfaces are presented in Appendix D. A summary of the obtained factors of safety (FS) is presented in Table 5 below. The FS values are presented for three potential failure surfaces: Type 1 being the local failure during construction by the load of the truck, Type 2 represents the potential failure surface that initiates in the slope and passes through the foundation, and Type 3 presents the global failure surfaces that include at least 5 m of the pile's crest.

For the effective stress analyses (ESA), FS values are above the targeted value (1.3). For the case of intermediate stages analyzed for Cross Sections 1-1' and 2-2' with application of the truck load, the FS is below 1.3. However, since the method of application of truck load in the model is conservative, and given that this load case is temporary, this FS is considered to be acceptable. Yet, the construction must be supervised by an engineer and in case of observation of foundation soil or embankment movements, the construction must be seized, and the consultant must be advised (see Section 0 for details).

Given the potential of degradation of the waste rock, a series of sensitivity analyses was carried out considering a range of internal friction angles for the waste rock. The FS values obtained by the sensitivity analyses remained above the required value, and the sensitivity of the results to the friction angle of the waste rock in the range considered in the analyses was found to be minimal.

Table 5. Factors of Safety Obtained in the ESA Slope Stability Analyses for the Original Profile (max. elevation = 190 m)

WSRF Configuration	Analysis Method	Static FS (min: 1.3)		
		Type 1	Type 2	Type 3
Cross section 1-1'				
Construction of ultimate lift (190 m)	ESA	1.22	-	1.81
Final configuration (190 m)	ESA	-	1.82	1.83
Final configuration (190 m) - Lower bound resistance of waste rock	ESA	-	-	1.72
Cross section 2-2'				
Construction of 1st lift (140 m)	ESA	-	-	1.11
Construction of ultimate lift (190 m)	ESA	1.22	-	1.61

WSRF Configuration	Analysis Method	Static FS (min: 1.3)		
		Type 1	Type 2	Type 3
Final configuration (190 m)	ESA	-	1.58	1.62
Final configuration (190 m) - Lower bound resistance of waste rock	ESA	-	-	1.54
Cross section 3-3'				
Final configuration (190 m)	ESA	-	1.92	1.94
Final configuration (190 m) - Lower bound resistance of waste rock	ESA	-	-	1.77

6.2 Undrained Strength Analyses (USA)

The USA analyses were initially carried out in static conditions for the original final elevation (190 m). Since the original final elevation of the pile did not meet the stability criteria, the USA analyses were carried out for different final pile elevations to determine a maximum achievable elevation. The pile's maximum elevation was determined as 170 m. For this final elevation, as explained in Section 5.7.5, the analyses were carried out considering different failure modes (circular, non-circular and bloc-specified), as well as pseudo-static and post-liquefaction conditions. Additionally, as explained in Section 5.7.6, the analyses were carried out with and without considering porewater pressure buildup during construction.

USA Analyses without Consideration of Porewater Pressure Buildup during Construction

Table 6 presents the results of the stability analyses for the three studied cross-sections for a maximum elevation of 170 m. For the cross-section 2-2' (southeastern side of the WRSF), the factor of safety (FS) in static conditions for non-circular slip surfaces is below 1.3 (1.22). In order to meet the stability criteria, a setback bench of 15 m at an elevation of 150 m is required. The static, pseudo-static and post-liquefaction analyses are carried out and presented by considering this amendment in the pile's geometry. By adopting this new geometry for cross-section 2-2', the safety factors are greater than 1.3 for static conditions and 1.1 for pseudo-static and post-liquefaction conditions. Because of the presence of the surface clayey till in cross-sections 2-2' and 3-3', the FS values are presented for both circular (deep) and non-circular slip surfaces. The non-circular slip surfaces are more critical and control the design.

The use of the 15 m wide setback bench at 150 m elevation is further detailed in Section 0.

USA Analyses with Consideration of Porewater Pressure Buildup during Construction

The stability analyses in static condition were also carried out by considering a $\bar{B}$ = 1, i.e., the application of any load on the ground generates the same amount of porewater pressure (PWP) buildup. The FS values obtained by considering a $\bar{B}$ = 1 both for the non-circular and circular deep slip surfaces are presented in Table 6.

For the shallow non-circular slip surfaces, the FS values are below the unity. However, given that these slip surfaces pass close to the groundwater table, and given that the surface clayey till is loose, it is assumed to have relatively high permeability, and it can be expected that the porewater pressure generated due to the fill placement dissipates rapidly close to the failure surface. Therefore, the assumption of $\bar{B}$ = 1 remains very conservative and the FS<1 does not mean that the slope is unstable.

For the deep circular slip surfaces, for cross-sections 2-2' and 3-3', the FS is above 1.3 by considering the porewater pressure buildup. However, for cross-section 1-1', FS is below 1.3 (1.23). Here again, the assumption of $\bar{B}$=1 can be conservative. Yet, the results suggest that the dissipation of the generated PWP and mobilization of the soil shear strength must be ensured. The required time between placement of the lifts can be explicitly determined if the hydraulic and mechanical properties of the foundation soil are available. However, considering the lack of detailed field and laboratory data on the foundation soil, and the fact that the maximum elevation is not yet attained to observe the behaviour of foundation soil, as well as the low consequence of failure, it is considered reasonable to adopt an observational method as the pile construction proceeds. This method includes management of the pile placement sequences and zones by closely observing the pile and foundation behaviour as the waste rock lifts are placed in order to prevent failures triggered by porewater pressure buildup (see the Section 0, geotechnical recommendations for details on the observational method).

Table 6: Factors of Safety against Sliding for Maximum Attainable Elevation, 170 m (USA analysis)

Cross-Section	Condition	Required Factor of Safety	Continuous 3H : 1V Slope	Continuous 3H : 1V Slope; $\bar{B}$ =1	15 m Bench at 150 m Elevation	15 m Bench at 150 m Elevation; $\bar{B}$ =1
1-1'	Static	1.3	1.35	1.23	-	-
	Pseudo-static	1.1	1.29	-	-	-
	Post-liquefaction (seismic/static)	1.1	1.11	-	-	-
2-2'	Static - Circular	1.3	-	-	1.45	1.43
	Static – Non-circular	1.3	1.22	-	1.30	0.98
	Pseudo-static	1.1	-	-	1.24	-
	Post- liquefaction (seismic/static)	1.1	-	-	1.12	-
3-3'	Static - Circular	1.3	1.53	1.48	-	-
	Static - non-circular	1.3	1.31	0.81	-	-
	Pseudo-static	1.1	1.25	-	-	-
	Post- liquefaction (seismic/static)	1.1	1.14	-	-	-

7.0 GEOTECHNICAL RECOMMENDATIONS

As per the stability assessment presented above based on actual available data, the waste rock storage facility (WRSF) can be constructed up to an elevation of 170 m. The stability criteria did not meet the original 190 m elevation deemed required to stock all waste rock material per the life-of-mine plan effective at the time of this report. The available field and laboratory data used for the stability assessment were not enough to support the establishment of less conservative geotechnical soil parameters.

An observational approach is proposed below to deal safely with potential instability caused by excess pore water pressure in the foundation or other mechanisms. This approach demands thorough monitoring and surveillance and accumulation of data about the behaviour of the pile along the construction period. Based on good monitoring and surveillance data, if the pile is built to 170 m elevation without development of significant settlement or sign of instability, it is considered that the WRSF may be built safely to a higher elevation without completing additional investigation. Golder would be pleased to support AMNS in this opportunity.

If the long-term approach using the observational method for potentially raising the pile to the original elevation does not meet the actual needs of AMNS, alternatively, it is considered that additional investigation and testing would probably lead to a change in design and higher elevations could be reached to meet the original capacity requirement. Recommendation for an investigation program is presented in Section 8.0.

The following geotechnical recommendations are provided to ensure the WRSF performance through its construction (operation) and closure phases up to the current design final elevation of 170 m.

7.1 The Proposed WRSF Configuration

The target WRSF's outline is based on the original profile provided by AMNS but was amended to meet the design criteria presented in Section 5.0.

It is recommended that the WRSF be built in lifts of 2 m thick up to a maximum bench height of 10 m with a temporary slope at the angle of repose of the waste rock material. The additional bench will set back at 15 m from the crest of the underneath bench. The setback is primarily established as a catchment area bench for the placement of the bench above but is also required to allow the circulation of construction equipment for further construction steps. For employee safety and traffic purposes, all intermediate temporary slopes should be protected by a safety berm made of waste rock material having a height equivalent to 2/3 of the maximum wheel diameter accessing the WRSF, typically from the largest haul truck on site.

For closure purposes, all intermediate temporary slopes will be profiled to a global constant slope of 3 horizontal: 1 vertical (3H:1V), which will be covered by a topsoil layer of 300 mm thick. A continuous 3H : 1V slope meets the geotechnical stability requirements for the pile with a maximum elevation of 170 m, except for the southeastern zone, where a setback bench of 15 m at an elevation of 150 m is required to meet the required factor of safety against sliding. Figure 3 at the end of the text identifies this zone. AMNS is to manage the pile configuration to meet the requirements.

A haul road access ramp needs to be planned as part of the design. This was not part of the intent of the current mandate. As a general guideline, the slope angle is to be at a maximum of 10% and the width for a one-way road should be 2 times the width of the largest equipment and 3 times for a two-way road. For bends, the widths should be 2.5 times and 3.5 times larger than the largest equipment for one- and two-way roads, respectively.

7.2 Foundation Preparation

The surface topsoil and peat should be removed prior to placement of the waste rock. This is a typical recommendation to ensure that the required geotechnical stability at the external footprint of the waste rock material around the pile. Extending the foundation preparation further within the center of the facility is also a good practice that serve well the closure purpose where typically the topsoil can be reuse on the final surface of the WRSF. No minimal foundation preparation width at the periphery of the pile is has been established determined. Additionally, it is recommended to excavate all topsoil for reuse as part of the cover system for growth of vegetation.

7.3 Waste Rock Material Placement

It is important that the WRSF be developed as described above to ensure its stability. Only blasted rock material is planned to be stored in the WRSF, waste rock and temporary ore material. Should the need to place other types of materials arise (i.e. granular, cohesive or organic soil), the WRSF design would need to be reviewed by the designer to determine if it is possible and how is it safe to do it.

A development plan should be developed by the mine for short- and long-term perspectives. This plan should be updated periodically to ensure that the design criteria are met. This plan is particularly important for planning the location and encapsulation of the potential acid-generating waste rock portion. It should be shared with all WRSF stakeholders to ensure smooth coordination. Good communication should take place between the personnel responsible for establishing the dump development plan and the personnel responsible for the operation of the structure on the ground. Poor communication can cause the trucks to unload in the wrong sectors, resulting in non-compliance with the geometries (no catchment bench, too high benches, too steep slopes, etc.) or the transportation of materials to the wrong dump sector (mixture of organic and inorganic overburden, too rapid development, etc.). The trucks can unload on the bench in a designated place and a bulldozer can place the material, which reduces the risk of miscommunication, since only one operator takes care of placing the material in all the trucks. It is also recommended to restrict access to sectors that have been completed or that may cause confusion with barricades or cones, and to inform operators of the sign meanings. The development of the WRSF should be monitored by the surveyors to ensure compliance with the designed geometry.

The waste rock should be placed in lifts of up to 2 m in height by unloading the haul truck on the top lift and placed by the bulldozer to ensure the maximum strength for the pile. Free dumping should be avoided. The placement of 2 m thick intermediate lifts is considered good practice for the conditions of the WRSF rather than using a complete 10 m thick lift at once. First, this practice decreases the thickness of the rubble accumulation zones that form at the base of each lift. A rubble accumulation zone is characterized by the accumulation of the coarser particles of the material placed at the bottom of the lift. The concentration of coarser particles involves a higher void ratio that influences the air convection movement through the waste rock pile. As the WRSF will contain pockets of potentially acid-generation material, it is required to limit air convection in the pile as much as possible. Secondly, a thin lift allows better control of foundation pore water generation build up. The soil nature described in Section 3.5 is considered susceptible to pore water build up, and thus the porewater pressure generation must be controlled. Finally, placing thinner placement lifts allows for good compaction of each intermediate lift, thus increasing the strength of the material.

No formal compaction method is specified, because it is considered that the circulation of haul truck and bulldozer would be adequate for the compaction of the waste rock. Circulation of haul trucks should be managed in a way to increase the compaction of the overall lift surface, rather than driving on the same access road routinely.

The development of the WRSF must be carried out while minimizing, as much as possible, the generation of porewater pressures in the foundation soils causing instability. This can be achieved by increasing the active deposition length at the crest or by placing materials simultaneously at various locations around the dump. Consecutive rises in the same area in a short period of time should be avoided. In case of uncertainty, the installation of piezometers can confirm that the development speed in an area is adequate. Additional details about management of pore water pressure are presented in Section 7.7.

The surface of the lift should be tilted outwards to minimize the water infiltration inside the WRSF. Accumulation of water inside and at the bottom of the dump is undesirable and can affect the stability of the structure. To this end, breaching through the temporary safety berm at a periodic interval may be required in order to deal appropriately with runoff run-off- water.

7.4 Potential Acid-Generating Waste Rock Management

As discussed in Section 5.4, as per AMNS, a small portion of the waste rock demonstrates a potential acid rock drainage (Lorax, 2019). The current procedure adopted by AMNS is that once acid-generating rock has been identified, it will be kept together and encapsulated within neutral/acid consuming rock.

For closure purposes and for reporting to the regulator, it is recommended to carefully survey the location (Northing, Easting, and elevation) of those pockets during the operation to demonstrate that the design criteria regarding this aspect is met.

Characterization, management and developing a planning strategy to deal with the acid rock of the acid-generating rocks has not been a part of Golder's study. It is the responsibility of AMNS to develop and ensure proper management and register the location of each pocket.

7.5 Surveillance Program

The operation of any WRSF involves instability risks. To manage these risks, an observational approach is proposed. This approach consists in regularly monitoring the condition of the structure by establishing a monitoring program. The operation of the structure is subsequently adjusted during its development according to the data collected.

The monitoring program includes an inspection and instrumentation program (if needed later) to ensure the integrity of the WRSF. The main objective of this program is to identify, assess, correct and document any conditions representing a deviation from the normal operating conditions of the structure. Regularly reviewing the information obtained during the program can help identify problematic situations and adjust operations accordingly to reduce risks. It is important that the monitoring program be carried out on a routine basis according to a defined schedule.

Anyone working or travelling regularly in the waste rock dump area should be able to contribute to the monitoring program by being familiar with the normal operating conditions of the structure and able to identify deviating conditions of a normal operation. These people should be aware of the procedures and able to document these aspects or able to transmit these observations to the responsible people.

The monitoring program must be adapted to the changing conditions of the structure and well documented.

7.5.1 Inspections

A series of regular inspections is necessary to ensure that the WRSF behaves as expected, to identify problematic areas requiring rectification and to complete these actions in time. The inspections include a visual evaluation of the structure with photographs, the analysis of the instrumentation and the preparation of written and photographic documentation. The chief engineering officer is responsible for coordinating formal inspections, for receiving and reviewing the documentation produced during these inspections and for carrying out the necessary follow-up.

The various types of inspections are:

- Daily patrols - carried out by personnel working routinely on the structure;
- Formal monthly or fortnightly inspections - carried out by the engineering department;
- Special inspections - carried out by the engineering department and by external geotechnical consultants as required.

7.5.1.1 Daily Patrol

Daily patrols must be carried out by AMNS personnel from the various departments (environmental, mining) with basic knowledge of the various components of the WRSF during their passage in this sector. These routine inspections must include active development areas to ensure that each component of the structure is functioning properly and that there are no abnormal conditions. It is not necessary to complete an inspection report following these patrols unless abnormal conditions are observed.

7.5.1.2 Formal Inspection

Formal monthly or bimonthly inspections must be carried out by the personnel of the surface mine operation having a very good knowledge of the WRSF and its various components. The frequency of these formal inspections should be adjusted according to site conditions and the results of previous inspections. All the components of the structure must be monitored during this inspection and photographs must be taken.

The following points should be noted during these inspections:

- Description of structure development activities;
- Inspection of slopes, crest and banks of the WRSF for signs of instability (tension cracks, localized slope failure, erosion, foundation bulging);
- Any observation of water flow or significant accumulation in the structure (seepage of water on slopes, flow at the base, generation of water inflows or outflows);
- Inspection of peripheral ditches around the WRSF, depth of water in the ditches, signs of erosion, high presence of sediment;
- Condition of access roads in the footprint of the WRSF;
- Reading of instrumentation data for the period covering the inspection.

7.5.1.3 Special Inspection

Special inspections may be required in addition to formal inspections following periods of intense rain or rapid snow melt, abnormal instrumentation readings, a seismic event, or the development of signs of movement and rupture. The special inspection must be carried out immediately after the event using the formal inspection form. The period of special inspections must be determined based on the observations made or the irregularities observed.

7.6 Instrumentation

Considering the groundwater table which is close to the ground surface, and the high proportion of fines in the soil matrix, excess porewater pressure can potentially be generated in the foundation due to the waste rock placement.

As presented in Section 6.2, the stability analyses results indicate that excess porewater pressure in the foundation can reduce the factor of safety against sliding to below the required factor of safety. Recommendations are presented above to best manage the conditions.

Installation of vibrating wire piezometers (VWP) in the foundation to obtain information on the generation and dissipation of porewater pressure as waste rock is placed would well support the decision making before appearance of signs of instability. On the other hand, it is common for waste rock dumps to accept occurrence of movements and modify the deposition plan to let the movements seize before further placing materials over the area of concern. Therefore, the construction of the WRSF can continue without installation of VWPs, as long as a good observational approach method is established, including future planning for installation of additional instrumentation to best support any situation that may arise.

It is also recommended to install survey monuments in order to monitor the pile's performance. The displacement measurement during construction can provide information on both waste rock mechanical behaviour, and the foundation soil's hydraulic and mechanical (consolidation) behaviour. This can provide guidance for management of waste rock placement sequences and locations. The use of basic survey monuments (movable) is highly recommended, particularly if no VWP is installed. Survey monuments will provide data on the magnitude and rate of settlement, that could later be used to control the rate of waste rock placement in the area of concern, where signs of instability are observed.

Wire extensometers are instruments installed as required when tension cracks with an opening greater than 1 cm or continuous over a length of more than 5 m are observed in operating areas. An extensometer is installed perpendicularly above the tension crack. At defined time intervals, the crack opening is measured using the cable as a reference point. This instrument makes it possible to measure the opening of the crack and to calculate the speeds of displacement by comparing with the previous reading. Each reading must be documented in an Excel file indicating the date and time of the reading, the location of the reading and the measured opening. It is recommended that the readings be done at least 4 times on the first day (minimum interval of 1 hour between readings) and then additional readings taken at the intervals recommended in Table 7, until the movement stabilizes or operations are moved to another area.

The wire extensometer can be replaced by steel rods installed on either side of the crack. The opening of the tension crack is then measured manually with a tape.

Table 7. Guidelines for the Crack Control

Wire Extensometers Displacement Rate	Interval between Manual Readings	Procedure to Follow
8 mm/h or less	4 h	Normal settlement, no procedure to follow
8-16 mm/h	2 h	Short unloading and pushing by bulldozer
Greater than 16 mm/h	1 h	Closure of cracked sections until the displacement rate reduces to below 8 mm/h

7.7 Management of Potential Foundation Pore Water Generation Buildup

Considering the groundwater table which is close to the ground surface, and the high proportion of fines cohesive particles in the soil matrix, excess porewater pressure can potentially be generated in the foundation due to the waste rock placement. As presented in Section 6.2, the stability analyses results indicate that excess porewater pressure in the foundation can reduce the factor of safety from sliding to below the required factor of safety. As no piezometer is installed in the pile, and limited information is available on the foundation soil characteristics, the porewater pressure response to the fill placement cannot be monitored or predicted. However, the risk of instability occurrence can be mitigated effectively.

In order to mitigate the risk of instability triggered by porewater pressure generation or other mechanisms, an observational method is proposed. This method assumes that AMNS has competent staff and required apparels to monitor the pile and foundation's behaviour throughout the operation period, and that staff is able to react accordingly should signs of instability be noticed. The monitoring requires not only observation of the pile and foundation, but also preparedness to react based on a well-defined and well-known (by the workers) action plan. The observational method is based on a Plan-Do-Assess concept:

- **Plan:** the observational method plan. The plan is what presented in all of the above subsections of Section 0.
- **Do:** Perform appropriate monitoring and surveillance while accumulating data, react according to the plan if needed.
- **Assess:** Review the plan cyclically to make sure it meets the original objective. As an example, the use of a 2 m thick lift may need to be reduced to smaller lift thickness to limit the weight added at one time. As another example, at the moment, no delay between placing 2 m thick lifts is prescribed, as it is considered unnecessary. However, a specific delay between placing lifts may need to be put in place, as a function of the pile's behaviour during the placement of first lifts.

Should excess porewater pressure in the foundation need to dissipate to an acceptable level, materials would be deemed unsafe to be placed at a given location. As an essential part of the plan presented above, AMNS should develop an alternative temporary dump area in this eventuality, to let porewater pressure comes back to normal. This is not an issue for the short-term, but at higher elevations of the pile, there will be less available active deposition areas available. Therefore, any delay before placement of next lift could become a concern. Proper monitoring and surveillance data will therefore be required to establish such a delay.

7.8 WRSF Closure

The closure concept consists of profiling the external slope to a constant 3H:1V slope and placing a 0.3 m thick lift of topsoil material over top. This section provides general geotechnical recommendations for the closure of the WRSF.

Placement of topsoil directly above coarse rockfill material may be subject to ingress of topsoil into the rockfill. Robust geotechnical solutions would involve placement of compatible filter layers between the rockfill and the topsoil. However, an alternative approach could be used as the hydraulic gradient is relatively low and AMNS can start the slope closure progressively, allowing observation of restored area and later mitigation if required.

The alternative approach yet involves placing topsoil directly on the top of the rockfill, by paying particular attention to the rockfill material placed at the surface. The final rockfill slope surface should be placed by removing the largest rockfill particles to avoid nest of coarser particles of high void ratio. In other words, the slope must be made of well-graded materials, where the remaining large particles are all well surrounded by a finer rockfill materials. Although this approach cannot be seen as a robust solution, it is expected to be probably enough to keep the topsoil in place until regrowth of the vegetation. The vegetation will stabilize the topsoil material.

The topsoil placed as a cover closure layer is susceptible to the runoff water erosion process. This process will be active until the vegetation starts to grow, further stabilizing the topsoil. A progressive closure will allow adding mitigation measures if the erosion process does not allow the growth of vegetation. If needed, alternative measures such as seeding or hydroseeding may help quicker growth of the vegetation, which would increase the erosion resistance.

The final pile's crest surface at closure should be sloped towards the external slope to avoid ponding of water at the surface. It is recommended to achieve a minimum slope of 1%. Placement of the topsoil layer should be done after the end of the settlement process as it may be required to add or reprofile the material to meet this recommendation.

8.0 INVESTIGATION AND TESTING PROGRAM TO POTENTIALLY INCREASE THE WRSF CAPACITY

The original design provided by AMNS considered a maximum elevation of 190 m for the WRSF to attain the maximum storage capacity required to manage the waste rock per the life-of-mine plan. Considering the minimum required safety factor of 1.3 in static conditions, and using the existing data on subsurface soil properties, for the final elevations greater than 170 m, the stability criteria are not met.

To define the minimum required safety factor the level of confidence on the data was considered as low. Additionally, the shear strength parameters of the foundation soil were determined based on the limited available field and laboratory testing data. Potentially higher elevations could be attained if the design is supported by more detailed data on the foundation soil. The design can be improved if:

- A higher shear resistance is obtained for the foundation soil.
- With higher level of confidence on the material properties, a lower safety factor is considered to be acceptable (see Table 2 in Section 5.5).

Therefore, a geotechnical site investigation is required to obtain the necessary information on the foundation soil characteristics. The investigation should provide either the possibility of targeting a lower FS by improving the level of confidence on the material properties or by obtaining higher shear strength for the foundation soil. Both field and laboratory tests can be carried out for this purpose:

Field Tests

In order to characterize the mechanical behaviour of the clayey till foundation and to evaluate diverse locations of the site, the following in situ tests are proposed:

- Cone Penetration Test (CPT) with porewater pressure dissipation tests, which provides detailed data on soil shear resistance and an understanding of the distribution of undrained shear strength (Su) over the site, including the hydraulic conductivity and consolidation rate of the clayey till.

- Installation of piezometers (VWP) to monitor the porewater generation in the foundation due to construction of the WRSF.
- SPT tests with hammer energy calibration in order to correct and possibly obtain higher N_{SPT} values, compared to the previous site investigation.

Laboratory Tests

The following laboratory tests are proposed:

- One-dimensional consolidation (oedometer) testing to determine the overconsolidation state of the clayey till. The material behaviour while sheared is a function of the level of overconsolidation, and the preconsolidation pressure can be related to the soil shear strength.
- Laboratory (miniature) vane shear tests to determine the undrained soil shear strength to support the calibration of the CPT tests.
- Soil index testing to characterize soil index properties and compare them to the previous geotechnical site investigation.
- Triaxial consolidated undrained shear (CU) testing (to determine the soil's undrained shear strength, in the eventuality that the oedometer tests do not provide the necessary results).

It should be noted that both in situ and laboratory testing are proposed to ensure that the necessary data is gathered within AMNS's schedule limitations. If the timeframe is extended, a more limited site investigation program could be defined, and the investigation scope could be revised while the studies are conducted.

9.0 CONCLUSION

This report summarizes the stability assessment carried out for the WRSF at Touquoy Mine. The pile's outline proposed by AMNS is evaluated and a maximum attainable height on the foundation soil is determined according to the available data. Based on the Guideline for Mine Waste Dump and Stockpile Design (Hawley and Cunning 2017), the level of consequence of the waste rock dump failure on the site was regarded as low, and the level of confidence on material properties and failure mechanisms was also considered low. Therefore, the minimum factor of safety was considered as 1.3 for static, and 1.1 for pseudo-static and post-liquefaction conditions. Based on the results of the stability analyses and using the existing information on the foundation soil, the pile is stable up to a maximum elevation of 170 m. This maximum height is achievable assuming that the porewater pressure generated in the foundation during construction dissipates fast enough that the soil shear strength is fully mobilized.

Considering that the groundwater table is close to the ground surface, and the foundation soil has a high percentage of fines, porewater pressure can potentially generate in the foundation due to the pile's construction. Due to the lack of knowledge on the hydraulic behaviour of the foundation soil, the rate of porewater pressure cannot be calculated. Since the consequence of failure of the pile is low, an observational method was proposed to ensure that mitigation measures are planned in case of presence of instability signs in the pile or in the foundation. Given that there is enough space within the WRSF's footprint, in case signs of instability are observed after placement of a lift, the construction can be seized in the sector and other sections of the pile can be used while the foundation soil consolidates and pile/foundation movements cease. The adoption of the observational method requires training of AMNS's engineers, technicians and workers to duly react based on the observational method plan. A first draft of the plan is provided in this report and will be updated as the operation progresses and as the pile and foundation behaviours are observed.

Based on good monitoring and surveillance data, as well as the good use of the observational approach, if the pile is built to 170 m elevation without development of significant settlement or sign of instability, it is considered that the WRSF may be built safely to a higher elevation without completing additional investigation. Golder would be pleased to support AMNS with this approach once the pile reaches the elevation of 170 m. Alternatively, if a quicker response is needed to attain the originally proposed pile height (maximum elevation of 190 m), it is required to carry out a geotechnical site investigation to obtain the necessary information on the foundation soil characteristics. The study should provide either the possibility of targeting a lower factor of safety by improving the level of confidence on the material properties or obtaining higher shear strength values for the foundation soil. The site investigation plan is summarized in Section 8.0 of the present report and is presented to AMNS in a separate document (Golder 2020).

10.0 CLOSURE

We trust this information is sufficient for your needs at this time. Should you have any questions or concerns, please contact the undersigned.

Golder Associates Ltd.

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Geotechnical Engineer

Yves Boulianne, P.Eng.(Qc)
Associate, Senior Geotechnical Engineer

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MK/YB/MR/kd/cc

11.0 REFERENCES

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NOT FOR CONSTRUCTION

NOTE

THE GOOGLE EARTH PHOTO AIMS O SHOW THE FOOTPRINT OF THE WRSF, AND DOES NOT INDICATE THE LATEST OPERATIONS.

REFERENCE

AERIAL IMAGERY : GOOGLE EARTH, 2020

0 100 200
1:5 000 METRES

CLIENT
ATLANTIC MINING NS CORP.
TOUQUOY GOLD MINE

PROJECT
WASTE ROCK STORAGE FACILITY
GEOTECHNICAL STABILITY REVIEW

TITLE
PLAN VIEW OF THE MINE SITE

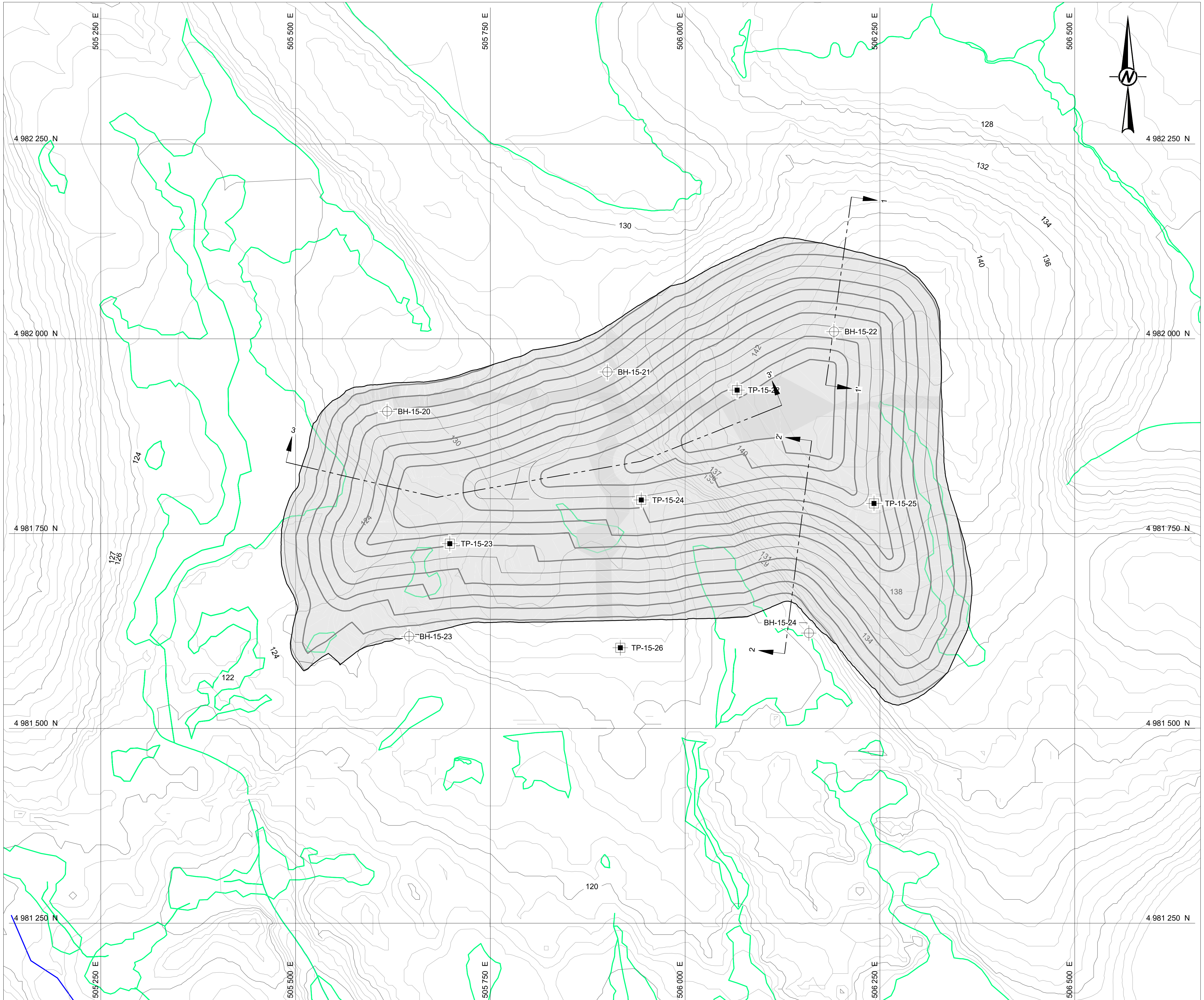
CONSULTANT

GOLDER

YYYY-MM-DD	2020-04-07
DESIGNED	A. Touchette
PREPARED	E. Nkamegue
REVIEWED	M. Kermani
APPROVED	Y. Boulianne

PROJECT NO.	PHASE	REV.	FIGURE
18108591	7000	0	1

25 mm
IF THIS MEASUREMENT DOES NOT MATCH WHAT IS SHOWN, THE SHEET SIZE HAS BEEN MODIFIED FROM: ANSI B



LEGEND

- BOREHOLE LOCATIONS
- TEST PIT LOCATIONS
- TOPOGRAPHIC CONTOUR (INTERVAL 1 m)
- WATERCOURSE
- WETLAND

NOT FOR CONSTRUCTION

NOTE

COORDINATES SYSTEM: UTM NAD83, ZONE 20

REFERENCES

- DRAWING "1m LIDAR surface.dxf"
- DRAWING "Wetlands and watercourses - 2019.dxf"

0 100 200

1:5 000 METRES

CLIENT

ATLANTIC MINING NS CORP.
TOUQUOY GOLD MINE

PROJECT

WASTE ROCK STORAGE FACILITY
GEOTECHNICAL STABILITY REVIEW

TITLE

STUDIED CROSS-SECTIONS

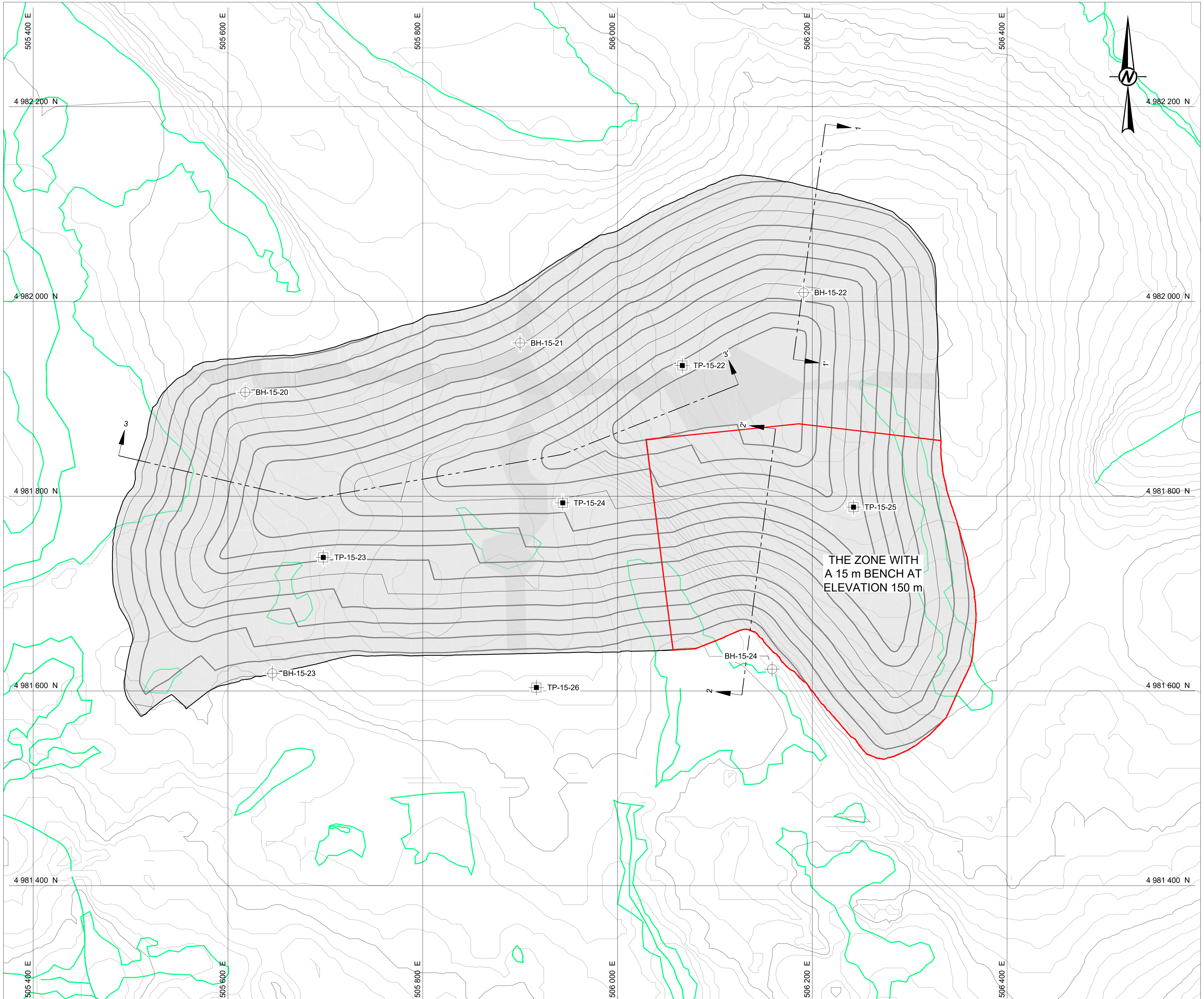
CONSULTANT

GOLDER

YYYY-MM-DD	2020-04-07
DESIGNED	R. Gravel
PREPARED	E. Nkamegue
REVIEWED	M. Kermani
APPROVED	Y. Boulianne

PROJECT NO.	PHASE	REV.	FIGURE
18108591	7000	0	2

25 mm IF THIS MEASUREMENT DOES NOT MATCH WHAT IS SHOWN, THE SHEET SIZE HAS BEEN MODIFIED FROM: ANSI B



LEGEND

BOREHOLE LOCATIONS

TEST PIT LOCATIONS

TOPOGRAPHIC CONTOUR (INTERVAL 1 m)

WATERCOURSE

WETLAND

THE ZONE WITH A 15 m BENCH AT ELEVATION 150 m

NOT FOR CONSTRUCTION

NOTE

COORDINATES SYSTEM: UTM NAD83, ZONE 20

REFERENCES

- DRAWING "1m LIDAR surface.dxf"
- DRAWING "Wetlands and watercourses - 2019.dxf"

0 100 200

1:5 000 METRES

CLIENT
ATLANTIC MINING NS CORP.
TOUQUOY GOLD MINE

PROJECT
WASTE ROCK STORAGE FACILITY
GEOTECHNICAL STABILITY REVIEW

TITLE
THE ZONE WITH A 15 m WIDE BENCH AT ELEVATION 150

CONSULTANT

GOLDER

YYYY-MM-DD	2020-04-07
DESIGNED	A. Touchette
PREPARED	E. Nkamegue
REVIEWED	M. Kermani
APPROVED	Y. Boulianne

PROJECT NO.	PHASE	REV.	FIGURE
18108591	7000	0	3

IF THIS MEASUREMENT DOES NOT MATCH WHAT IS SHOWN, THE SHEET SIZE HAS BEEN MODIFIED FROM: ANSI B

25 mm

APPENDIX A

WRSF Classification Based on Hawley and Cunning (2017)

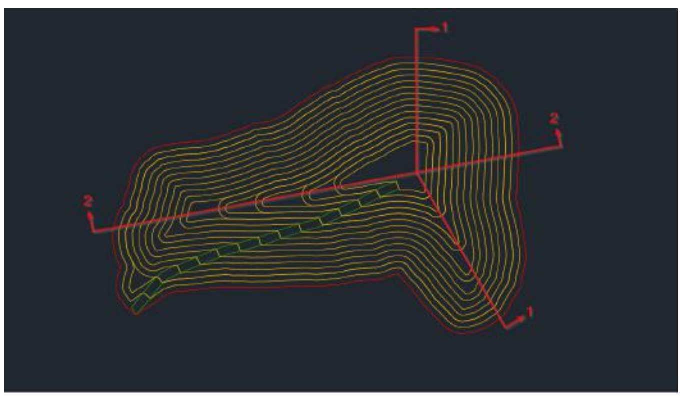
GOLDER

WASTE DUMP HAZARD CLASS (WHC) AND INSTABILITY HAZARD ASSESSMENT BASED ON HAWLEY AND CUNNING (2017)

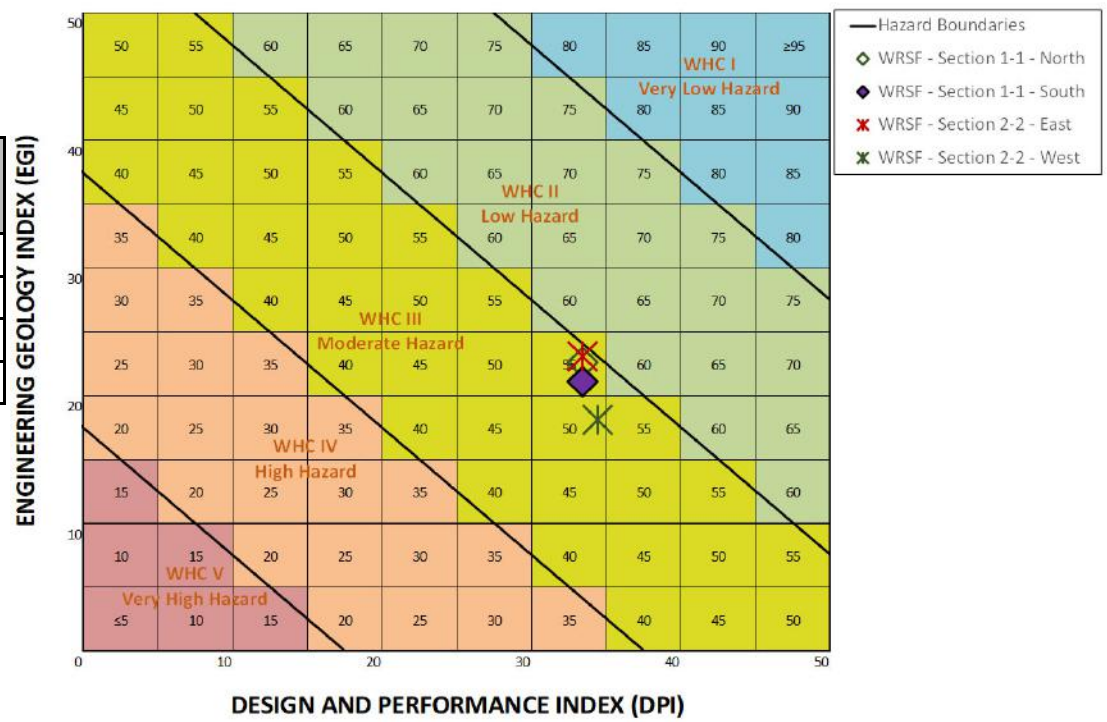
Project 18108591-2000 Client
Atlantic Mining NS Corp.
Prepared by: PDGF 2019-11-10
Revised by: MK 2020-03-19

Summary of the results of the hazard classification (WSRHC)

Empilement ou Halde	Design and Performance Index	Engineering Geology Index (EGI)	Waste Dump and Stockpile Stability Rating (WSR)	Waste Dump and Stockpile Hazard Class (WHC)
WRSF - Section 1-1 - North	33.5	22.5	56	III
WRSF - Section 1-1 - South	33.5	21	54.5	III
WRSF - Section 2-2 - East	33.5	23	56.5	III
WRSF - Section 2-2 - West	34.5	18	52.5	III



Waste dump and stockpile stability rating (WSR) and Hazard Class (WHC) chart



WASTE DUMP AND STOCKPILE STABILITY RATING AND HAZARD CLASSIFICATION (WSRHC) SYSTEM AFTER HAWLEY AND CUNNING (2017)

Section 1-1

	Factors [1]	North SCRP	North Rating	South SCRP	South Rating	Notes	Source
Regional Setting	**Seismicity (0 - 2)**	Very Low	2.0	Very Low	2.0	based on 1/475 year return period, PGA=0,023 for soil class C soil	Expected ground peak acceleration (g) obtained from RNCAN (NBCC 2015)
Regional Setting	**Total Annual Precipitation: Equivalent Rainfall (0 - 8)**	1000-2000 mm	2.0	1000-2000 mm	2.0	Annual rainfall = 903,5 mm - Moderate Annual snowfall = 270,7 mm - Very High Rating = High Doesn't take climate change into account.	Greenwood station, located at approximately at 15 km north of the site, from 1981-2010 (Data of environment Canada)
Foundation Conditions	**Average overall foundation slope angle (0 - 5)**	<5°	5.0	5-15°	4.0	South = 5-15° North =±5 °	Slope based on boreholes elevations Stantec (2016)
Foundation Conditions	**Foundation Shape (0 - 2)**	Planar shape on moderate slope with no natural con finement	1.0	Concave shape on steep slopes with no natural confinement	0.5	South : Section shape = Concave on steep slopes =0,5 Plan shape = Planar slopes with no lateral confinement = 1 North : Section shape = Planar shape on moderate slope - 1 Plan shape = Planar slopes with no lateral confinement - 1	Based on the ground elevation at BH15-22; BH-15-24; TP-15-22 and TP15-25
Foundation Conditions	**Overburden Type (0 - 4)**	Type IV	3.0	Type IV	3.0	Stiff to hard fine-grained soil - Type III based on the investigation Stantec (2016)	Based on the interpretation of boreholes in the area and the materials properties Stantec (2016)
Foundation Conditions	**Overburden Thickness (m) (0 - 2)**	>5	0.0	>5	0.0	Up to 12,2 m of overburden	Based on the interpretation of the geotechnical investigation stantec (2016) and the 3D Design.
Foundation Conditions	**Undrained Failure Potential (-20 - 0)**	Moderate	-5.0	Moderate	-5.0	Assumption: Low to moderate hydraulic conductivity; Moderate potential for generation of pore water pressure when loaded rapidly.	Based on the Overburden Type and the interpretation of the investigation. Stantec (2016)
Foundation Conditions	**Foundation Liquefaction Potential (-20 - 0)**	Negligible	0.0	Negligible	0.0	Liquefaction Potential Moderate (or unknown)	Given the proportion of fine-grained soil= Moderate
Foundation Conditions	**Bedrock (0 - 4)**	Type C	2.0	Type C	2.0	Poor to high quality rock (0% <RQD< 87%)	Based on the interpretation of the 2016 investigation (Stantec 2016)
Foundation Conditions	**Groundwater (0 - 3)**	High	0.0	High	0.0	Water level close to the ground surface	Based on the water levels on the test pit.

	Factors [1]	North SCRP	North Rating	South SCRP	South Rating	Notes	Source
Material Quality	**Gradation (0 - 7)**	Mixed	3.5	Mixed	3.5	Assumption: % passing 0.075 mm < 25% ; and % coarser than 75 mm > 25%	Based on the information received from the client on the waste rock quality
Material Quality	**Intact Strength and Durability (0 - 8)**	Type 3	4.0	Type 3	4.0	Relatively high UCS (37.0 to 119.7) based on 2007 Geotechnical assessment of open-pit mining (Peter O'Brian & Associates) . waste rock material does not breake down during placement or at least limited. Degradation due to slacking under freeze-thaw cycle or crushing under static loading is more liekly.	Based on the information received from the client on the waste rock quality
Material Quality	**Material Liquefaction Potential (-20 - 0)**	Negligible	0.0	Negligible	0.0	The waste dumps materials are not susceptible to liquefaction - Negligeable.	Based on the information on the waste rock
Material Quality	**Chemical Stability (-5 - 5)**	Neutral	5.0	Neutral	5.0	The acid generating rocks will be surrounded by acid consuming rocks.	s.o.
Geometrey and Mass	**Height (0 - 4)**	Low	3.0	Low	3.0	Overall height = ±65 m; Max vertical thickness = ±50 m; Max individual lift height = 2m (lifts of 2 m)	Interpretation of the investigation and the WRD Design 3D
Geometrey and Mass	**Overall Fill Slope Angle (0 - 4)**	15-25°	3.0	15-25°	3.0	±19,5° = Flat	WRD Design UPDATED 190912.dxf Data obtained from Atlantic Mining NS Corp.
Geometrey and Mass	**Volume and Mass (0 - 2)**	Small	1.5	Small	1.5	Total volume estimated to be ±8 M m^3	s.o.
Stability Analysis	**Static Stability - Factor of Safety (0 - 7)**	1.3-1.5	5.0	1.3-1.5	5.0	The consequence of failure is rated as moderate and the level of confidence on the material is rated as low	the stability acceptance criteria suggest a minimum factor of safety of 1.4 to 1.5 in static analysis . The SWP will be designed to achieve a factor of safety exceeding 1.5 in static condition
Stability Analysis	**Dynamic Stability - Factor of Safety (0 - 3)**	1.10-1.15	2.0	1.10-1.15	2.0	The consequence of failure is rated as moderate and the level of confidence on the material is rated as low	the stability acceptance criteria suggest a minimum factor of safety of 1.10 to 1.15 in pseudo-static analysis. The SWP will be designed to achieve a factor of safety exceeding 1.15 in pseudo-static condition
Construction	**Construction Method (0 - 8)**	Method V	8.0	Method V	8.0	Ascending, lifts of 2 m, foundation slope of 15-18° = Method IV	Based on information provided by the client on the construction sequences.
Construction	**Loading Rate (0 - 7)**	Moderate	3.5	Moderate	3.5	Based on the construction period and the total volume.	Total volume estimated of ±8 M m3 and 4-year construction period
Performance	**Stability Performance (-15 - 15)**	Good	7.5	Good	7.5	Good stability performance	Based on the quarterly inspection visits done by Golder.

[1] Numbers in parentheses represent the lower and higher bounds of possible ratings

WASTE DUMP AND STOCKPILE STABILITY RATING AND HAZARD CLASSIFICATION (WSRHC) SYSTEM AFTER HAWLEY AND CUNNING (2017)

Section 2-2

	Factors [1]	East SCRP	East Rating	West SCRP	West Rating	Notes	Source
Regional Setting	**Seismicity (0 - 2)**	Very Low	2.0	Very High	0.0	based on 1/475 year return period, PGA=0,023 for soil class C soil	Expected ground peak acceleration (g) obtained from RNCAN (NBCC 2015)
Regional Setting	**Total Annual Precipitation: Equivalent Rainfall (0 - 8)**	1000-2000 mm	2.0	1000-2000 mm	2.0	Annual rainfall = 903,5 mm - Moderate Annual snowfall = 270,7 mm - Very High Rating = High Doesn't take climate change into account.	Greenwood station, located at approximately at 15 km north of the site, from 1981-2010 (Data of environment Canada)
Foundation Conditions	**Average overall foundation slope angle (0 - 5)**	<5°	5.0	15-25°	2.5	West = 0 ° East = 19 °	slope based on boreholes elevations Stantec (2016)
Foundation Conditions	**Foundation Shape (0 - 2)**	Planar shape on flat slope with no lateral confinement	1.5	Planar shape on moderate slope with no lateral confinement	1.0	East : Section shape = Planar on flat slopes - 2 Plan shape = Planar slopes with no lateral confinement - 1 West : Section shape = Planar shape on moderate slope - 1 Plan shape = Planar slopes with no lateral confinement - 1	Based on the ground elevation at TP-15-23, TP-15-24, TP-15-25
Foundation Conditions	**Overburden Type (0 - 4)**	Type IV	3.0	Type IV	3.0	Stiff to hard fine-grained soil - Type III based on the investigation Stantec (2016)	Based on the interpretation of boreholes in the area and the materials properties Stantec (2016)
Foundation Conditions	**Overburden Thickness (m) (0 - 2)**	>5	0.0	>5	0.0	Up to 8 m of Overburden	Based on the interpretation of the geotechnical investigation Stantec (2016) and the 3D Design
Foundation Conditions	**Undrained Failure Potential (-20 - 0)**	Moderate	-5.0	Moderate	-5.0	Assumption: Low to moderate hydraulic conductivity; Moderate potential for generation of pore water pressure when loaded rapidly.	Based on the Overburden Type and the interpretation of the investigation. Stantec (2016)
Foundation Conditions	**Foundation Liquefaction Potential (-20 - 0)**	Negligible	0.0	Negligible	0.0	Liquefaction Potential Moderate (or unknown)	Given the proportion of fine-grained soil= Moderate
Foundation Conditions	**Bedrock (0 - 4)**	Type C	2.0	Type C	2.0	Poor to high quality rock (0% <RQD< 87%)	Based on the interpretation of the 2016 investigation (Stantec 2016)
Foundation Conditions	**Groundwater (0 - 3)**	High	0.0	High	0.0	Water level close to the ground surface	Based on the water levels on the test pit.

	Factors [1]	East SCRP	East Rating	West SCRP	West Rating	Notes	Source
Material Quality	**Gradation (0 - 7)**	Mixed	3.5	Mixed	3.5	Assumption: % passing 0.075 mm < 25% ; and % coarser than 75 mm > 25%	Based on the information received from the client on the waste rock quality
	Intact Strength and Durability (0 - 8)	Type 3	4.0	Type 3	4.0	Relatively high UCS (37.0 to 119.7) based on 2007 Geotechnical assessment of open-pit mining (Peter O'Brian & Associates) . waste rock material does not breake down during placement or at least limited. Degradation due to slacking under freeze-thaw cycle or crushing under static loading is more liekly.	Based on the information received from the client on the waste rock quality
	Material Liquefaction Potential (-20 - 0)	Negligible	0.0	Negligible	0.0	The waste dumps materials are not susceptible to liquefaction - Negligeable.	Based on the information on the waste rock
	Chemical Stability (-5 - 5)	Neutral	5.0	Neutral	5.0	The acid generating rocks will be surrounded by acid consuming rocks.	s.o.
Geometrey and Mass	**Height (0 - 4)**	Low	3.0	Low	3.0	West: Overall height & Max vertical thickness =±65 & 50m; max individual lift height = 2m East : Overall height =± 60 m; Max vertical thickness = ±10 m; Max individual lift height = 2m (lift of 2 m)	Interpretation of the investigation and the WRD Design 3D
	Overall Fill Slope Angle (0 - 4)	15-25°	3.0	<15°	4.0	West : 9° = Very flat East : 18,3° = Flat	WRD Design UPDATED 190912.dxf Data obtained from Atlantic Mining NS Corp.
	Volume and Mass (0 - 2)	Small	1.5	Small	1.5	Total volume estimated to be ±8 M m^3	s.o.
Stability Analysis	**Static Stability - Factor of Safety (0 - 7)**	1.3-1.5	5.0	1.3-1.5	5.0	The consequence of failure is rated as moderate and the level of confidence on the material is rated as low	the stability acceptance criteria suggest a minimum factor of safety of 1.4 to 1.5 in static analysis . The SWP will be designed to achieve a factor of safety exceeding 1.5 in static condition.
	Dynamic Stability - Factor of Safety (0 - 3)	1.10-1.15	2.0	1.10-1.15	2.0	The consequence of failure is rated as moderate and the level of confidence on the material is rated as low	the stability acceptance criteria suggest a minimum factor of safety of 1.10 to 1.15 in pseudo-static analysis. The SWP will be designed to achieve a factor of safety exceeding 1.15 in pseudo-static condition
Construction	**Construction Method (0 - 8)**	Method V	8.0	Method V	8.0	Ascending, lifts of 2 m, foundation slope of 15-18° = Method IV	Based on the information provided by the client on the construction sequences
	Loading Rate (0 - 7)	Moderate	3.5	Moderate	3.5	Based on the construction period and the total volume.	Total volume estimated of ±8 M m3 and 4-year construction period
Performance	**Stability Performance (-15 - 15)**	Good	7.5	Good	7.5	good satbility performance	Based on the quarterly inspection visits done by Golder.

[1] Numbers in parentheses represent the lower and higher bounds of possible ratings

APPENDIX B

Seismic Hazard Calculation Sheet

2015 National Building Code Seismic Hazard Calculation

INFORMATION: **Eastern Canada English (613) 995-5548 français (613) 995-0600 Facsimile (613) 992-8836**
Western Canada English (250) 363-6500 Facsimile (250) 363-6565

Site: 44.989N 62.925W **User File Reference:** Touquoy Mine, Halifax, Nova Scotia 2019-11-14 20:20 UT

Requested by: Golder Associates Ltd.

Probability of exceedance per annum	0.000404	0.001	0.0021	0.01
Probability of exceedance in 50 years	2 %	5 %	10 %	40 %
Sa (0.05)	0.076	0.042	0.026	0.009
Sa (0.1)	0.107	0.062	0.039	0.015
Sa (0.2)	0.106	0.065	0.043	0.018
Sa (0.3)	0.093	0.059	0.041	0.017
Sa (0.5)	0.080	0.053	0.037	0.015
Sa (1.0)	0.052	0.035	0.024	0.009
Sa (2.0)	0.028	0.019	0.013	0.004
Sa (5.0)	0.008	0.005	0.003	0.001
Sa (10.0)	0.003	0.002	0.001	0.001
PGA (g)	0.061	0.036	0.023	0.008
PGV (m/s)	0.068	0.042	0.028	0.009

Notes: Spectral (Sa(T), where T is the period in seconds) and peak ground acceleration (PGA) values are given in units of g (9.81 m/s^2). Peak ground velocity is given in m/s. Values are for "firm ground" (NBCC2015 Site Class C, average shear wave velocity 450 m/s). NBCC2015 and CSAS6-14 values are highlighted in yellow. Three additional periods are provided - their use is discussed in the NBCC2015 Commentary. Only 2 significant figures are to be used. **These values have been interpolated from a 10-km-spaced grid of points. Depending on the gradient of the nearby points, values at this location calculated directly from the hazard program may vary. More than 95 percent of interpolated values are within 2 percent of the directly calculated values.**

References

National Building Code of Canada 2015 NRCC no. 56190; Appendix C: Table C-3, Seismic Design Data for Selected Locations in Canada

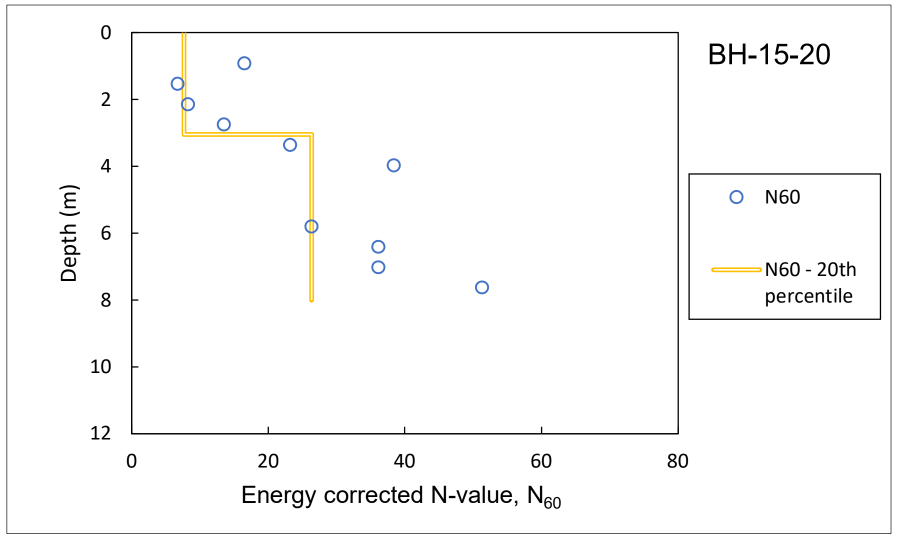
Structural Commentaries (User's Guide - NBC 2015: Part 4 of Division B)
Commentary J: Design for Seismic Effects

Geological Survey of Canada Open File 7893 Fifth Generation Seismic Hazard Model for Canada: Grid values of mean hazard to be used with the 2015 National Building Code of Canada

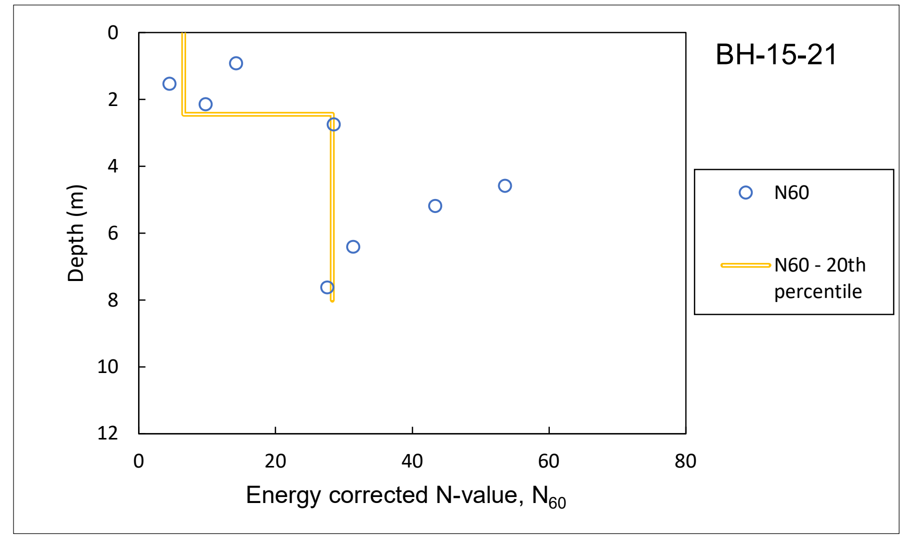
See the websites www.EarthquakesCanada.ca and www.nationalcodes.ca for more information

APPENDIX C

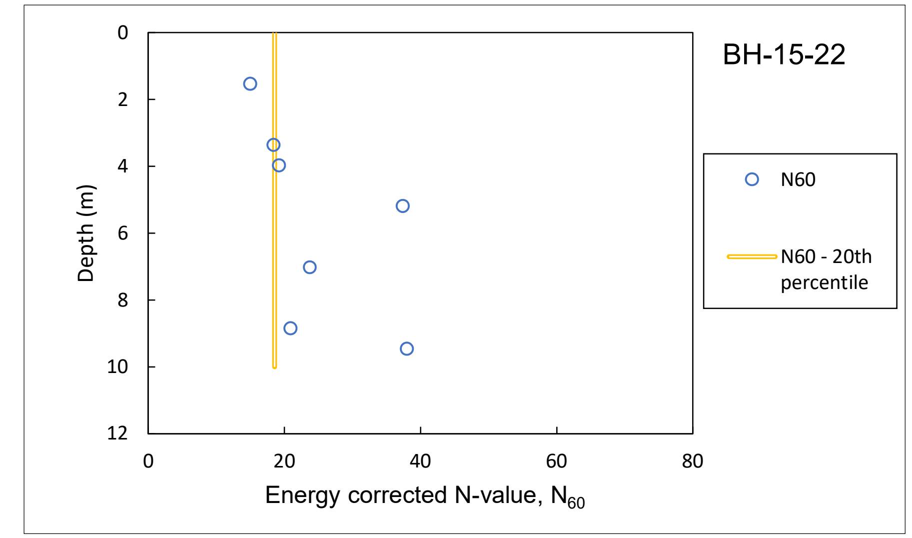
Material Properties Definition



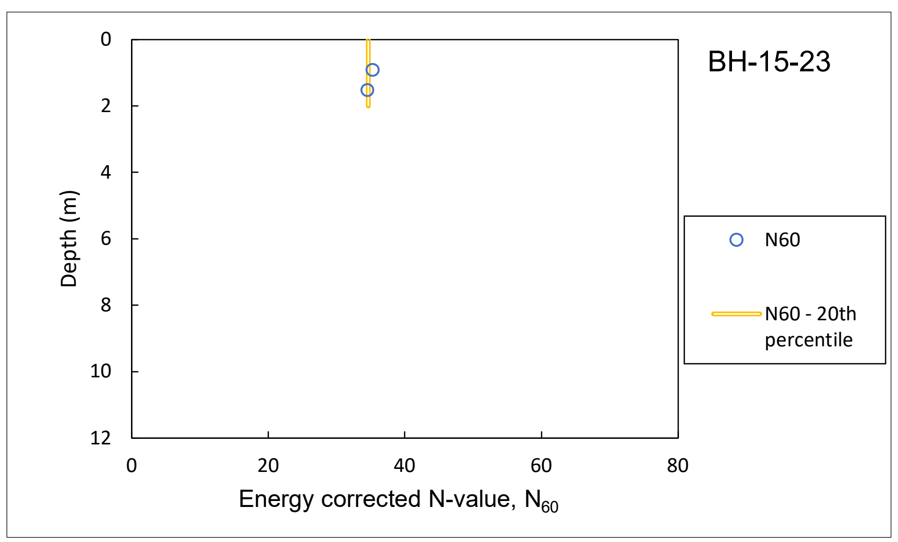
BH-15-20	3-3'		
z_{min} (m)	z_{max} (m)	$N_{60\text{-}20thperc.}$	su (kPa)
0	3.05	8	50
3.05	8	26	173



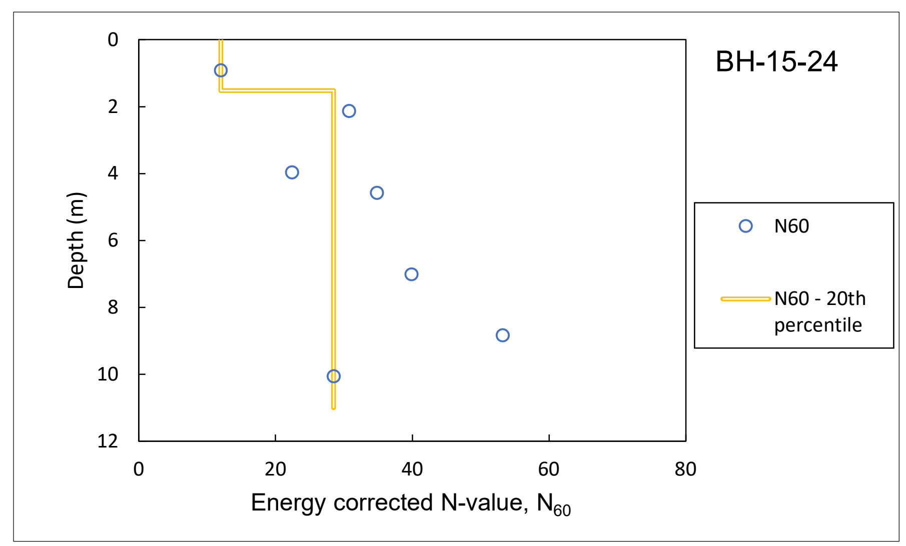
BH-15-21	3-3'		
z_{min} (m)	z_{max} (m)	$N_{60\text{-}20thperc.}$	su (kPa)
0	2.44	7	44
2.44	8	28	187



BH-15-22	1-1'		
z_{min} (m)	z_{max} (m)	$N_{60\text{-}20thperc}$	su (kPa)
0	10	19	127



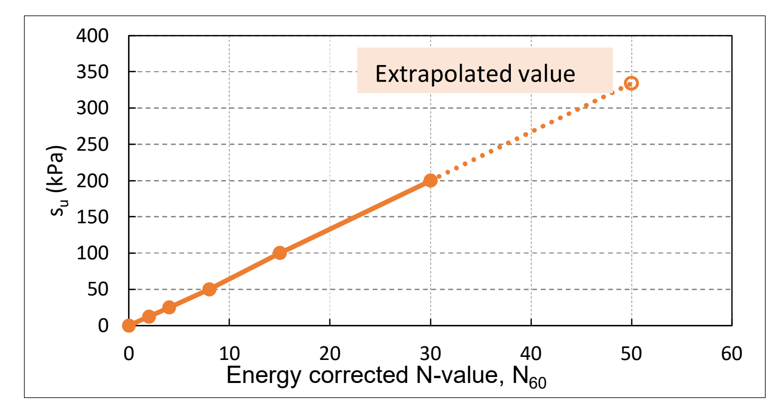
BH-15-23	3-3'		
z_{min} (m)	z_{max} (m)	$N_{60\text{-}20thperc.}$	su (kPa)
0	2	35	231



BH-15-24	2-2'		
z_{min} (m)	z_{max} (m)	$N_{60\text{-}20thperc.}$	su (kPa)
0	1.52	12	75
1.52	11	29	193

Consistency (cohesive soils)

Term	Undrained shear strength, s_u (kPa)			Energy corrected N-value, N_{60}		
Very soft	0	to	12	0	to	2
Soft	12	to	25	2	to	4
Firm	25	to	50	4	to	8
Stiff	50	to	100	8	to	15
Very stiff	100	to	200	15	to	30
Hard	>200			>30		



APPENDIX D

Stability Analysis Results

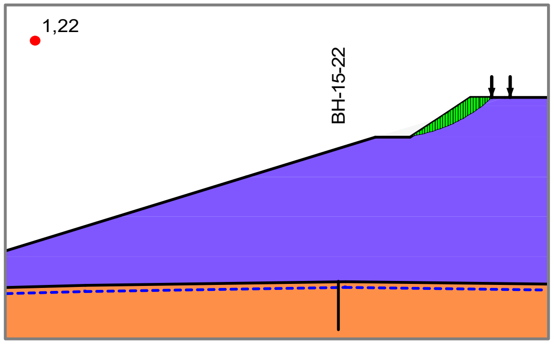
STABILITY ANALYSES
TOUQUOY MINE WASTE ROCK STORAGE FACILITY

FIGURE D-1

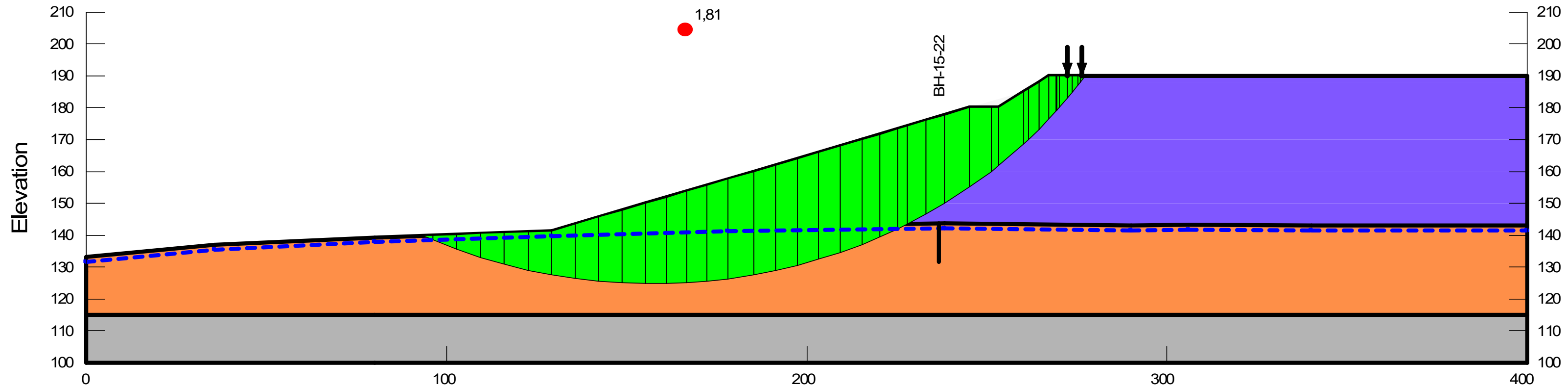
Section 1-1' - Construction of ultimate lift (190 m)
Static condition
ESA

Color	Name	Model	Unit Weight (kN/m³)	Phi' (°)
	01 - Clayey till	Mohr-Coulomb	21	27
	02 - Bedrock	Bedrock (Impenetrable)		
	03a - Waste rock	Mohr-Coulomb	21	35

Type 1 failure FS = 1.22



Type 3 failure **FS =1.81**



Date : January 2020
Project: 18108591-4000

Golder Associés Ltée

Prepared by: EN
Reviewed by: MLBL

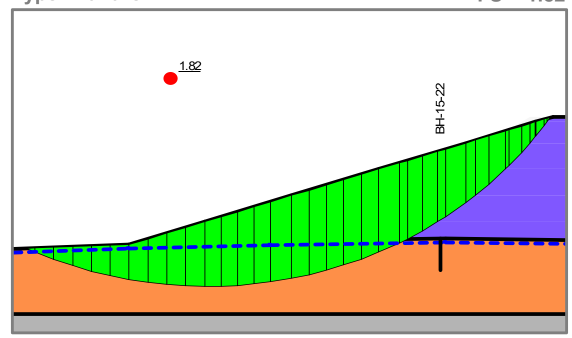
STABILITY ANALYSES
TOUQUOY MINE WASTE ROCK STORAGE FACILITY

FIGURE D-2

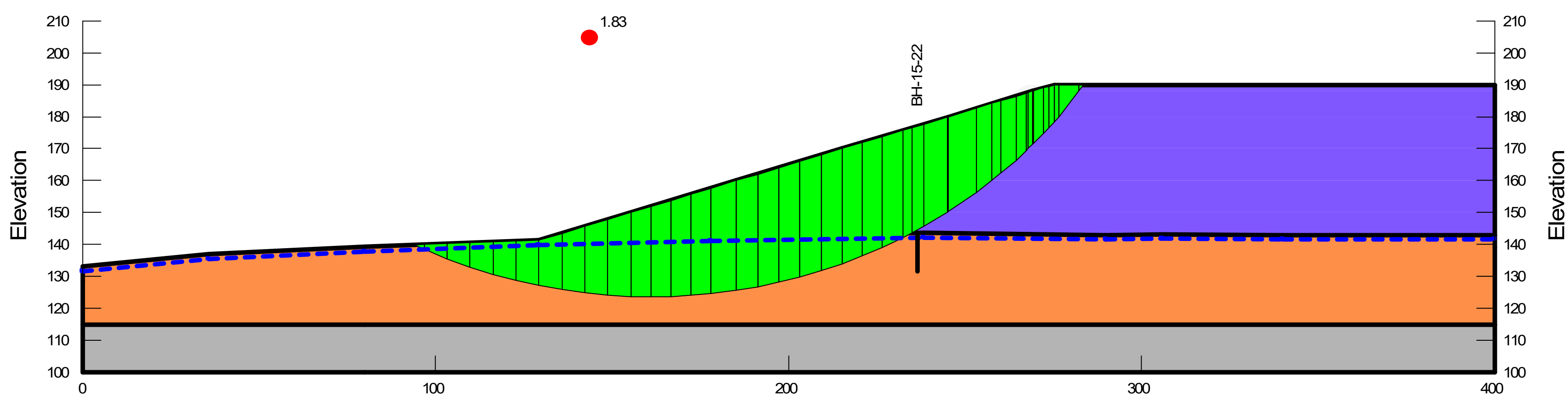
Section 1-1' - Final configuration (190 m)
Static condition
ESA

Color	Name	Model	Unit Weight (kN/m³)	Phi' (°)
(orange)	01 - Clayey till	Mohr-Coulomb	21	27
(grey)	02 - Bedrock	Bedrock (Impenetrable)		
(purple)	03a - Waste rock	Mohr-Coulomb	21	35

Type 2 failure FS = 1.82



Type 3 failure **FS =1.83**



Date : January 2020
Project: 18108591-4000

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STABILITY ANALYSES
TOUQUOY MINE WASTE ROCK STORAGE FACILITY

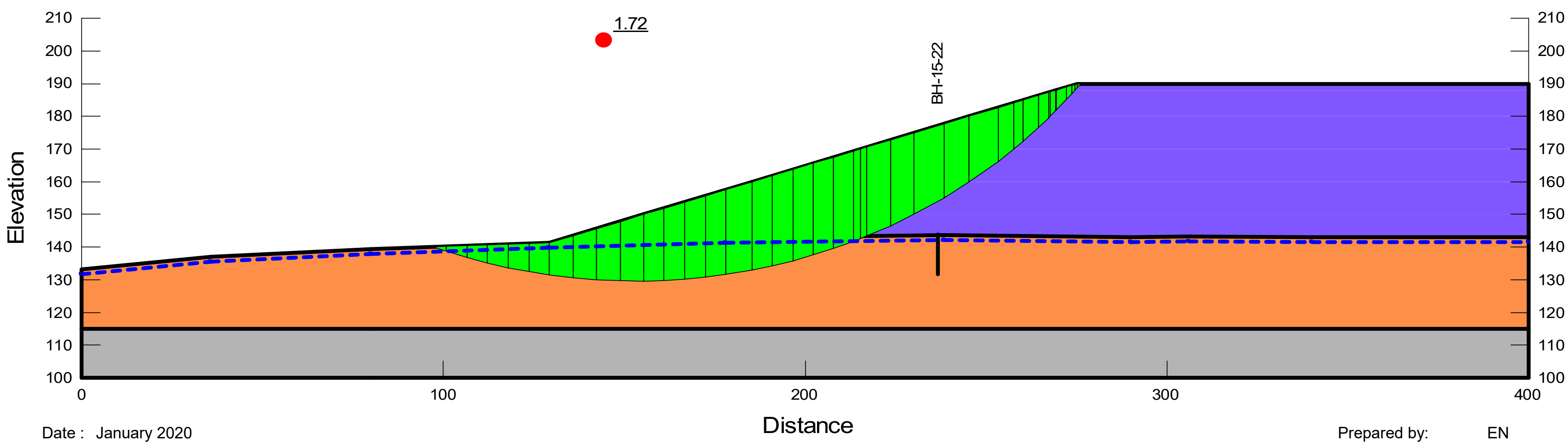
FIGURE D-3

Section 1-1' - Final configuration (190 m) - Lower bound resistance of waste rock
Static condition
ESA

Color	Name	Model	Unit Weight (kN/m³)	Phi' (°)
	01 - Clayey till	Mohr-Coulomb	21	27
	02 - Bedrock	Bedrock (Impenetrable)		
	03a - Waste rock	Mohr-Coulomb	21	30

Type 2* failure
*extends to the crest

FS =1.72



Date : January 2020
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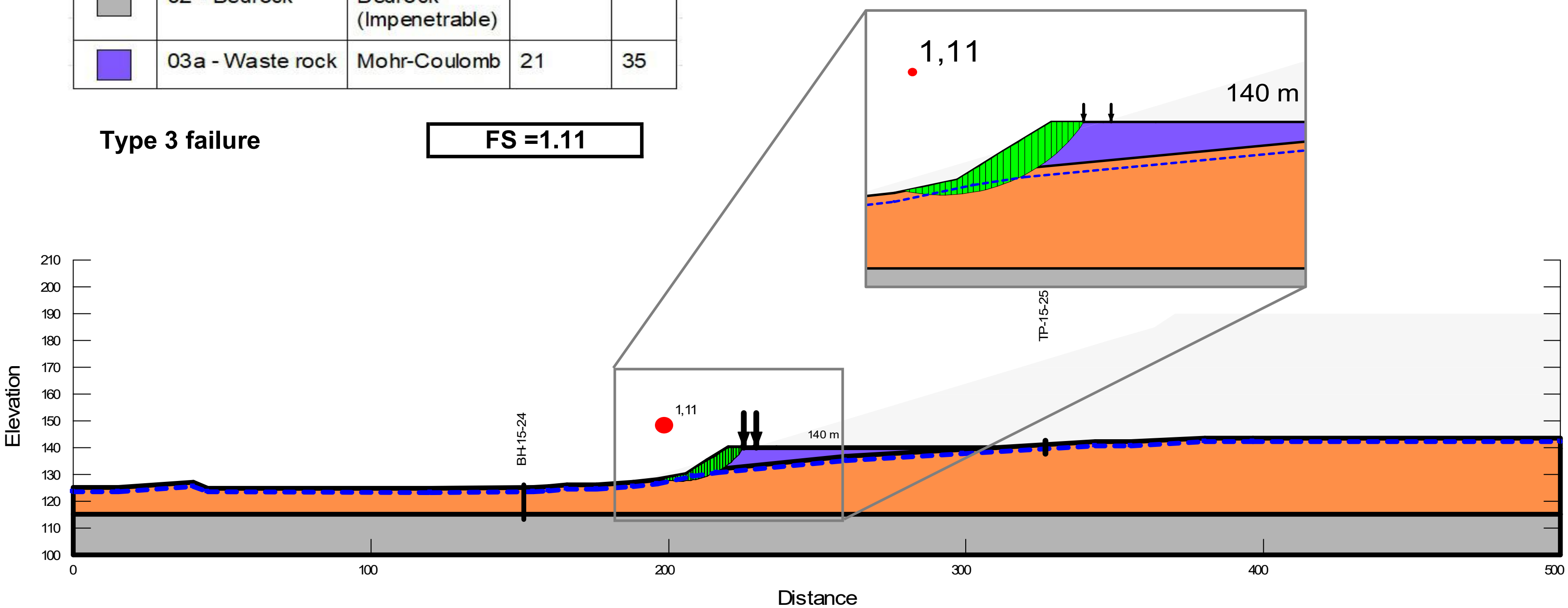
STABILITY ANALYSES
TOUQUOY MINE WASTE ROCK STORAGE FACILITY

FIGURE D-4

Section 2-2' - Construction of first lift (140 m)
Static condition
ESA

Color	Name	Model	Unit Weight (kN/m³)	Phi' (°)
	01 - Clayey till	Mohr-Coulomb	21	27
	02 - Bedrock	Bedrock (Impenetrable)		
	03a - Waste rock	Mohr-Coulomb	21	35

Type 3 failure **FS =1.11**



Date : January 2020
Project: 18108591-4000

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Reviewed by: MLBL

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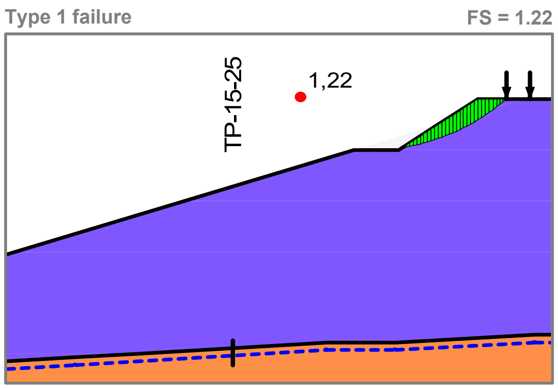
TOUQUOY MINE WASTE ROCK STORAGE FACILITY

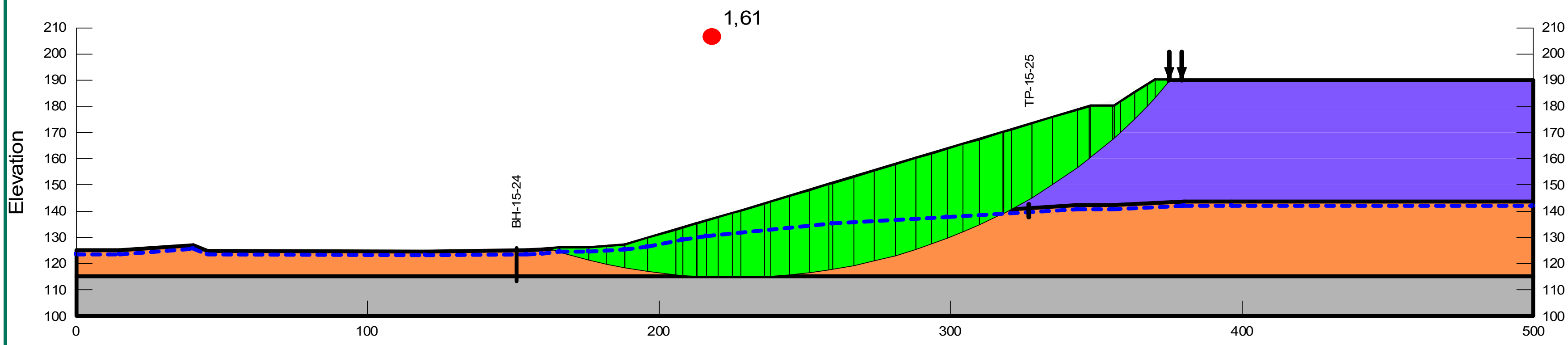
FIGURE D-5

Section 2-2' - Construction of ultimate lift (190 m)
Static condition
ESA

Color	Name	Model	Unit Weight (kN/m³)	Phi' (°)
	01 - Clayey till	Mohr-Coulomb	21	27
	02 - Bedrock	Bedrock (Impenetrable)		
	03a - Waste rock	Mohr-Coulomb	21	35



Type 3 failure **FS =1.61**



Date : January 2020
Project: 18108591-4000

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Reviewed by: MLBL

STABILITY ANALYSES
TOUQUOY MINE WASTE ROCK STORAGE FACILITY

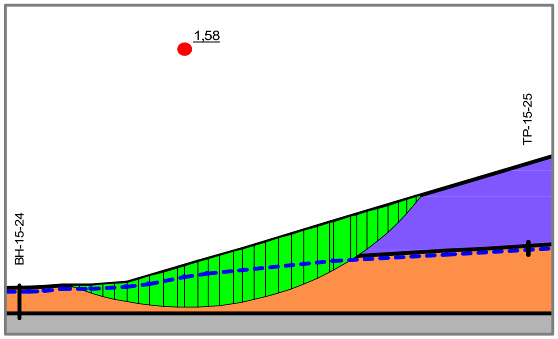
FIGURE D-6

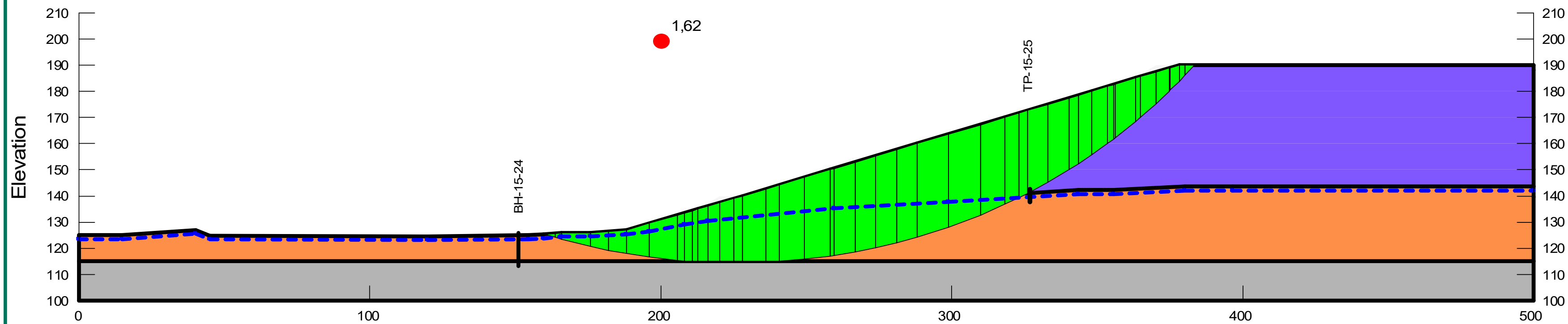
Section 2-2' - Final configuration (190 m)
Static condition
ESA

Color	Name	Model	Unit Weight (kN/m³)	Phi' (°)
	01 - Clayey till	Mohr-Coulomb	21	27
	02 - Bedrock	Bedrock (Impenetrable)		
	03a - Waste rock	Mohr-Coulomb	21	35

Type 3 failure **FS =1.62**

Type 2 failure FS = 1.58





Date : January 2020
Project: 18108591-4000

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Reviewed by: MLBL

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TOUQUOY MINE WASTE ROCK STORAGE FACILITY

FIGURE D-7

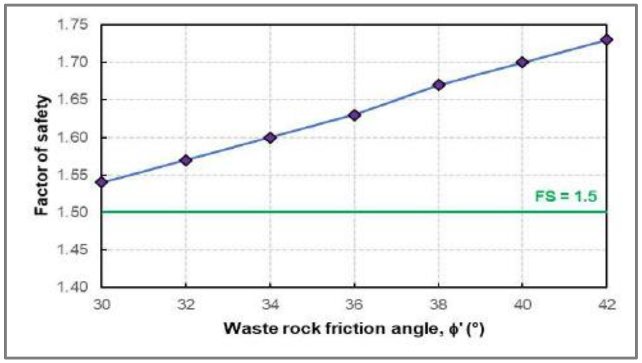
Section 2-2' - Final configuration (190 m) - Lower bound resistance of waste rock
Static condition
ESA

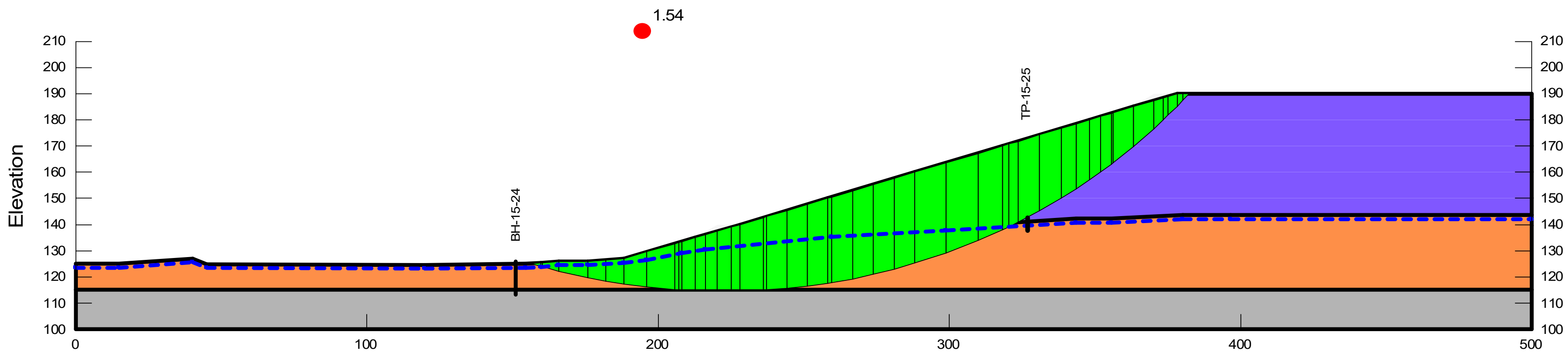
Color	Name	Model	Unit Weight (kN/m³)	Phi' (°)
	01 - Clayey till	Mohr-Coulomb	21	27
	02 - Bedrock	Bedrock (Impenetrable)		
	03a - Waste rock	Mohr-Coulomb	21	30

Type 2* failure
*extends to the crest

FS = 1.54

Sensitivity analysis on waste rock friction angle





Date : January 2020
Project: 18108591-4000

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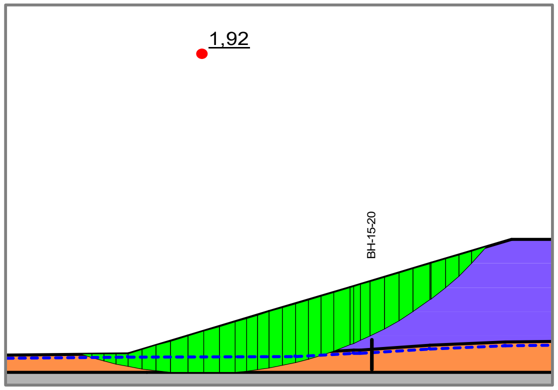
TOUQUOY MINE WASTE ROCK STORAGE FACILITY

FIGURE D-8

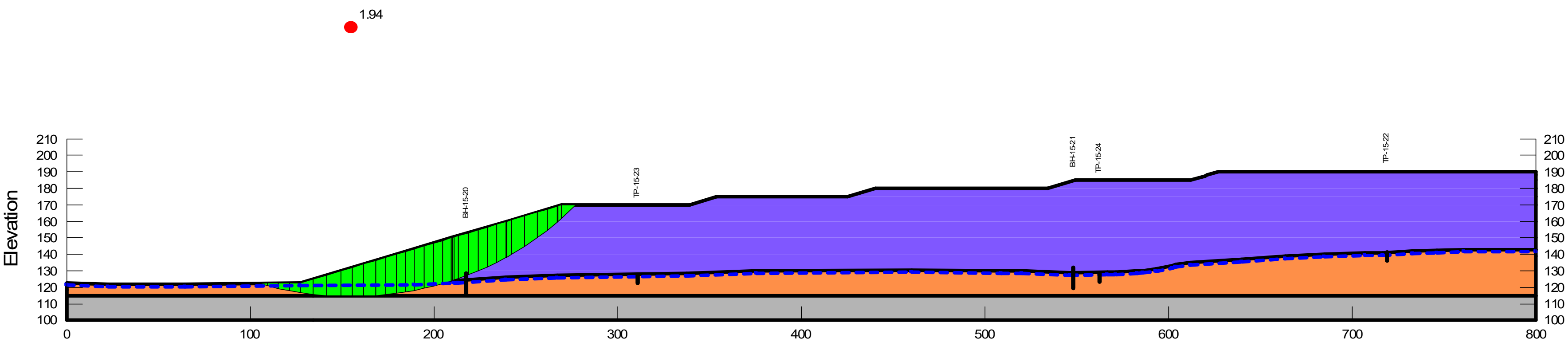
Section 3-3' - Final configuration (190 m)
Static condition
ESA

Couleur	Nom	Modèle	Poids volumique (kN/m³)	Phi' (°)	Résistance minimum (kPa)	Rapport Tau/Sigma
	01b - Clayey Till USA - sumin=50 kPa	S=f(couverture)	21		50	0,22
	01c - Clayey Till USA - sumin=170 kPa	S=f(couverture)	21		170	0,22
	02 - Bedrock	Substratum (impénétrable)				
	03a - Waste rock	Mohr-Coulomb	21	35		

Type 2 failure FS = 1.92



Type 3 failure **FS =1.94**



Date : January 2020
Project: 18108591-4000

Prepared by: EN
Reviewed by: MLBL

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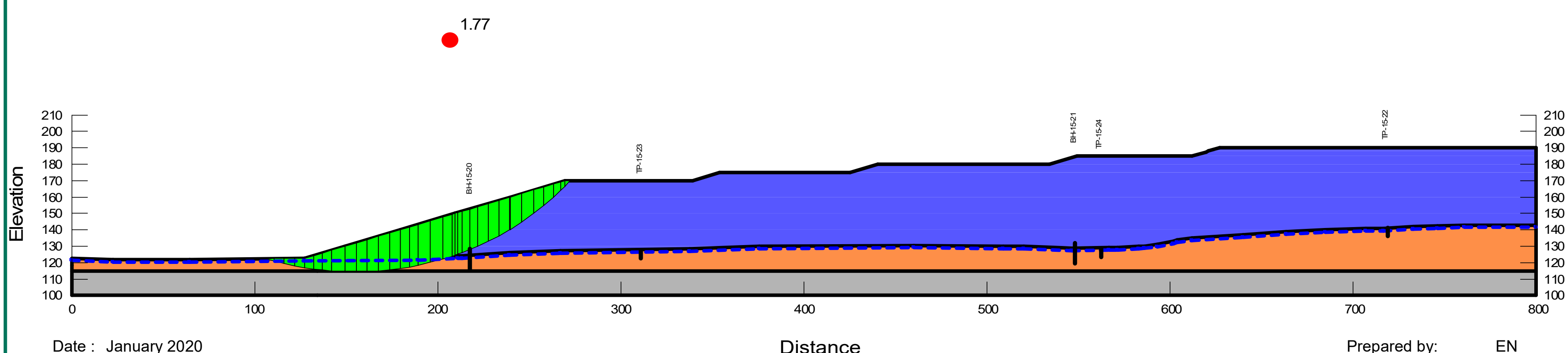
TOUQUOY MINE WASTE ROCK STORAGE FACILITY

FIGURE D-9

Section 3-3' - Final configuration (190 m) - Lower bound resistance of waste rock
Static condition
ESA

Color	Name	Model	Unit Weight (kN/m³)	Phi' (°)
	01 - Clayey Till	Mohr-Coulomb	21	27
	02 - Bedrock	Bedrock (Impenetrable)		
	03b - Waste rock - Low density	Mohr-Coulomb	21	30

Type 3 failure **FS =1.77**



Date : January 2020
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Reviewed by: MLBL

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TOUQUOY MINE WASTE ROCK STORAGE FACILITY

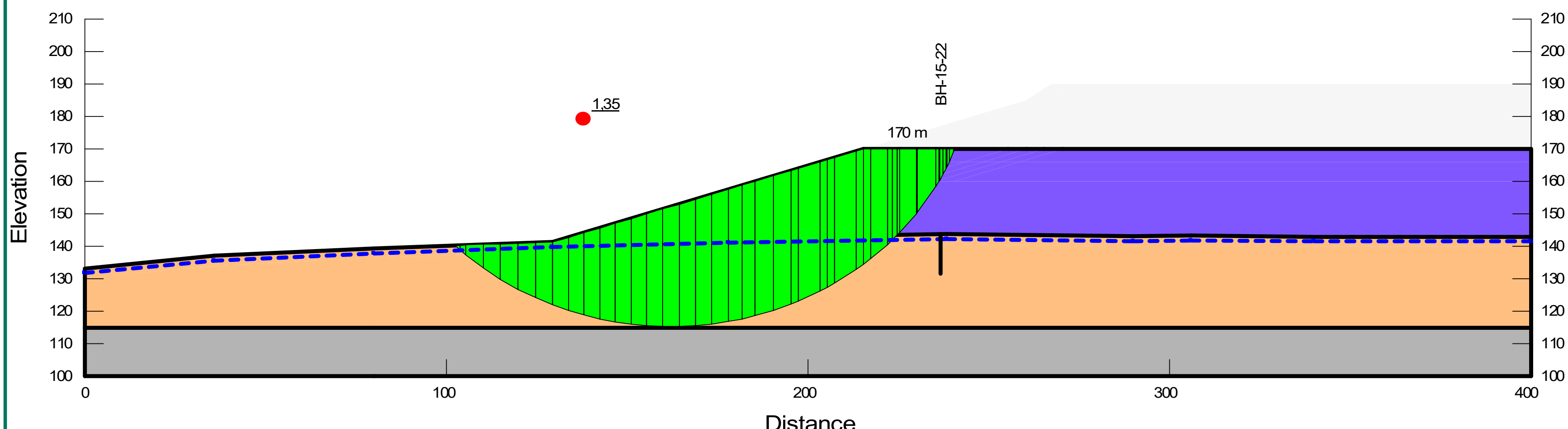
FIGURE D-10

Section 1-1' - 170 m
Static condition
USA peak

Color	Name	Model	Unit Weight (kN/m³)	Phi' (°)	Minimum Strength (kPa)	Tau/Sigma Ratio
	01c - Clayey till - USA peak_sumin=120kPa	SHANSEP	21		120	0,22
	02 - Bedrock	Bedrock (Impenetrable)				
	03a - Waste rock	Mohr-Coulomb	21	35		

Type 3 failure

FS = 1.35



Date : February 2020
Project: 18108591-4000

Prepared by: EN
Reviewed by: SB

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TOUQUOY MINE WASTE ROCK STORAGE FACILITY

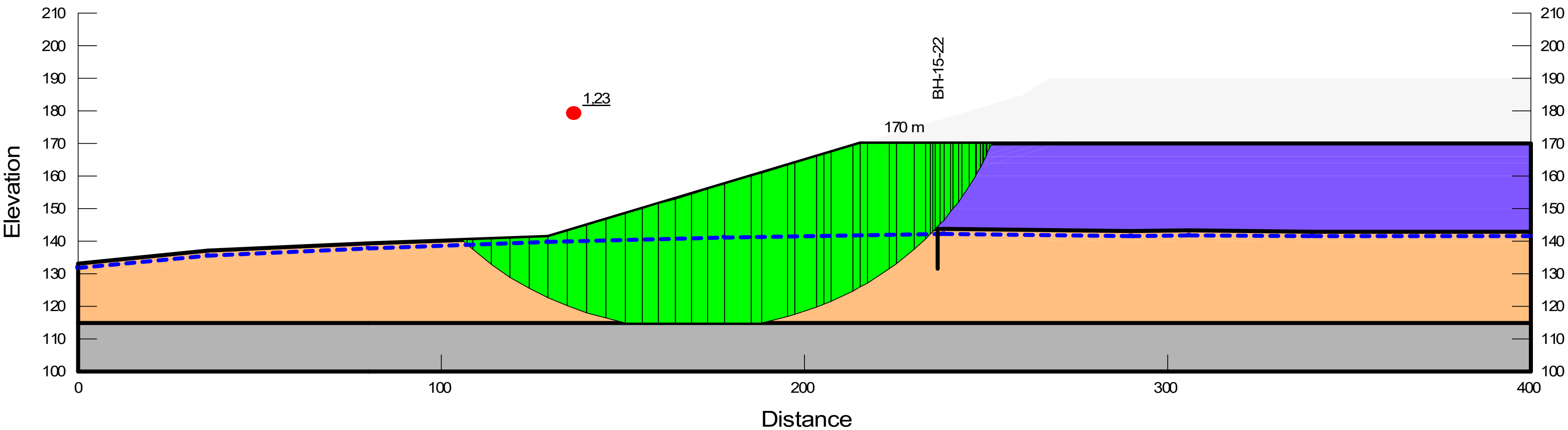
FIGURE D-11

Section 1-1' - 170 m
Static condition - $\bar{B}$= 1
USA peak

Color	Name	Model	Unit Weight (kN/m³)	Phi' (°)	Minimum Strength (kPa)	Tau/Sigma Ratio
	01c - Clayey till - USA peak_sumin=120kPa	SHANSEP	21		120	0,22
	02 - Bedrock	Bedrock (Impenetrable)				
	03a - Waste rock	Mohr-Coulomb	21	35		

Type 3 failure

FS = 1.23



Date : February 2020
Project: 18108591-4000

Prepared by: EN
Reviewed by: SB

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TOUQUOY MINE WASTE ROCK STORAGE FACILITY

FIGURE D-12

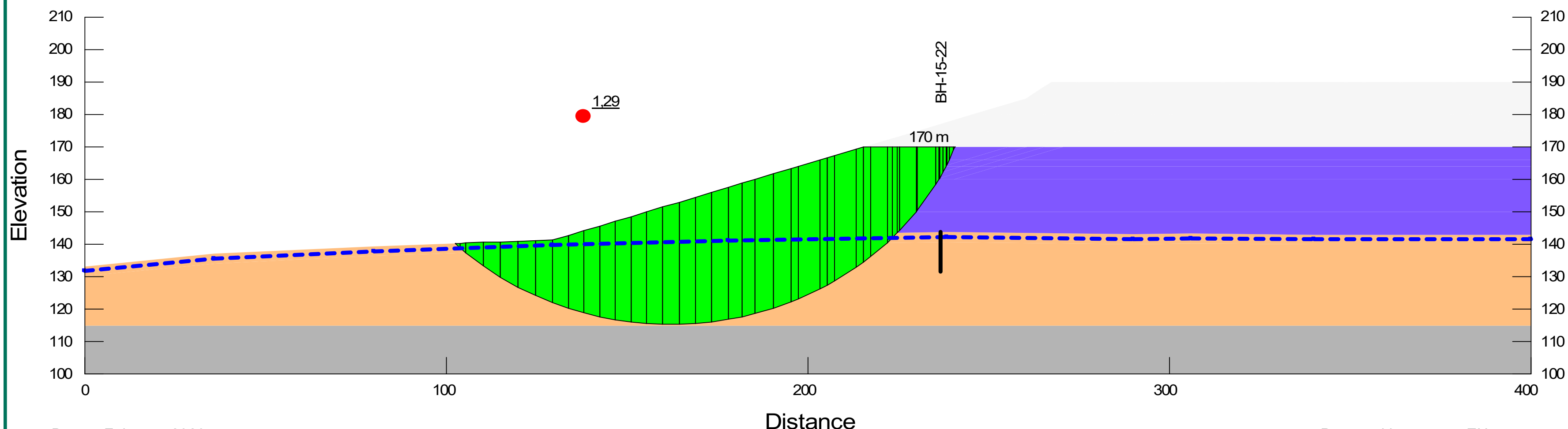
Section 1-1' - 170 m
Pseudo-static condition
USA peak

Color	Name	Model	Unit Weight (kN/m³)	Phi' (°)	Minimum Strength (kPa)	Tau/Sigma Ratio
	01c - Clayey till - USA peak_sumin=120kPa	SHANSEP	21		120	0,22
	02 - Bedrock	Bedrock (Impenetrable)				
	03a - Waste rock	Mohr-Coulomb	21	35		

kh = 0.115

Type 3 failure

FS = 1.29



Date : February 2020
Project: 18108591-4000

Prepared by: EN
Reviewed by: MK

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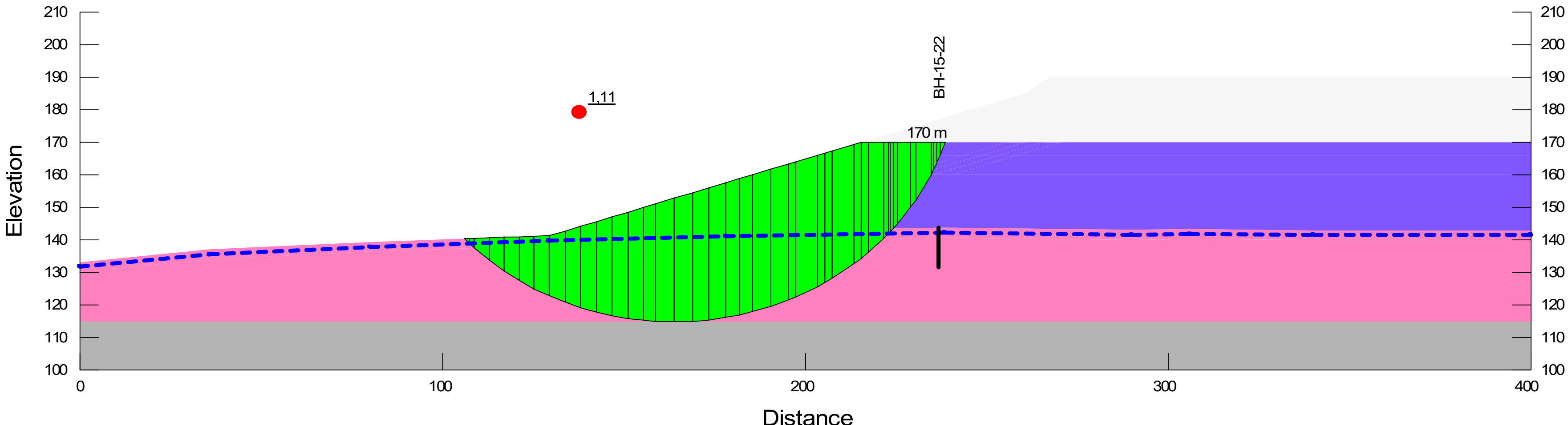
TOUQUOY MINE WASTE ROCK STORAGE FACILITY

FIGURE D-13

Section 1-1' - 170 m
Post-liquefaction condition
USA post-peak

Color	Name	Model	Unit Weight (kN/m³)	Phi' (°)	Minimum Strength (kPa)	Tau/Sigma Ratio
	01g - Clayey till - USA post-peak	SHANSEP	21		96	0,176
	02 - Bedrock	Bedrock (Impenetrable)				
	03a - Waste rock	Mohr-Coulomb	21	35		

Type 3 failure **FS = 1.11**



Date : February 2020
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Reviewed by: MK

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TOUQUOY MINE WASTE ROCK STORAGE FACILITY

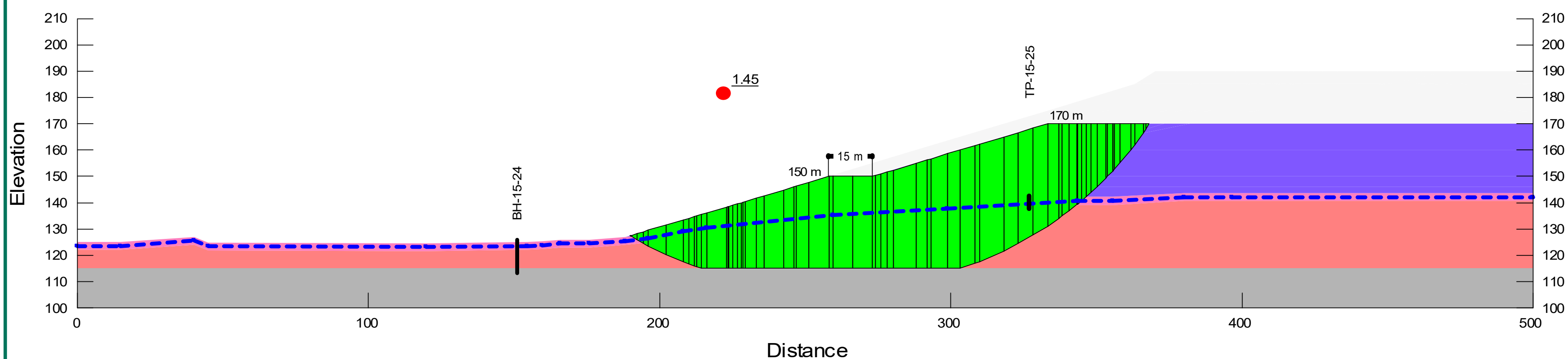
FIGURE D-14

Section 2-2' - 170 m - 15 m bench at 150 m
Static condition
USA peak

Color	Name	Model	Unit Weight (kN/m³)	Phi' (°)	Minimum Strength (kPa)	Tau/Sigma Ratio
	01b - Clayey till - USA peak_sumin=50 kPa	SHANSEP	21		50	0.22
	01e - Clayey till - USA post-peak_sumin=170 kPa	SHANSEP	21		170	0.22
	02 - Bedrock	Bedrock (Impenetrable)				
	03a - Waste rock	Mohr-Coulomb	21	35		

Type 3 failure

FS = 1.45



Date : March 2020
Project: 18108591-4000

Prepared by: EN
Reviewed by: MK

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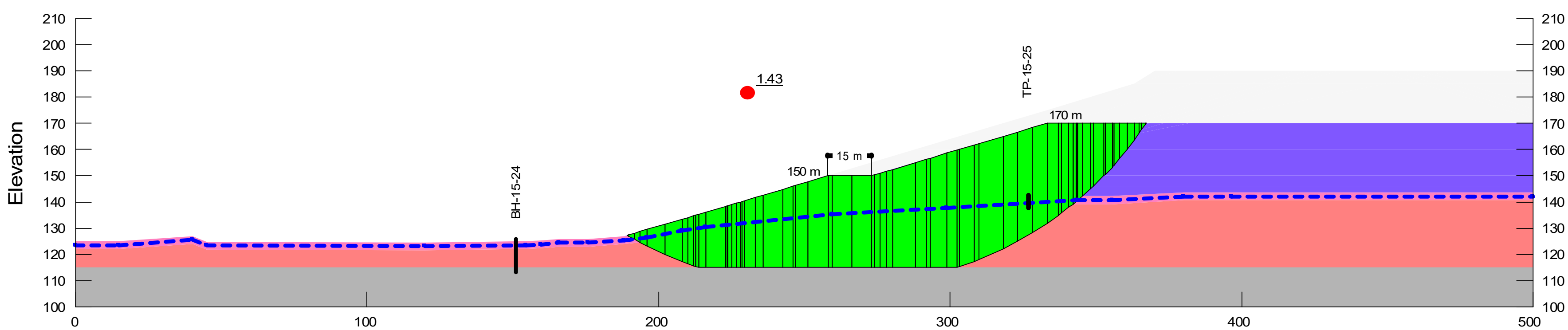
TOUQUOY MINE WASTE ROCK STORAGE FACILITY

FIGURE D-15

Section 2-2' - 170 m - 15 m bench at 150 m
Static condition - $\bar{B}$= 1
USA peak

Color	Name	Model	Unit Weight (kN/m³)	Phi' (°)	Minimum Strength (kPa)	Tau/Sigma Ratio
	01b - Clayey till - USA peak_sumin=50 kPa	SHANSEP	21		50	0.22
	01e - Clayey till - USA post-peak_sumin=170 kPa	SHANSEP	21		170	0.22
	02 - Bedrock	Bedrock (Impenetrable)				
	03a - Waste rock	Mohr-Coulomb	21	35		

Type 3 failure **FS = 1.43**



Date : March 2020
Project: 18108591-4000

Prepared by: EN
Reviewed by: MK

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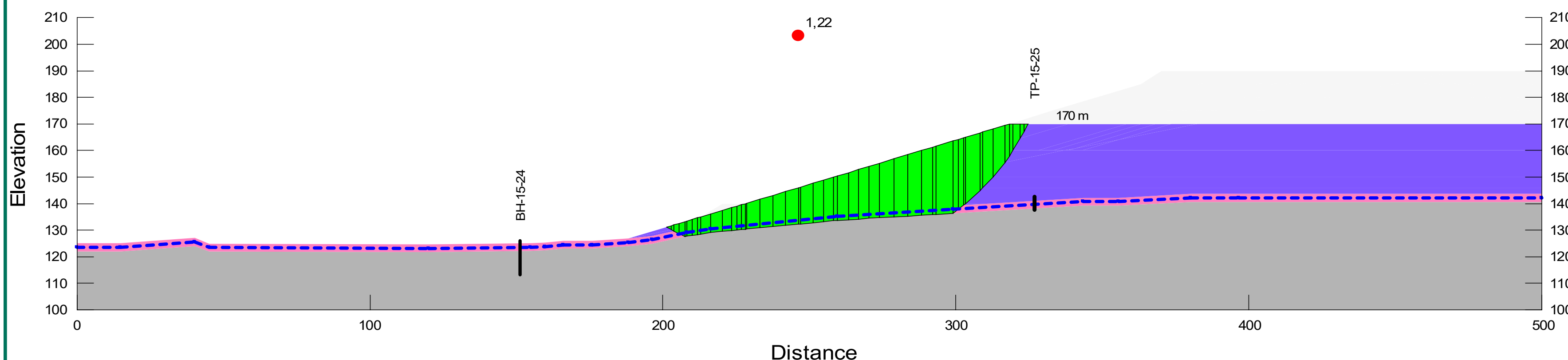
TOUQUOY MINE WASTE ROCK STORAGE FACILITY

FIGURE D-16

Section 2-2' - 170 m
Static condition
USA peak

Color	Name	Model	Unit Weight (kN/m^3)	Phi' (°)	Minimum Strength (kPa)	Tau/Sigma Ratio
	01b - Clayey till - USA peak_sumin=50 kPa	SHANSEP	21		50	0,22
	02 - Bedrock	Bedrock (Impenetrable)				
	03a - Waste rock	Mohr-Coulomb	21	35		

Non circular failure **FS = 1.22**



Date : February 2020
Project: 18108591-4000

Prepared by: EN
Reviewed by: MK

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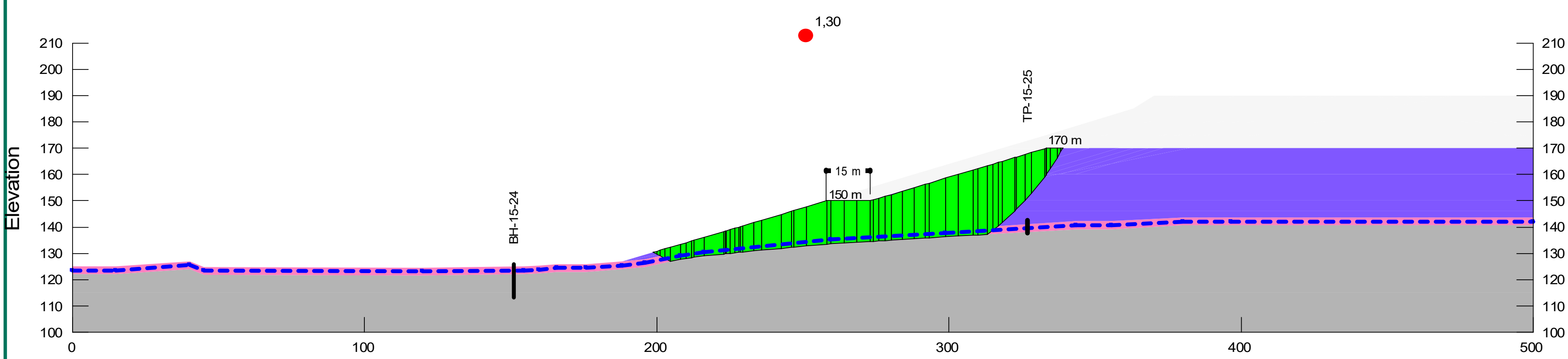
TOUQUOY MINE WASTE ROCK STORAGE FACILITY

FIGURE D-17

Section 2-2' - 170 m - 15 m bench at 150 m
Static condition
USA peak

Color	Name	Model	Unit Weight (kN/m³)	Phi' (°)	Minimum Strength (kPa)	Tau/Sigma Ratio
	01b - Clayey till - USA peak_sumin=50 kPa	SHANSEP	21		50	0,22
	02 - Bedrock	Bedrock (Impenetrable)				
	03a - Waste rock	Mohr-Coulomb	21	35		

Non circular failure **FS = 1.30**



Date : February 2020
Project: 18108591-4000

Prepared by: EN
Reviewed by: MK

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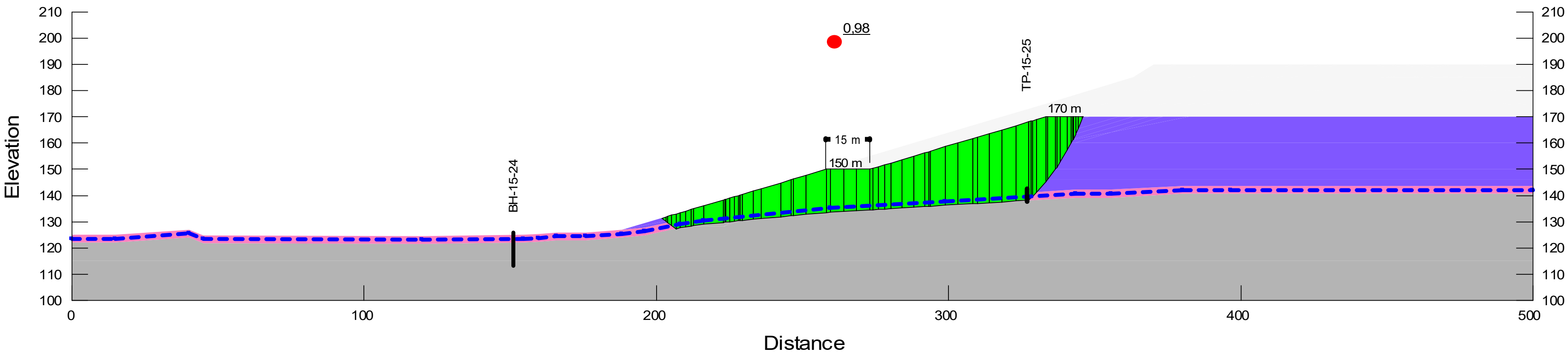
TOUQUOY MINE WASTE ROCK STORAGE FACILITY

FIGURE D-18

Section 2-2' - 170 m - 15 m bench at 150 m
Static condition - $\bar{B}$= 1
USA peak

Color	Name	Model	Unit Weight (kN/m³)	Phi' (°)	Minimum Strength (kPa)	Tau/Sigma Ratio
	01b - Clayey till - USA peak_sumin=50 kPa	SHANSEP	21		50	0,22
	02 - Bedrock	Bedrock (Impenetrable)				
	03a - Waste rock	Mohr-Coulomb	21	35		

Non circular failure **FS = 0.98**



Date : February 2020
Project: 18108591-4000

Prepared by: EN
Reviewed by: MK

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TOUQUOY MINE WASTE ROCK STORAGE FACILITY

FIGURE D-19

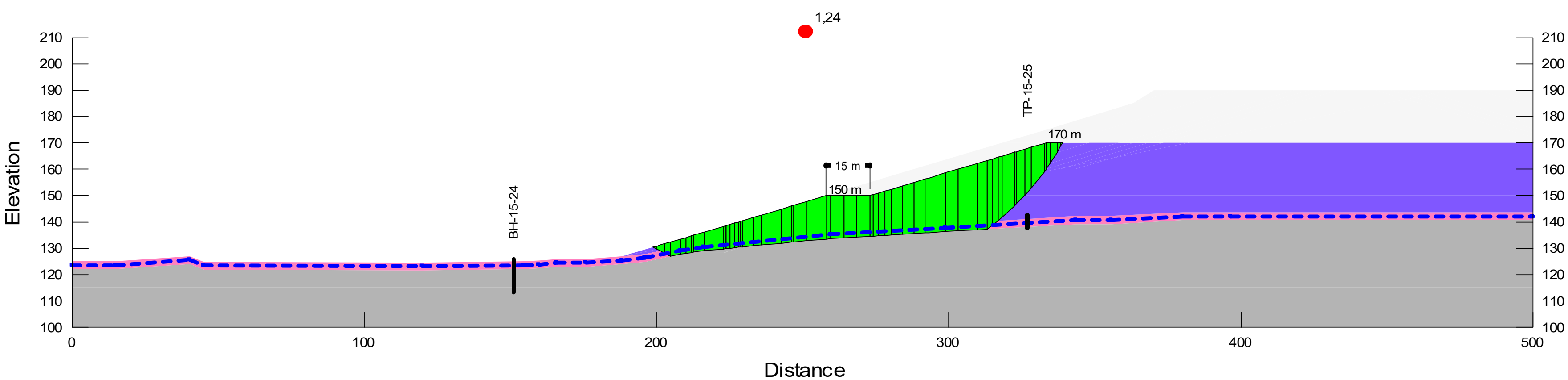
Section 2-2' - 170 m
Pseudo-static condition
USA peak

Color	Name	Model	Unit Weight (kN/m³)	Phi' (°)	Minimum Strength (kPa)	Tau/Sigma Ratio
	01b - Clayey till - USA peak_sumin=50 kPa	SHANSEP	21		50	0,22
	02 - Bedrock	Bedrock (Impenetrable)				
	03a - Waste rock	Mohr-Coulomb	21	35		

kh = 0.0115

Non circular failure

FS = 1.24



Date : February 2020
Project: 18108591-4000

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Reviewed by: MK

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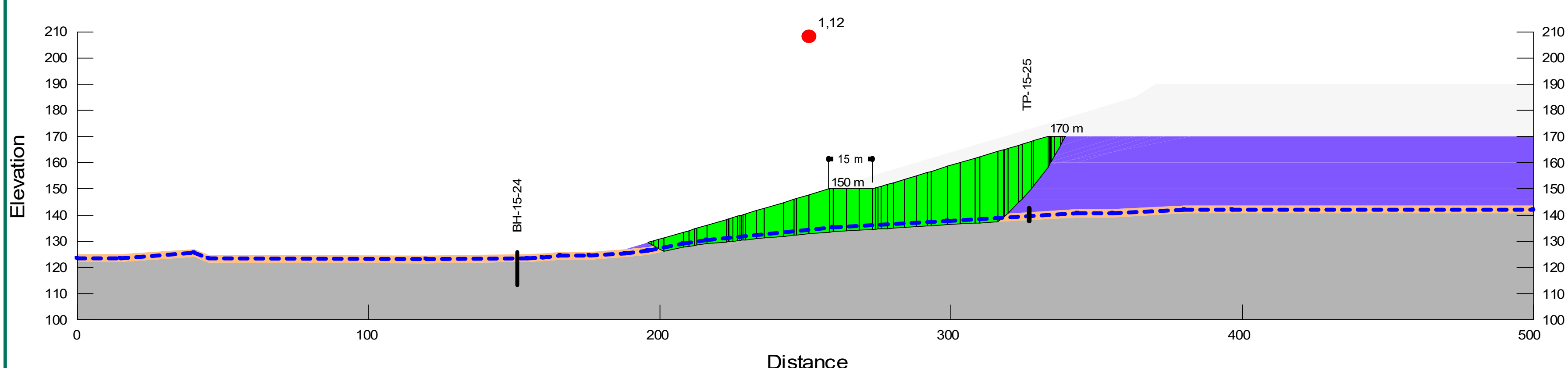
TOUQUOY MINE WASTE ROCK STORAGE FACILITY

FIGURE D-20

Section 2-2' - 170 m
Post-liquefaction condition
USA post-peak

Color	Name	Model	Unit Weight (kN/m³)	Phi' (°)	Minimum Strength (kPa)	Tau/Sigma Ratio
	01g - Clayey till - USA post-peak_sumin=40 kPa	SHANSEP	21		40	0,176
	02 - Bedrock	Bedrock (Impenetrable)				
	03a - Waste rock	Mohr-Coulomb	21	35		

Non circular failure **FS = 1.12**



Date : February 2020
Project: 18108591-4000

Prepared by: EN
Reviewed by: MK

Golder Associés Ltée

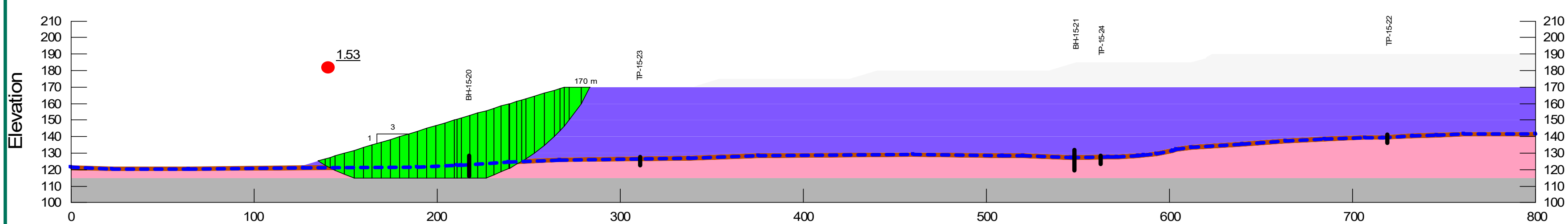
STABILITY ANALYSES
TOUQUOY MINE WASTE ROCK STORAGE FACILITY

FIGURE D-21

Section 3-3' - 170 m
Static condition
USA peak

Color	Name	Model	Unit Weight (kN/m³)	Phi' (°)	Minimum Strength (kPa)	Tau/Sigma Ratio
	01b - Clayey Till USA - sumin=50 kPa	SHANSEP	21		50	0.22
	01c - Clayey Till USA - sumin=170 kPa	SHANSEP	21		170	0.22
	02 - Bedrock	Bedrock (Impenetrable)				
	03a - Waste rock	Mohr-Coulomb	21	35		

Type 3 failure **FS = 1.53**



Date: March 2020
Project: 18108591-4000

Prepared by: EN
Reviewed by: MK

Golder Associés Ltée

STABILITY ANALYSES
TOUQUOY MINE WASTE ROCK STORAGE FACILITY

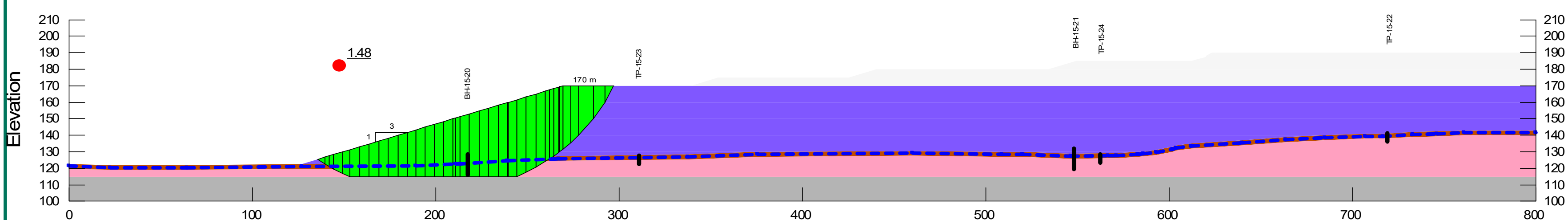
FIGURE D-22

Section 3-3' - 170 m
Static condition - $\bar{B}$= 1
USA peak

Color	Name	Model	Unit Weight (kN/m³)	Phi' (°)	Minimum Strength (kPa)	Tau/Sigma Ratio
	01b - Clayey Till USA - sumin = 50kPa	SHANSEP	21		50	0.22
	01c- ClayeyTill USA - sumin=170 kPa	SHANSEP	21		170	0.22
	02 - Bedrock	Bedrock (Impenetrable)				
	03a - Waste rock	Mohr-Coulomb	21	35		

Type 3 failure

FS = 1.48



Date: March 2020
Project: 18108591-4000

Prepared by: EN
Reviewed by: MK

Golder Associés Ltée

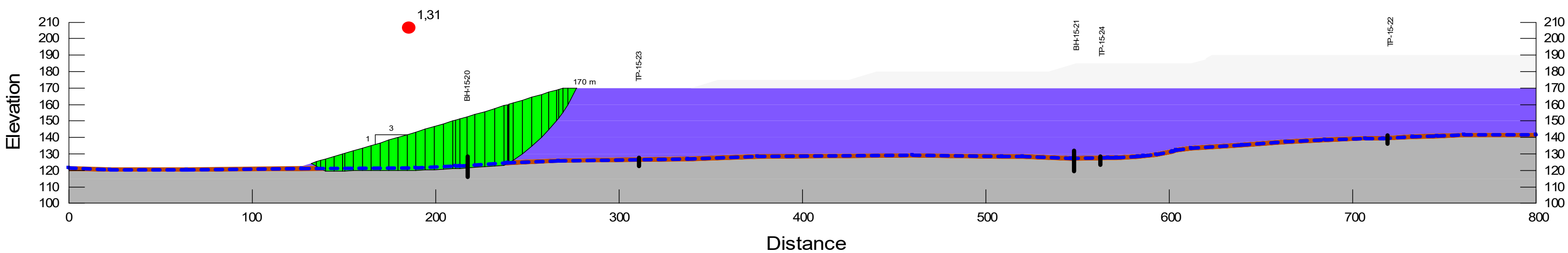
STABILITY ANALYSES
TOUQUOY MINE WASTE ROCK STORAGE FACILITY

FIGURE D-23

Section 3-3' - 170 m
Static condition
USA peak

Color	Name	Model	Unit Weight (kN/m³)	Phi' (°)	Minimum Strength (kPa)	Tau/Sigma Ratio
	01b - Clayey Till USA - sumin=50 kPa	SHANSEP	21		50	0,22
	02 - Bedrock	Bedrock (Impenetrable)				
	03a - Waste rock	Mohr-Coulomb	21	35		

Non circular failure **FS = 1.31**



Date: March 2020
Project: 18108591-4000

Prepared by: EN
Reviewed by: MK

Golder Associés Ltée

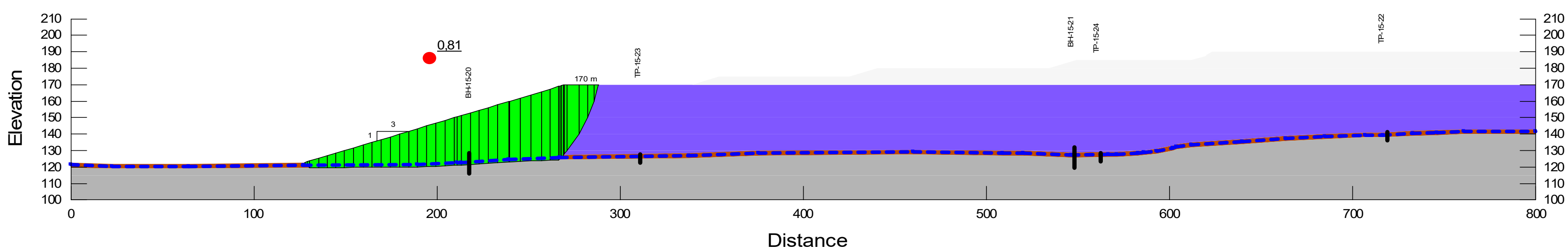
STABILITY ANALYSES
TOUQUOY MINE WASTE ROCK STORAGE FACILITY

FIGURE D-24

Section 3-3' - 170 m
Static condition - $\overline{B}$= 1
USA peak

Color	Name	Model	Unit Weight (kN/m³)	Phi' (°)	Minimum Strength (kPa)	Tau/Sigma Ratio
	01b - Clayey Till USA - sumin=50 kPa	SHANSEP	21		50	0,22
	02 - Bedrock	Bedrock (Impenetrable)				
	03a - Waste rock	Mohr-Coulomb	21	35		

Non circular failure **FS = 0.81**



Date : March 2020
Project: 18108591-4000

Prepared by: EN
Reviewed by: MK

Golder Associés Ltée

STABILITY ANALYSES
TOUQUOY MINE WASTE ROCK STORAGE FACILITY

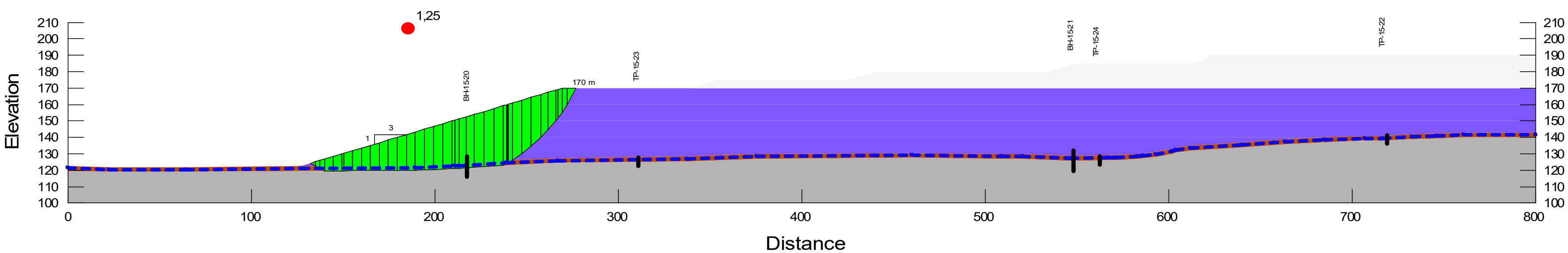
FIGURE D-25

Section 3-3' - 170 m
Pseudo-static condition
USA peak

Color	Name	Model	Unit Weight (kN/m³)	Phi' (°)	Minimum Strength (kPa)	Tau/Sigma Ratio
(orange)	01b - Clayey Till USA - sumin=50 kPa	SHANSEP	21		50	0,22
(grey)	02 - Bedrock	Bedrock (Impenetrable)				
(purple)	03a - Waste rock	Mohr-Coulomb	21	35		

kh = 0.0115

Non circular failure **FS = 1.25**



Date : March 2020
Project: 18108591-4000

Prepared by: EN
Reviewed by: MK

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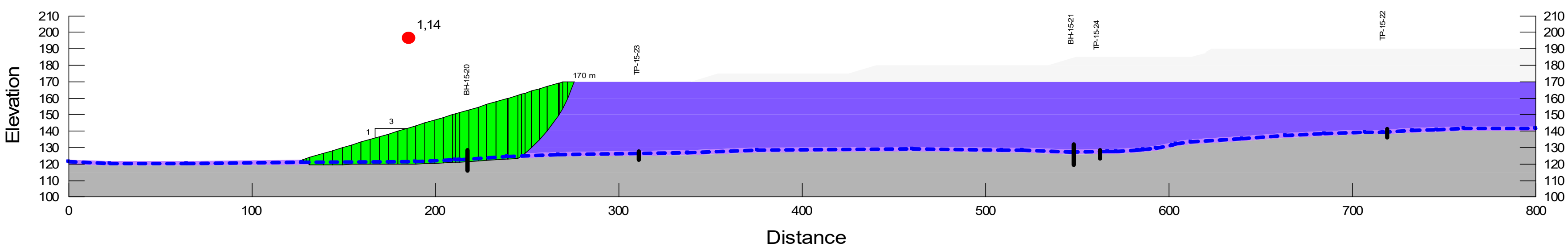
STABILITY ANALYSES
TOUQUOY MINE WASTE ROCK STORAGE FACILITY

FIGURE D-26

Section 3-3' - 170 m
Post-liquefactiion condition
USA post-peak

Color	Name	Model	Unit Weight (kN/m³)	Phi' (°)	Minimum Strength (kPa)	Tau/Sigma Ratio
	01e - Clayey Till USA - post-peak - sumin=40 kPa	SHANSEP	21		40	0,176
	02 - Bedrock	Bedrock (Impenetrable)				
	03a - Waste rock	Mohr-Coulomb	21	35		

Non circular failure **FS = 1.14**



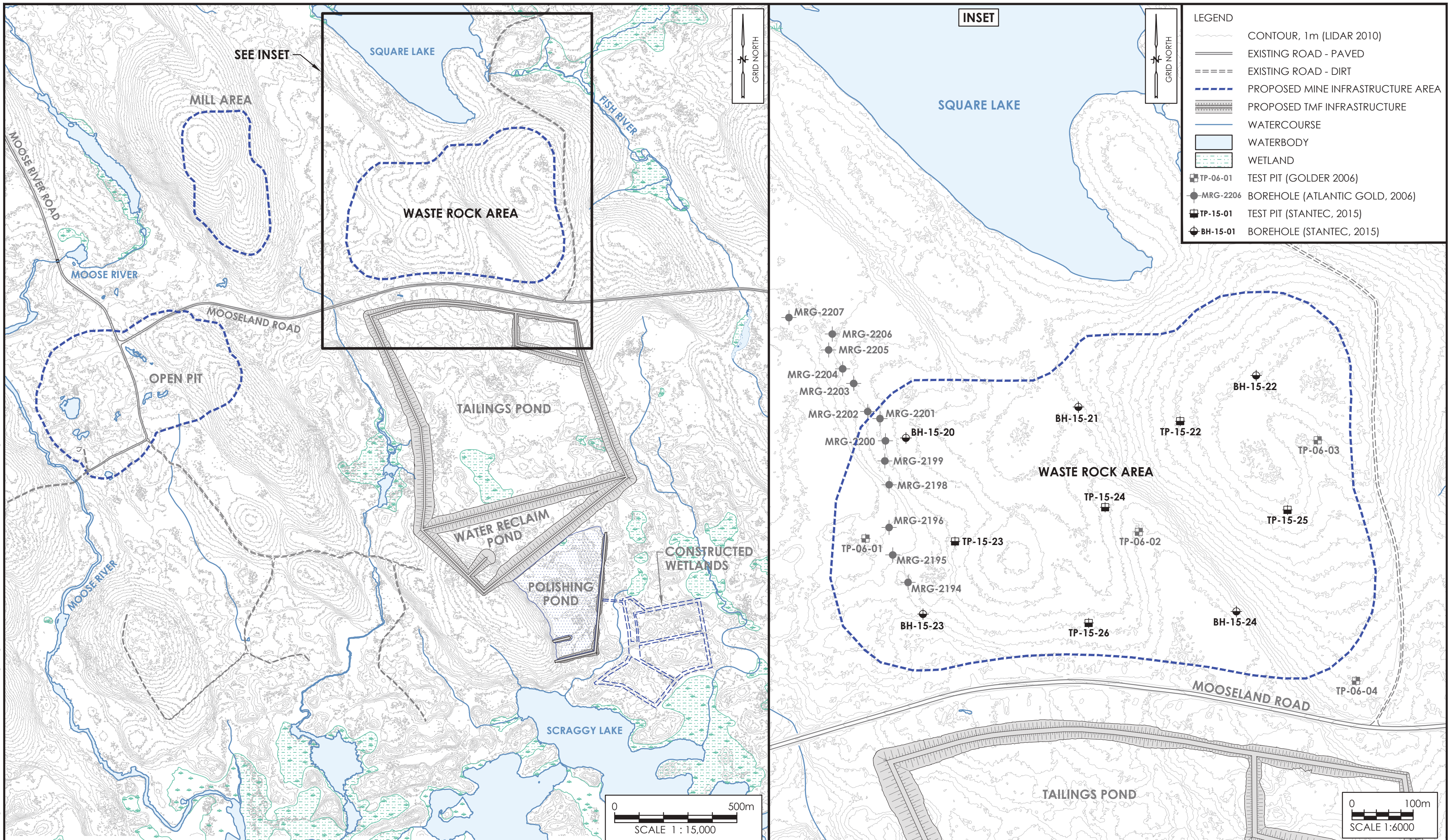
Date : March 2020
Project: 18108591-4000

Prepared by: EN
Reviewed by: MK

Golder Associés Ltée

APPENDIX E

Borehole and Test Pit Locations and Records



THIS DRAWING ILLUSTRATES SUPPORTING INFORMATION SPECIFIC TO A STANTEC CONSULTING LTD. REPORT AND MUST NOT BE USED FOR OTHER PURPOSES.

Reference:

1. CONESTOGA-ROVERS & ASSOCIATES; PROJECT No. 820933-E; DRAWING No. 02; DATE: AUGUST 2013.
2. GOLDER & ASSOCIATES: PROJECT No. 06-1118-041C; FIGURE No. 2; DATE: MAY, 2015.

WASTE ROCK AREA

TOUQUOY MINE TAILINGS MANAGEMENT FACILITY
HALIFAX COUNTY, NOVA SCOTIA

Client: DDV GOLD LTD.

Job No.:	121619250
Scale:	AS SHOWN
Date:	2016 01 18
Dwn. By:	JL
App'd By:	BL
Dwg. No.:	2

Stantec

V:\1216\active\121619250\1_geotechnical_deliverables\3_drawings_logs\600_TOUQUOY\SHEET FILES\19250C-WTRK-001.dwg

BOREHOLE RECORD

BH-15-20

Stantec

CLIENT **DDV Gold Ltd.**

LOCATION **Touquoy Mine Tailings Management Facility Halifax County, Nova Scotia**

DATES: BORING **2015-12-02**

WATER LEVEL **Not Encountered**

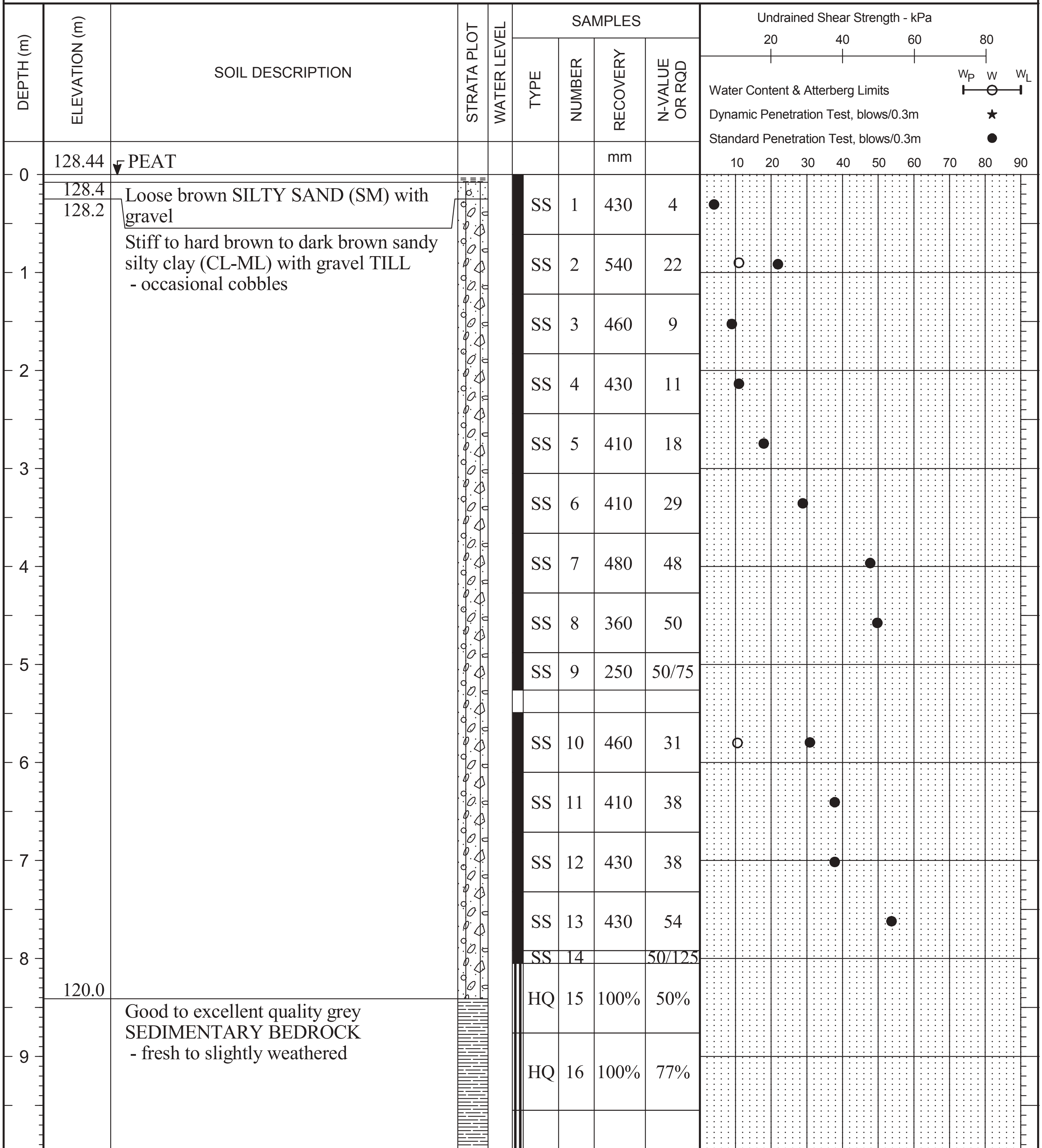
PROJECT No. **121619250**

BOREHOLE No. **BH-15-20**

DATUM **Geodetic**

ELEVATION (m)	SOIL DESCRIPTION
128.44	PEAT
128.4	Loose brown SILTY SAND (SM) with gravel
128.2	Stiff to hard brown to dark brown sandy silty clay (CL-ML) with gravel TILL - occasional cobbles
120.0	Good to excellent quality grey SEDIMENTARY BEDROCK - fresh to slightly weathered

TYPE	NUMBER	RECOVERY (mm)	N-VALUE OR RQD
SS	1	430	4
SS	2	540	22
SS	3	460	9
SS	4	430	11
SS	5	410	18
SS	6	410	29
SS	7	480	48
SS	8	360	50
SS	9	250	50/75
SS	10	460	31
SS	11	410	38
SS	12	430	38
SS	13	430	54
SS	14		50/125
HQ	15	100%	50%
HQ	16	100%	77%



Δ Unconfined Compression Test

☐ Field Vane Test ■ Remoulded

✖ Torvane

Continued Next Page

BOREHOLE RECORD

BH-15-20

Stantec

CLIENT: DDV Gold Ltd.
PROJECT No.: 121619250
LOCATION: Touquoy Mine Tailings Management Facility Halifax County, Nova Scotia
BOREHOLE No.: BH-15-20
DATES: BORING: 2015-12-02
WATER LEVEL: Not Encountered
DATUM: Geodetic

DEPTH (m)	ELEVATION (m)	SOIL DESCRIPTION	STRATA PLOT	WATER LEVEL	SAMPLES TYPE	NUMBER	RECOVERY (mm)	N-VALUE OR RQD
10		CONT'D Good to excellent quality grey SEDIMENTARY BEDROCK - fresh to slightly weathered			HQ	17	100%	87%
11					HQ	18	100%	87%
12	116.0							
		End of borehole						
13								
14								
15								
16								
17								
18								
19								
20								

Undrained Shear Strength - kPa: 20 40 60 80

Water Content & Atterberg Limits W_P W W_L

Dynamic Penetration Test, blows/0.3m ★

Standard Penetration Test, blows/0.3m ●

10 20 30 40 50 60 70 80 90

△ Unconfined Compression Test

□ Field Vane Test ■ Remoulded

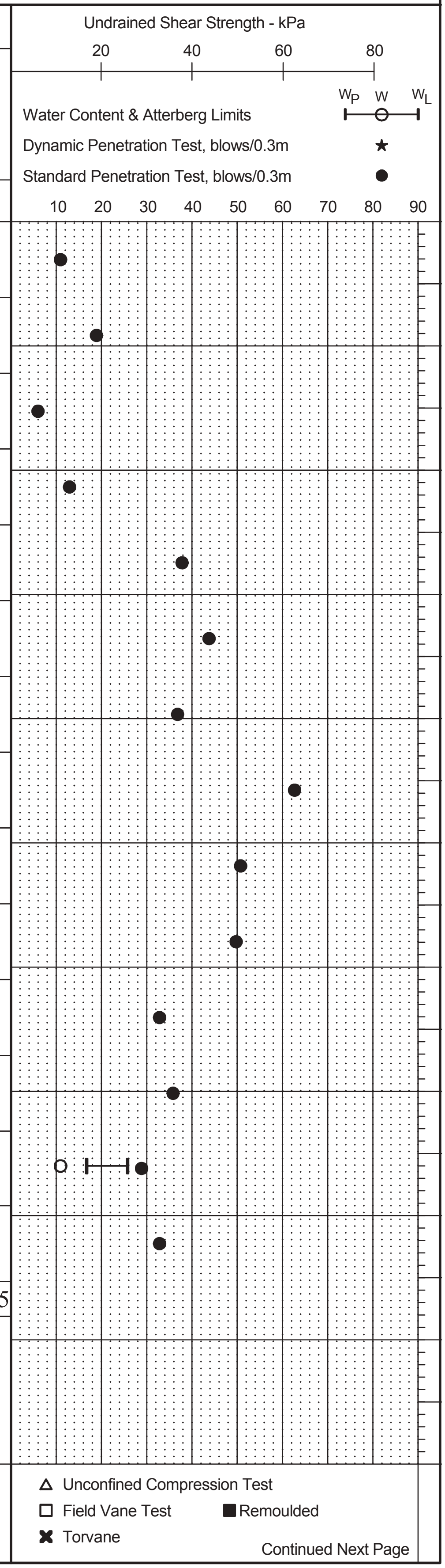
✖ Torvane

BOREHOLE RECORD

BH-15-21

CLIENT: **DDV Gold Ltd.**
PROJECT No.: **121619250**
LOCATION: **Touquoy Mine Tailings Management Facility Halifax County, Nova Scotia**
BOREHOLE No.: **BH-15-21**
DATES: BORING: **2015-12-01**
WATER LEVEL: **Not Encountered**
DATUM: **Geodetic**

DEPTH (m)	ELEVATION (m)	SOIL DESCRIPTION	SAMPLES: TYPE	NUMBER	RECOVERY (mm)	N-VALUE OR RQD
0	132.00	PEAT				
	131.9	Very stiff to hard brown sandy silty clay (CL-ML) with gravel TILL	SS	1	330	11
1			SS	2	430	19
			SS	3	380	6
2	129.9	Stiff to hard dark brown lean clay (CL) with sand TILL	SS	4	460	13
			SS	5	380	38
3			SS	6	310	44
4			SS	7	N/A	37
			SS	8	410	63
5			SS	9	460	51
6			SS	10	330	50
			SS	11	380	33
7			SS	12	150	36
			SS	13	410	29
8			SS	14	150	33
			SS	15	180	50/125
9	123.2	Good to excellent quality grey SEDIMENTARY BEDROCK - fresh to slightly weathered	HQ	16	98%	78%
10						



Continued Next Page

BOREHOLE RECORD

BH-15-21

Stantec

CLIENT DDV Gold Ltd.

LOCATION Touquoy Mine Tailings Management Facility Halifax County, Nova Scotia

DATES: BORING 2015-12-01

WATER LEVEL Not Encountered

PROJECT No. 121619250

BOREHOLE No. BH-15-21

DATUM Geodetic

DEPTH (m)	ELEVATION (m)	SOIL DESCRIPTION	STRATA PLOT	WATER LEVEL	SAMPLES TYPE	NUMBER	RECOVERY (mm)	N-VALUE OR RQD
10		CONT'D Good to excellent quality grey SEDIMENTARY BEDROCK - fresh to slightly weathered						
11					HQ	17		84%
12	119.4				HQ	18	97%	81%
		End of borehole						

Undrained Shear Strength - kPa: 20, 40, 60, 80

Water Content & Atterberg Limits: W_P W W_L

Dynamic Penetration Test, blows/0.3m ★

Standard Penetration Test, blows/0.3m ●

10 20 30 40 50 60 70 80 90

Depth (m): 10, 11, 12, 13, 14, 15, 16, 17, 18, 19, 20

△ Unconfined Compression Test

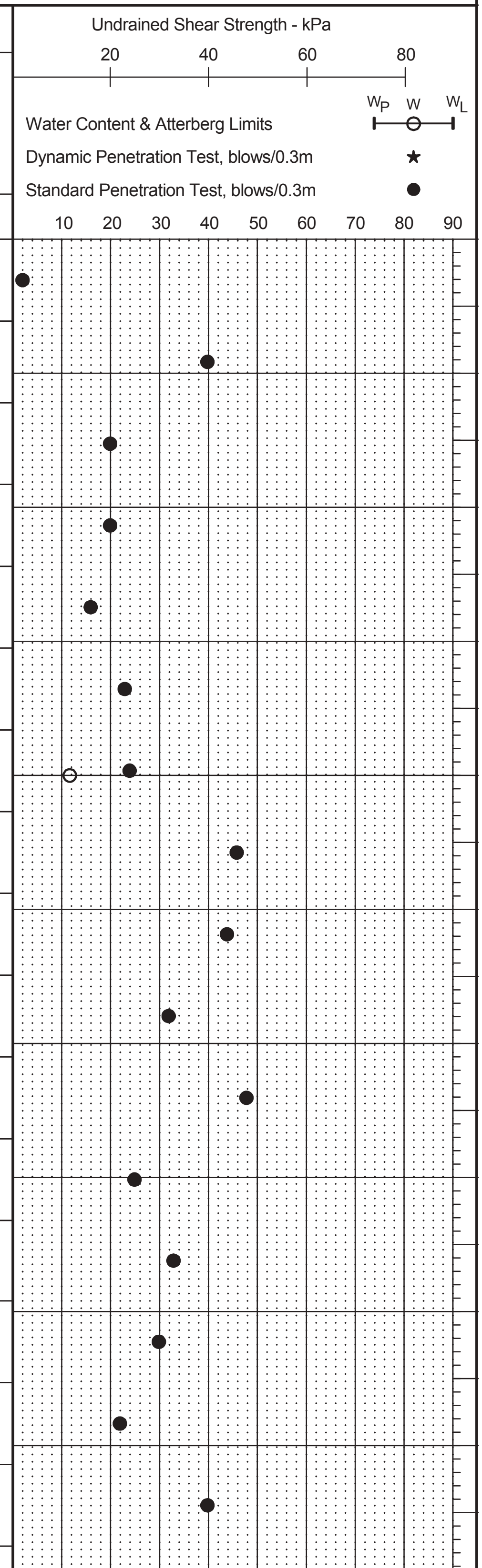
□ Field Vane Test ■ Remoulded

✖ Torvane

Stantec BOREHOLE RECORD BH-15-22

CLIENT: DDV Gold Ltd.
LOCATION: Touquoy Mine Tailings Management Facility Halifax County, Nova Scotia
DATES: BORING 2015-11-30
WATER LEVEL: Not Encountered
PROJECT No. 121619250
BOREHOLE No. BH-15-22
DATUM: Geodetic

DEPTH (m)	ELEVATION (m)	SOIL DESCRIPTION	STRATA PLOT	WATER LEVEL	SAMPLES TYPE	NUMBER	RECOVERY (mm)	N-VALUE OR RQD
0	143.79	PEAT						
	143.7	Very stiff to hard brown sandy silty clay (CL-ML) with gravel TILL			SS	1	150	2
1					SS	2	360	40
					SS	3	430	20
2	141.7				SS	4	360	20
		Stiff to hard dark brown lean clay (CL) with sand TILL			SS	5	310	16
3					SS	6	460	23
4					SS	7	560	24
					SS	8	360	46
5					SS	9	510	44
					SS	10	250	32
6					SS	11	310	48
7					SS	12	430	25
					SS	13	N/A	33
8					SS	14	330	30
9					SS	15	460	22
					SS	16	430	40
10								



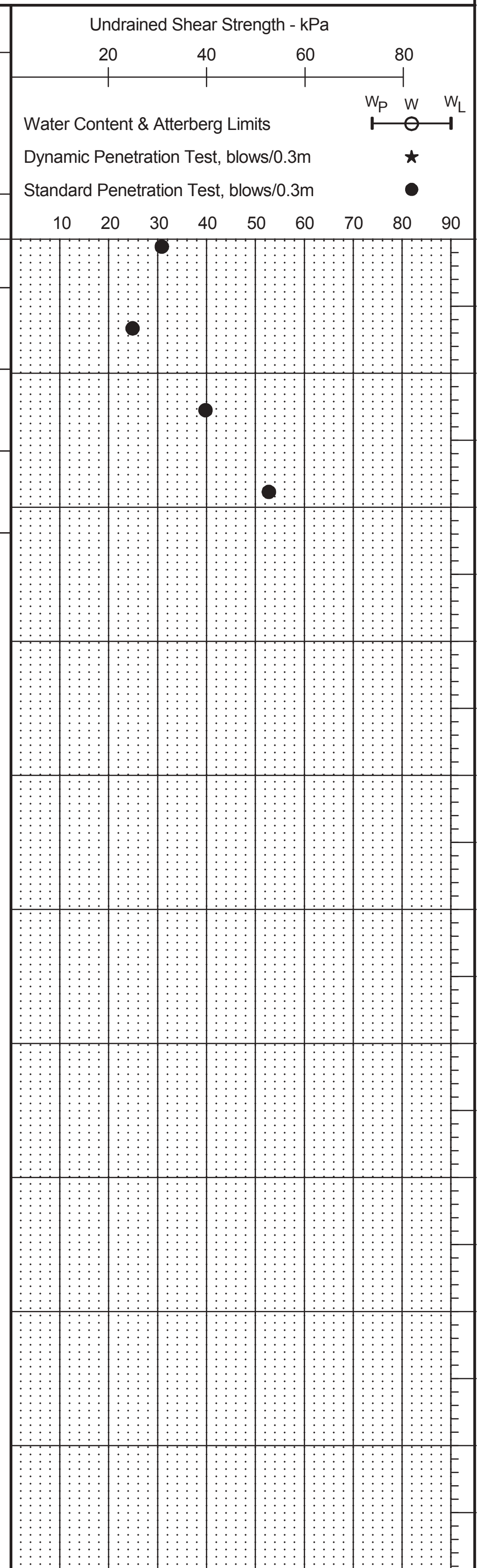
△ Unconfined Compression Test
□ Field Vane Test ■ Remoulded
✖ Torvane

Continued Next Page

Stantec BOREHOLE RECORD BH-15-22

CLIENT: DDV Gold Ltd.
LOCATION: Touquoy Mine Tailings Management Facility Halifax County, Nova Scotia
DATES: BORING 2015-11-30
WATER LEVEL: Not Encountered
PROJECT No. 121619250
BOREHOLE No. BH-15-22
DATUM: Geodetic

DEPTH (m)	ELEVATION (m)	SOIL DESCRIPTION	STRATA PLOT	WATER LEVEL	SAMPLES TYPE	NUMBER	RECOVERY (mm)	N-VALUE OR RQD
10		CONT'D Stiff to hard dark brown lean clay (CL) with sand TILL			SS	17	230	31
					SS	18	280	25
11					SS	19	310	40
12	131.6				SS	20	330	53
		End of borehole						



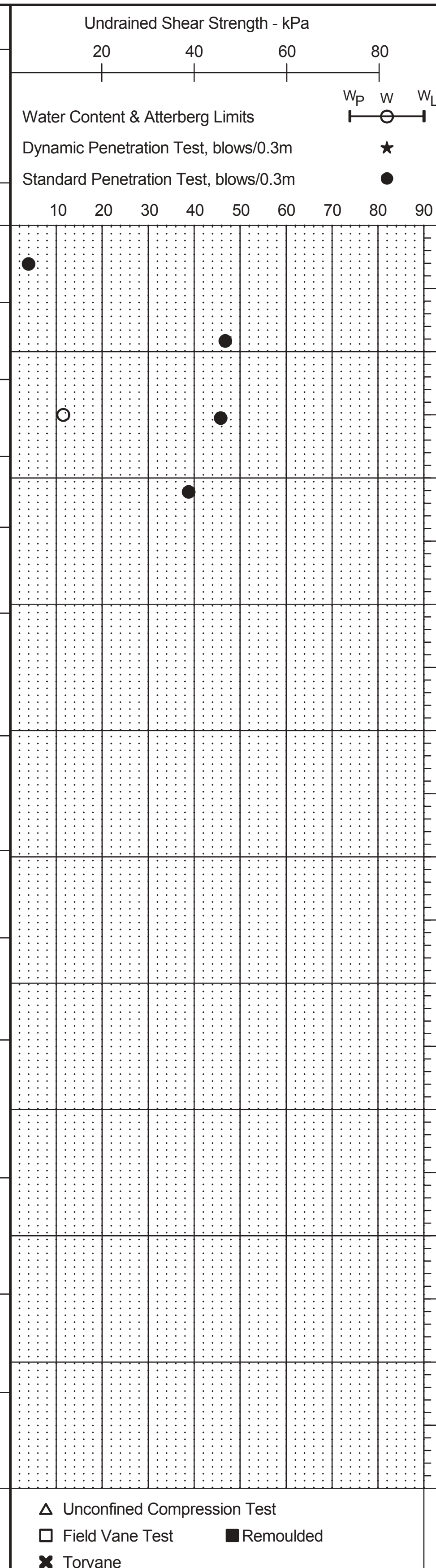
Unconfined Compression Test
Field Vane Test
Remoulded
Torvane

BOREHOLE RECORD

BH-15-23

CLIENT: DDV Gold Ltd.
LOCATION: Touquoy Mine Tailings Management Facility Halifax County, Nova Scotia
DATES: BORING 2015-11-03
WATER LEVEL: Not Encountered
PROJECT No.: 121619250
BOREHOLE No.: BH-15-23
DATUM: Geodetic

DEPTH (m)	ELEVATION (m)	SOIL DESCRIPTION	TYPE	NUMBER	RECOVERY (mm)	N-VALUE OR RQD
0	125.07	PEAT				
	124.8	Dense brown silty sand (SM) with gravel TILL - occasional cobbles	SS	1	330	4
			SS	2	430	47
			SS	3	460	46
			SS	4	360	39
	122.7	Very poor quality grey SEDIMENTARY BEDROCK - slightly to moderately weathered	HQ	5	100%	0%
			HQ	6	95%	0%
			HQ	7	94%	0%
			HQ	8	100%	15%
			HQ	9	91%	0%
	118.6	Poor to fair quality grey SEDIMENTARY BEDROCK - fresh to slightly weathered	HQ	10	98%	59%
			HQ	11	92%	44%
			HQ	12	100%	32%
10			HQ	13	100%	35%



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BOREHOLE RECORD

BH-15-23

CLIENT: DDV Gold Ltd.

LOCATION: Touquoy Mine Tailings Management Facility Halifax County, Nova Scotia

DATES: BORING 2015-11-03

WATER LEVEL: Not Encountered

PROJECT No. 121619250

BOREHOLE No. BH-15-23

DATUM: Geodetic

DEPTH (m)	ELEVATION (m)	SOIL DESCRIPTION	STRATA PLOT	WATER LEVEL	SAMPLES: TYPE	NUMBER	RECOVERY (mm)	N-VALUE OR RQD
10		CONT'D Poor to fair quality grey SEDIMENTARY BEDROCK - fresh to slightly weathered						
11					HQ	14	100%	68%
12					HQ	15	98%	50%
	112.7	End of borehole						
13								
14								
15								
16								
17								
18								
19								
20								

Undrained Shear Strength - kPa: 20, 40, 60, 80

Water Content & Atterberg Limits: W_P W W_L

Dynamic Penetration Test, blows/0.3m ★

Standard Penetration Test, blows/0.3m ●

10 20 30 40 50 60 70 80 90

△ Unconfined Compression Test

□ Field Vane Test ■ Remoulded

✖ Torvane

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BOREHOLE RECORD

BH-15-24

CLIENT: DDV Gold Ltd.

LOCATION: Touquoy Mine Tailings Management Facility Halifax County, Nova Scotia

DATES: BORING 2015-12-01

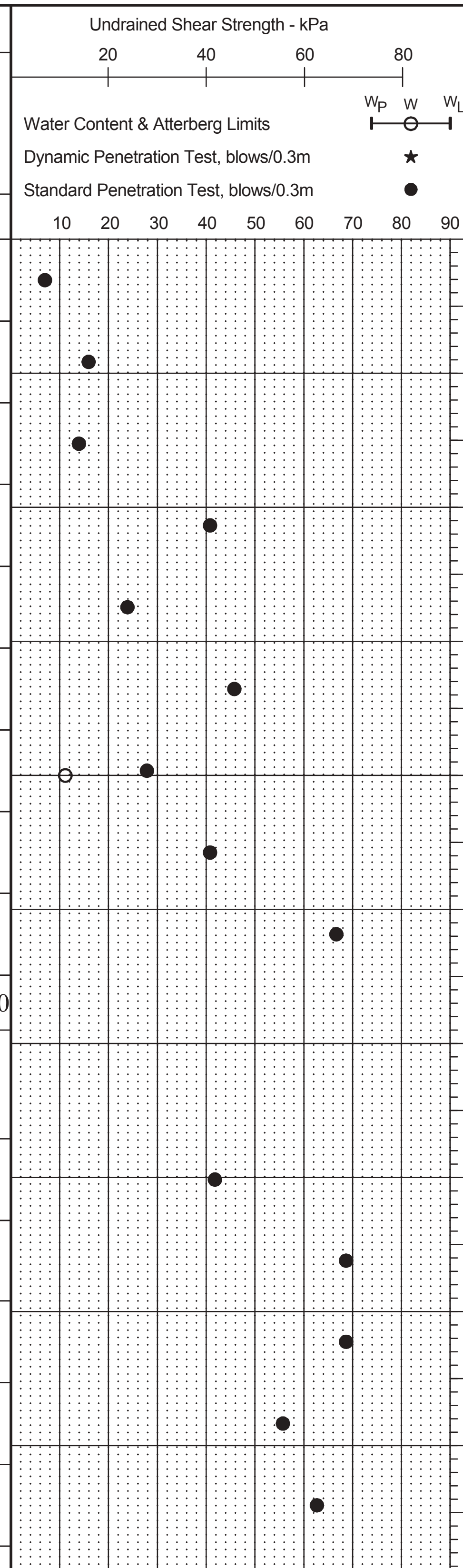
WATER LEVEL: Not Encountered

PROJECT No. 121619250

BOREHOLE No. BH-15-24

DATUM: Geodetic

DEPTH (m)	ELEVATION (m)	SOIL DESCRIPTION	TYPE	NUMBER	RECOVERY (mm)	N-VALUE OR RQD
0	125.99	PEAT				
	125.9	Hard brown sandy silty clay (CL-ML) with gravel TILL	SS	1	380	7
1	124.9		SS	2	410	16
		Stiff to hard dark brown sandy lean clay (CL) to sandy lean clay (CL) with gravel TILL	SS	3	N/A	14
2			SS	4	380	41
			SS	5	250	24
3			SS	6	360	46
4			SS	7	460	28
			SS	8	410	41
5			SS	9	360	67
			SS	10	250	50/100
7			SS	11	480	42
			SS	12	330	69
8			SS	13	360	69
9			SS	14	560	56
			SS	15	100	63
10						



△ Unconfined Compression Test

□ Field Vane Test ■ Remoulded

✖ Torvane

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BOREHOLE RECORD

BH-15-24

CLIENT DDV Gold Ltd.

LOCATION Touquoy Mine Tailings Management Facility Halifax County, Nova Scotia

DATES: BORING 2015-12-01

WATER LEVEL Not Encountered

PROJECT No. 121619250

BOREHOLE No. BH-15-24

DATUM Geodetic

DEPTH (m)	ELEVATION (m)	SOIL DESCRIPTION	STRATA PLOT	WATER LEVEL	SAMPLES TYPE	SAMPLES NUMBER	SAMPLES RECOVERY (mm)	SAMPLES N-VALUE OR RQD
10					SS	16	480	30
	115.4				SS	17	200	50/100
11		Poor quality grey SEDIMENTARY BEDROCK - slightly to highly weathered			HQ	18	88%	0%
	114.4				HQ	19	100%	0%
12		Fair quality grey SEDIMENTARY BEDROCK - slightly weathered			HQ	20	98%	68%
	113.3	End of borehole						
13								
14								
15								
16								
17								
18								
19								
20								

Undrained Shear Strength - kPa: 20, 40, 60, 80

Water Content & Atterberg Limits W_P W W_L

Dynamic Penetration Test, blows/0.3m ★

Standard Penetration Test, blows/0.3m ●

10 20 30 40 50 60 70 80 90

△ Unconfined Compression Test

□ Field Vane Test ■ Remoulded

✖ Torvane

TEST PIT RECORD

TP-15-22

CLIENT: DDV Gold Ltd.
LOCATION: Touquoy Mine Tailings Management Facility Halifax County, Nova Scotia
DATES: DUG 2015-12-15
WATER LEVEL 4.3 m on 2015-12-15

PROJECT No. 121619250
TEST PIT No. TP-15-22
DATUM Geodetic

DEPTH (m)	ELEVATION (m)	SOIL DESCRIPTION	STRATA PLOT	WATER LEVEL	TYPE	NUMBER	REMARKS
0	141.00						
	140.9	ROOTMAT/TOPSOIL					
		Loose to compact, orange-brown SILTY CLAYEY SAND (CL-ML) with gravel - occasional rootlets			SA	1	
	139.5	Very stiff to hard dark brown sandy lean clay (CL) to sandy lean clay (CL) with gravel TILL - occasional cobbles and boulders			SA	2	
3	138.0	Hard grey sandy lean clay (CL) to sandy lean clay (CL) with gravel TILL - occasional cobbles and boulders		▼ (4.3 m)	SA	3	
	136.1	End of Test Pit - Excavator limit reached					

Depth scale: 0, 1, 2, 3, 4, 5

TEST PIT RECORD

TP-15-23

CLIENT: DDV Gold Ltd.
LOCATION: Touquoy Mine Tailings Management Facility Halifax County, Nova Scotia
DATES: DUG 2015-12-15
WATER LEVEL: 1.5 m on 2015-12-15

PROJECT No. 121619250
TEST PIT No. TP-15-23
DATUM: Geodetic

DEPTH (m)	ELEVATION (m)	SOIL DESCRIPTION	STRATA PLOT	WATER LEVEL	TYPE	NUMBER	REMARKS
	127.70						
0	127.5	ROOTMAT/TOPSOIL					
		Compact brown silty sand (SM) with gravel TILL - occasional cobbles and boulders					
1	126.5				SA	1	
		Hard grey sandy lean clay (CL) to sandy lean clay (CL) with gravel TILL - occasional to some cobbles and boulders		▼			
2							
					SA	2	
3							
4							
	122.8	End of Test Pit - Excavator limit reached					
5							

Stantec TEST PIT RECORD TP-15-24

CLIENT: DDV Gold Ltd.
LOCATION: Touquoy Mine Tailings Management Facility Halifax County, Nova Scotia
DATES: DUG 2015-12-15
WATER LEVEL: 1.5 m on 2015-12-15
PROJECT No.: 121619250
TEST PIT No.: TP-15-24
DATUM: Geodetic

DEPTH (m)	ELEVATION (m)	SOIL DESCRIPTION	STRATA PLOT	WATER LEVEL	TYPE	NUMBER	REMARKS
	128.40						
0	128.1	ROOTMAT/TOPSOIL					
	126.0	Compact brown sandy lean clay (CL) to sandy lean clay (CL) with gravel TILL - trace rootlets from 0.3 m to 0.8m below surface - occasional cobbles and boulders		▼ (1.5 m)	SA	1	
	123.5	Hard grey sandy lean clay (CL) with gravel TILL - occasional cobbles			SA	2	
		End of Test Pit - Excavator limit reached					
5							

Stantec

TEST PIT RECORD

TP-15-25

CLIENT: DDV Gold Ltd.
PROJECT No.: 121619250
LOCATION: Touquoy Mine Tailings Management Facility Halifax County, Nova Scotia
TEST PIT No.: TP-15-25
DATES: DUG 2015-12-15
WATER LEVEL: 1.5 m on 2015-12-15
DATUM: Geodetic

DEPTH (m)	ELEVATION (m)	SOIL DESCRIPTION	STRATA PLOT	WATER LEVEL	TYPE	NUMBER	REMARKS
	142.60						
0	142.5	ROOTMAT/TOPSOIL					
	142.3	Loose to compact orange-brown SILTY SAND (SM) - frequent rootlets - trace gravel					
		Very stiff brown sandy lean clay (CL) with gravel TILL - occasional cobbles					
1					SA	1	
	141.1	Hard grey sandy lean clay (CL) with gravel TILL - occasional cobbles		▼			
2							
					SA	2	
3							
4							
	137.7	End of Test Pit - Excavator limit reached					
5							

Stantec

TEST PIT RECORD

TP-15-26

CLIENT: DDV Gold Ltd.
LOCATION: Touquoy Mine Tailings Management Facility Halifax County, Nova Scotia
DATES: DUG 2015-12-15
WATER LEVEL: 1.8 m on 2015-12-15
PROJECT No.: 121619250
TEST PIT No.: TP-15-26
DATUM: Geodetic

DEPTH (m)	ELEVATION (m)	SOIL DESCRIPTION	STRATA PLOT	WATER LEVEL	TYPE	NUMBER	REMARKS
	127.50						
0	127.2	ROOTMAT/TOPSOIL					
	126.9	Loose to compact orange-brown SILTY SAND (SM) with gravel - trace rootlets			SA	1	
1		Stiff brown sandy lean clay (CL) with gravel TILL - occasional cobbles and boulders		▼			
2					SA	2	
3	124.5	Hard grey sandy lean clay (CL) with gravel TILL - occasional cobbles					
4					SA	3	
	122.6	End of Test Pit - Excavator limit reached					
5							

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APPENDIX A.3
WASTE ROCK STORAGE DRAINAGE DITCH – PHASE 3, TOUQUOY GOLD MINE

Stantec

Memo

To:	**Melissa Nicholson, P.Eng.**	From:	Jeff Gilchrist, P.Eng.
	Atlantic Mining NS Inc.		Stantec Consulting Ltd.
Cc	Scot Klingmann Sara Wallace Paul Deering Dan McQuinn		
File:	121619250	Date:	February 23, 2021
Doc No.	**MEM-147-900.300-D-19FEB21**		

Reference: Waste Rock Storage Area Drainage Ditch – Phase 3, Touquoy Gold Mine

It is understood that additional storage volume for waste rock is required at the Touquoy Mine Site. To accommodate the increase in volume, AMNS proposes to expand the previously defined WRSA footprint to the north/northwest, as shown on Drawing No. SK-86, Rev. 2. Stantec has completed design for Phase 3 of the of the Waste Rock Storage Area (WRSA) drainage ditch.

The purpose of the drainage ditches and ponds associated with the WRSA design is to collect and convey surface water runoff and shallow seepage from the WRSA stockpile during active mine operation.

This is an update to MEM-147-900.300-C-23DEC20, issued on December 23, 2020. The purpose of this memo is to outline the design criteria and highlight any deviation from the previously outlined design basis.

BACKGROUND

Stantec completed the Phase I and II design for the WRSA ditches. Details of the designs were provided in Stantec memo numbers MEM-059-900.400-B-23Mar18 and MEM-112-900.300-B-02AUG19, issued March 23, 2018 and Auguste 2, 2019.

In general, it is understood that the Phase I and Phase II designs are functioning adequately. Therefore, the same general design criteria, concepts and methodology used for the Phase I and II design have been used for Phase 3. As a due diligence, the design basis was reviewed as part of the Phase II work and any required changes are summarized in this memo.

Stantec completed an as-built drawing of WRSA Phase I drainage, titled Phase I Waste Rock Pile Storage Area As-Built Surveys 2018-10-25 and 2018-11-12. The submitted drawing was based on survey information and construction details provided by AMNS, since the site construction monitoring was completed by others, not Stantec.

During our site investigation for Phase 3 (results further discussed below), a test pit was completed at the north end of Phase I, within the existing berm. The test pit inferred that the constructed berm that forms the ditch does not meet the Phase I design. The Phase 1 ditch/berm should be investigated further and modified/replaced as required to meet the design as presented in our document MEM-

059-900.400-B-23Mar18. **This should be completed prior to the initiation of the Phase 3 ditch construction.**

Phase II ditch construction was monitored in the field by Stantec staff and an as-built drawing was completed. The field records and as-built information has been incorporated into this design.

EXISTING SITE CONDITIONS FOR PHASE 3 DESIGN

A geotechnical investigation was completed in January 2021 to support the design of the Phase 3 drainage ditch. The report is titled "*Geotechnical Investigation – Waste Rock Storage Area Drainage Ditches – Phase 3*", dated February 22, 2021.

The general subsurface conditions encountered at the test pit locations consisted of vegetation and rootmat overlying silty SAND (SM) with gravel, underlain by till and inferred bedrock. At one test pit excavated within the existing Phase I ditch the following was encountered: a surficial layer of rockfill overlying approximately 1 m of sandy lean clay fill overlying rockfill underlain by native silty sand (SM) and gravel. The anticipated conditions and recommendations are summarized in the geotechnical report. The stratigraphy at the test pit locations has been added to the profile on the attached drawing.

There is an existing wetland near 0+300 of the Phase 3 ditch that will be contained by a berm outside of the waste rock pile to create a ditch. The wetland area in the footprint of the WRSA if left in place may promote the pooling of water and impact slope stability of the pile. For drainage purposes, it would be prudent excavate unsuitable materials in the area and replace with local borrow clay till graded to promote drainage to the ditch. This item should be reviewed at the time of construction in consultation with the designers of the waste rock pile.

SUMMARY OF PHASE 3 DESIGN CRITERIA AND CONSTRAINTS

WRSA Phase 3 Design Footprint

Design footprint of the WRSA was confirmed by AMNS prior to proceeding with design and is shown on the attached drawing. The following additional constraints were provided regarding footprint;

- Minimum 30 metre (m) offset from existing water bodies, unless permitted to disturb (permitting completed by others);
- Minimum 30 metre (m) offset from existing wetlands, unless permitted to disturb (permitting completed by others);

WRSA Phase 3 Perimeter Drainage

Phase 3 of the design includes the addition of perimeter ditching along the north and northwest of the WRSA. The ditches are to be excavated into native till/bedrock or lined on the exterior slope and bottom with clay till liner to minimize seepage from the ditches to the surrounding environment. In addition there is a section to the north that incorporates a buried culvert. The culvert was selected as the preferred options based on discussions with AMNS. General design criteria include:

- A standard ditch geometry of 1.0 metre (m) width at the bottom and one (1) m minimum depth with 2:1 (Horizontal: Vertical) side slopes
- Surface catchment area of 132,176 m² for the WRSA Phase 3 expansion

- Ditch is to be lined with rip rap for erosion control as per the drawings
- Geotextile to be placed between the subgrade and the rip rap in areas noted on the drawings or determined based on the site conditions assessed during construction.
- Minimum ditch/culvert slope is -0.5%
- Culvert is to be a minimum inside diameter of 0.75 m and have a trash rack installed on the invert to prevent debris from entering the pipe.
- Convey peak flows for the 25-year return period storm event

It is recommended that water quality of drainage water from the WRSA continue to be assessed as data becomes available to verify that the design assumptions are acceptable.

West Collection Pond Storage and Pumping Requirements

Drainage from Phase 3 will be conveyed through the existing west dich/swale and into the west collection pond. The west collection pond size was limited due to site constraints during initial design, therefore pumping is required to meet storage requirements of the west collection pond. As part of the Phase II design, the west collection pond has storage capacity up to the 2-year return period storm. Larger storms require pumping from the west pond to the TMF or spilling from the emergency spillway. The spillways should be completed prior to Phase 3 expansion ditching completion or preparations should be made for additional pumping during larger storm events.

It is understood that the current infrastructure is set up to pump from the west collection pond directly to the TMF using a CD100M pump. The discharge lines are 150 mm diameter HDPE pipes to the TMF as well as one additional pipe from the west collection pond to the east collection pond. The infrastructure would also allow the use of CD150M pumps to increase capacity in storm events.

Based on the capacity of the pumps provided by AMNS, the west collection pond will require additional pumping during the design 25-year return period storm event. To achieve the pumping requirements, the following pumps (or equivalent) could be used: one CD100M pump (existing at site), and two additional CD150M pumps. This scenario would involve additional piping infrastructure which should be installed prior to completion of the ditches at the site. The additional pumps would have to be sourced in preparation for the storm or in a short time period during the storm to reduce the risk of spilling from the spillway. We understand from AMNS that this pump can be sourced and onsite within 6 hours of notification. For larger storm events, or back to back design storm occurrences within 60 hours, the ponds will overflow and require spilling from the emergency spillways or require additional pumping.

As an alternative to additional pumping and pumping infrastructure, modification to the west pond could be reviewed to increase the storage capacity and reduce pumping requirements.

Existing Monitoring Well WRW-1 A/B

Monitoring well WRW-1A/B is located within the footprint on the proposed WRSA Expansion. Monitoring of this well is required as part of the Industrial Approval for the site. It is proposed to decommission the existing well and install a replacement well to the north, outside the WRSA footprint. Location of the well will be submitted to Nova Scotia Environment for approval prior to installation.

Reference: Waste Rock Storage Area Drainage Ditch – Phase 3, Touquoy Gold Mine

It is recommended that the new well be installed prior to decommissioning the existing well and a series of sampling events of both wells be completed. The period that data overlaps will be used to relate the new well to the baseline data set for WRW-1 A/B.

DESIGN DRAWING

Issued for Tender Drawing No. SK-86, Rev.2, Waste Rock Storage Area Phase 3 Ditch Design.

CONSTRUCTION RECOMMENDATIONS

Construction of the ditching should be monitored full time by Stantec geotechnical personnel.

LIMITATIONS

Water Quality

It is understood that water quality is being reviewed as part water quality modeling updates and routine water quality monitoring and therefore is not included in this scope of work. Should water quality results indicate potential issues from the WRSA, they should be investigated at that time and mitigated to minimize impact on the surrounding environment.

Stockpile Design

It is understood that the stockpile design is being completed by others. Stantec should be given an opportunity to review the proposed drainage design once the final geometry of the stockpile as designed by others is finalized.

Detailed Modelling

Stantec assumed that the existing ditch that will be covered by waste rock pile would act as a conduit for water flow and represent the drainage divide for toe seepage flowing into the designed ditch. No modeling was completed to predict the drainage path of water through the waste rock pile as the pile is "wetted". Stantec recommends modelling this scenario once the final waste rock pile is designed to confirm the drainage design requirements. In addition, this modeling will provide additional information for closure and water balance analyses.

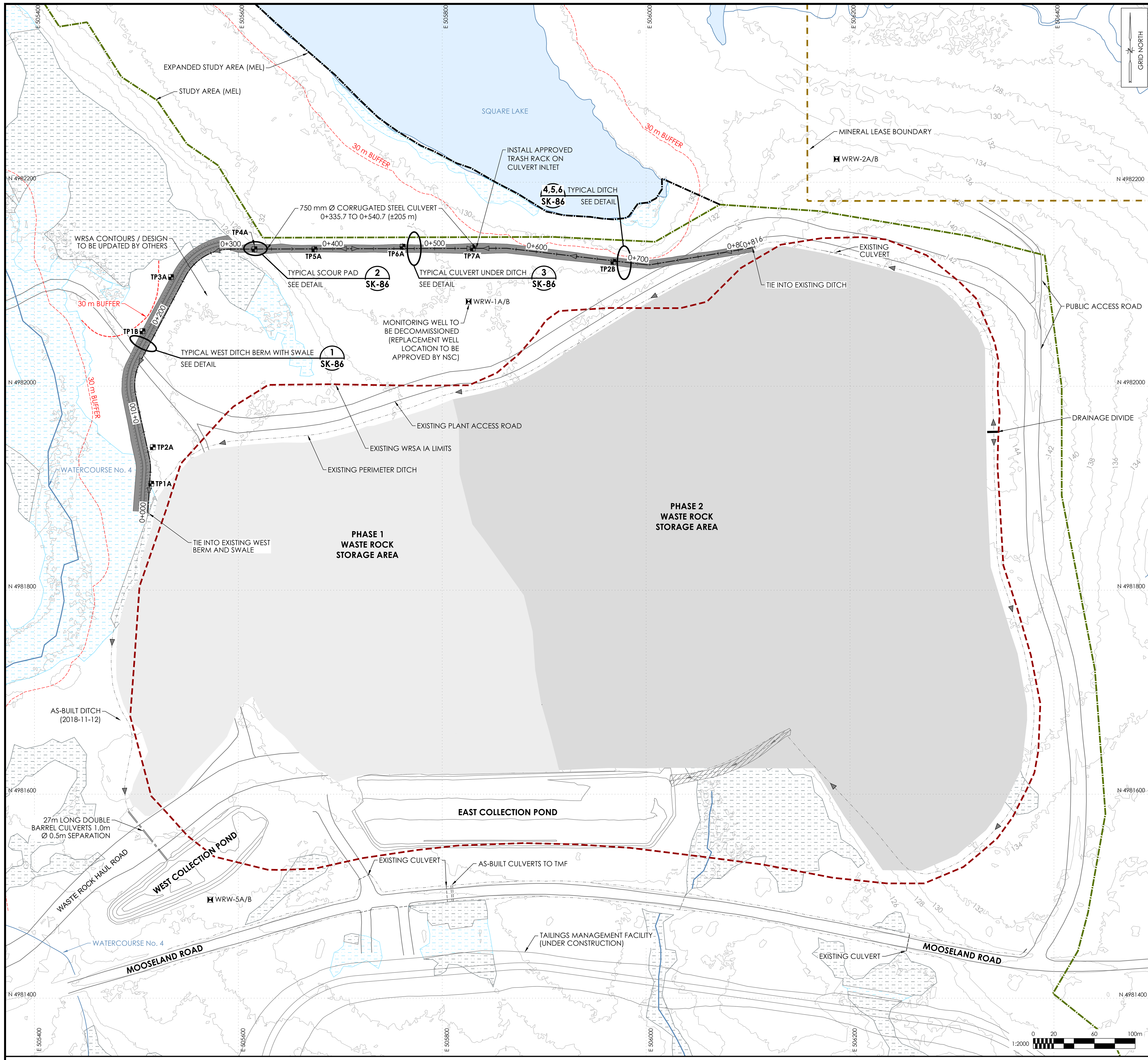
CLOSURE

We trust this meets your current requirements. If you have any questions, please contact us at your convenience.

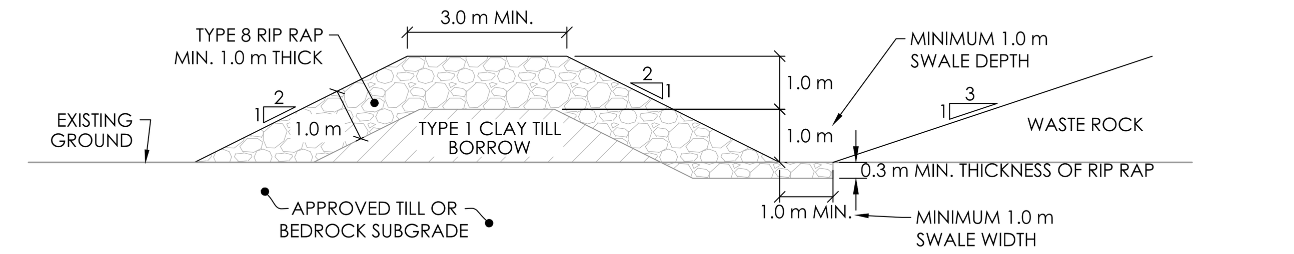
STANTEC CONSULTING LTD.

Jeff Gilchrist, P.Eng.

Attachment: Drawing SK-86 Rev.2, Waste Rock Storage Area Phase 3 Ditch Design, Issued for Tender

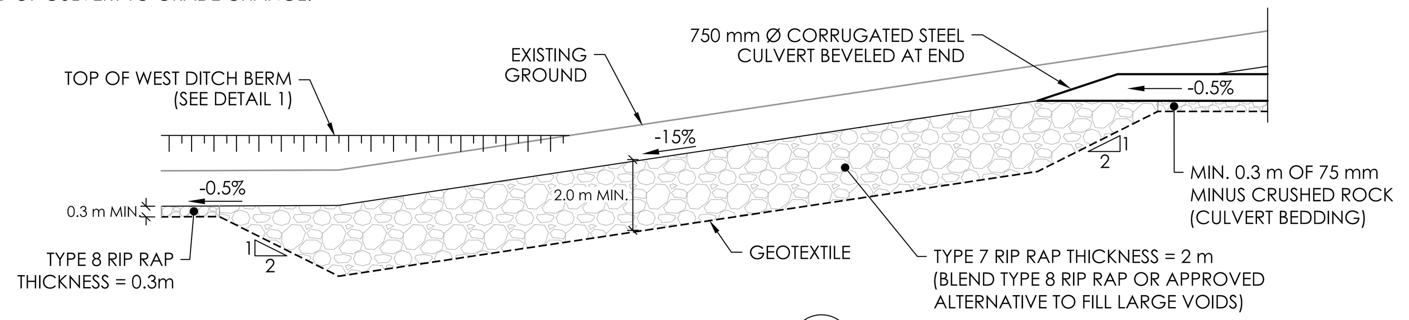


NOTE:
GEOTEXTILE TO BE PLACED BETWEEN THE SUBGRADE/TYPE 1 CLAY TILL BORROW AND THE RIP RAP IN AREAS NOTED ON THE DRAWING; IF GROUNDWATER, WET CONDITIONS OR SEEPAGE EXIST DURING CONSTRUCTION; OR OTHERWISE DIRECTED BY THE ENGINEER.

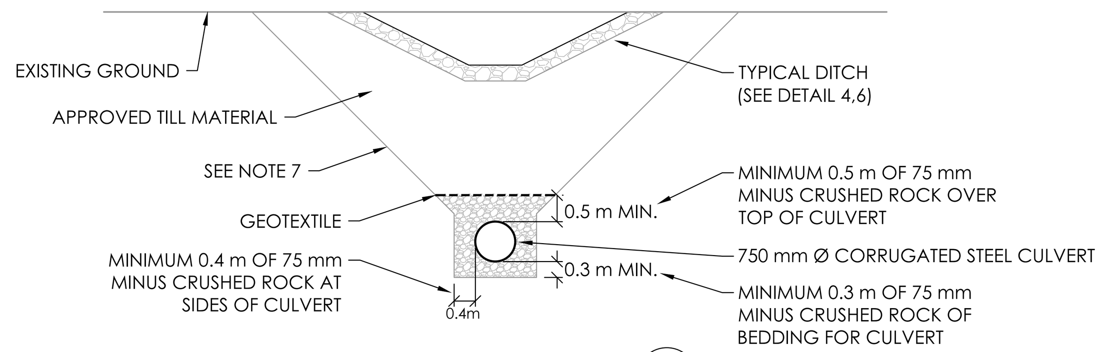


TYPICAL WEST DITCH BERM WITH SWALE (1, SK-86)
SCALE 1 : 100

NOTE:
SCOUR PAD 2.0 m DEPTH TO EXTEND FROM END OF CULVERT TO GRADE CHANGE.

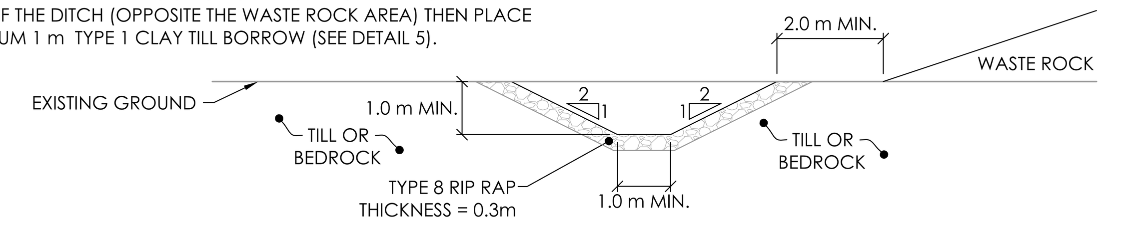


TYPICAL SCOUR PAD (2, SK-86)
SCALE 1 : 150



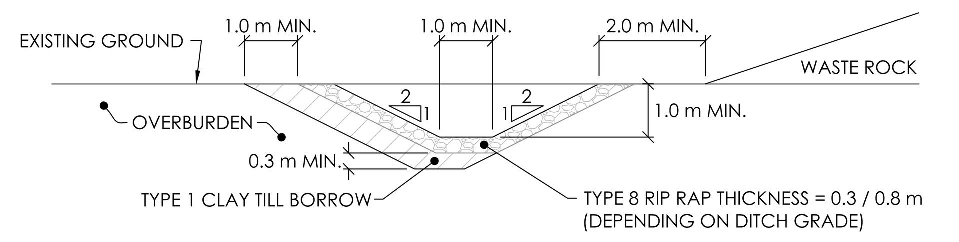
TYPICAL CULVERT UNDER DITCH (3, SK-86)
SCALE 1 : 100

NOTE:
1. GEOTEXTILE TO BE PLACED BETWEEN THE SUBGRADE AND THE RIP RAP IN AREAS NOTED ON THE DRAWING; IF GROUNDWATER, WET CONDITIONS OR SEEPAGE EXIST DURING CONSTRUCTION; OR OTHERWISE DIRECTED BY THE ENGINEER.
2. IF SUITABLE TILL OR BEDROCK IS NOT PRESENT ON THE DOWNSTREAM SIDE OF THE DITCH (OPPOSITE THE WASTE ROCK AREA) THEN PLACE MINIMUM 1 m TYPE 1 CLAY TILL BORROW (SEE DETAIL 5).



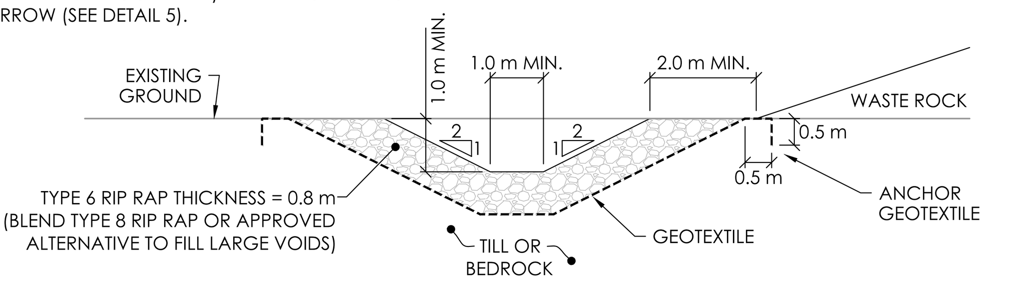
TYPICAL DITCH FOR SLOPES <-6% (4, SK-86)
SCALE 1 : 100

NOTE:
GEOTEXTILE TO BE PLACED BETWEEN THE SUBGRADE AND THE RIP RAP IN AREAS NOTED ON THE DRAWING; IF GROUNDWATER, WET CONDITIONS OR SEEPAGE EXIST DURING CONSTRUCTION; OR OTHERWISE DIRECTED BY THE ENGINEER.

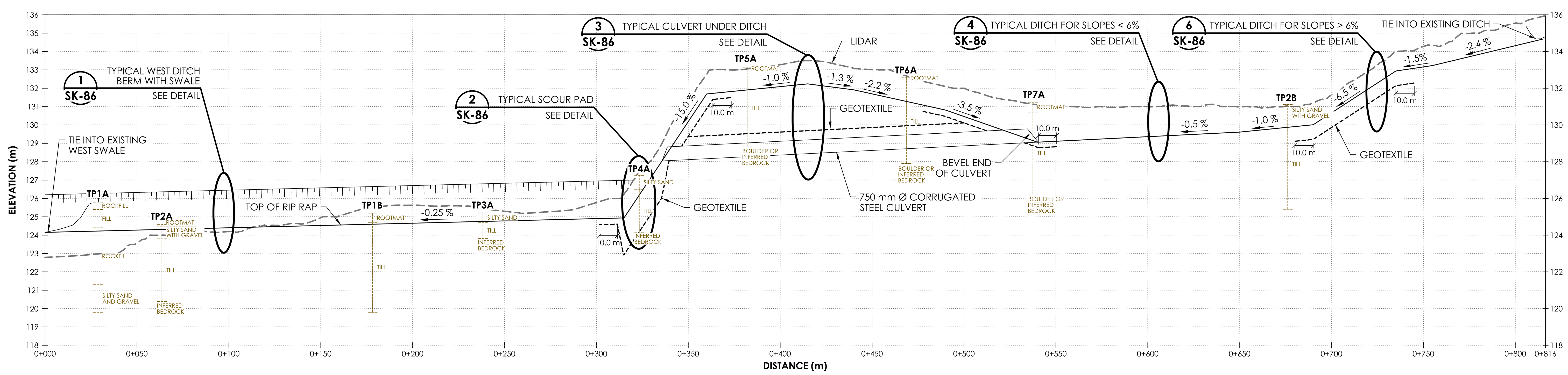


TYPICAL DITCH WITH CLAY LINER (5, SK-86)
SCALE 1 : 100

NOTE:
IF SUITABLE TILL OR BEDROCK IS NOT PRESENT ON THE DOWNSTREAM SIDE OF THE DITCH (OPPOSITE THE WASTE ROCK AREA) THEN PLACE MINIMUM 1 m TYPE 1 CLAY TILL BORROW (SEE DETAIL 5).

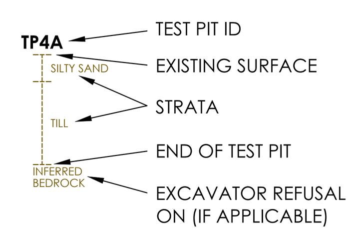


TYPICAL DITCH FOR SLOPES >-6% (6, SK-86)
SCALE 1 : 100



LONGITUDINAL PROFILE (2, SK-86)
HORIZONTAL SCALE 1 : 1500
VERTICAL SCALE 1 : 150

TEST PIT NOTES:



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Tel. 506.452.7000
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Legend

- LIDAR, 2010
- EXISTING ROAD
- EXISTING WATERCOURSE
- WATERBODY
- WETLAND
- PERMITTED WETLAND ALTERATION
- STUDY AREA (McCALLUM ENVIRONMENTAL LTD.)
- EXPANDED STUDY AREA (McCALLUM ENVIRONMENTAL LTD.)
- MINERAL LEASE BOUNDARY
- EXISTING WRSA IA LIMITS
- EXISTING PERIMETER DITCH CENTERLINE
- 30 m BUFFER
- PROPOSED DITCH FOOTPRINT
- CENTERLINE OF PROPOSED DITCH
- DITCH FLOW DIRECTION
- GROUNDWATER MONITORING WELL
- TEST PIT

Notes

1. BACKGROUND TOPOGRAPHY SHOWN IS BASED ON LIDAR DATA PROVIDED BY CONESTOGA-ROVERS & ASSOCIATES (2010). ACTUAL FIELD CONDITIONS MAY VARY.
2. EXISTING DITCHES AND PONDS BASED ON SURVEY DATA PROVIDED BY ATLANTIC GOLD CORPORATION.
3. PLEASE REFER TO THE EROSION AND SEDIMENT CONTROL PLAN (STANTEC 2010) DEVELOPED FOR THE TOUQUOY SITE FOR BEST MANAGEMENT PRACTICES FOR CONSTRUCTION WORKS RELATED TO EROSION AND SEDIMENT CONTROL AT THE SITE. TO ALIGN WITH THE KEY PRINCIPLES OF THE PLAN THE CONSTRUCTION WORK SHALL MINIMIZE THE AMOUNT OF EXPOSED SOIL AS WELL AS THE TIME OF THE EXPOSURE TO MINIMIZE EROSION AND SEDIMENT TRANSPORT. IF REQUIRED, EXPOSED SOILS SHALL BE COVERED WITH NON-ERODIBLE MATERIALS, SEALED WITH A ROLLER OR TARPED BEFORE END OF DAY EACH CONSTRUCTION DAY.
4. ADDITIONAL BEST MANAGEMENT PRACTICES MAY NEED TO BE IMPLEMENTED TO MINIMIZE EROSION AND SEDIMENT TRANSPORT FROM THE SITE. THESE SHALL BE IMPLEMENTED AS REQUESTED BY ATLANTIC MINING NOVA SCOTIA OR DESIGNATE DURING CONSTRUCTION.
5. GRADING WILL BE REQUIRED TO MEET SUBGRADE ELEVATION AND ACHIEVE MINIMUM OF -0.5% GRADE SHOWN ON DRAWING. THIS MAY INCLUDE COMMON EXCAVATION, ROCK EXCAVATION AND BERM CONSTRUCTION.
6. ROUTINE CLEAN OUT OF DITCH MAY BE REQUIRED TO REMOVE SEDIMENT.
7. TEMPORARY STABILITY OF EXCAVATION IS THE RESPONSIBILITY OF THE CONTRACTOR.

COLLECTION POND	CATCHMENT AREA BASED ON TOE SEEPAGE	CATCHMENT AREA BASED ON FINAL WASTE ROCK SURFACE
WEST	242,900 m^2	261,600 m^2
EAST	250,840 m^2	232,135 m^2

ISSUED FOR TENDER

STAMP

STAMP

No.	DETAILS	BY	APPD.	DATE
2	TEST PITS ADDED TO PLAN VIEW AND LONGITUDINAL PROFILE	JG	PD	2021-02-23
1	DITCH ALIGNMENT CHANGE ISSUED FOR REVIEW	JG	PD	2020-12-21
0	ISSUED FOR REVIEW NOT FOR CONSTRUCTION	JG	PD	2020-04-23

REVISIONS

CLIENT:
ATLANTIC MINING NS INC.

PROJECT:
TOUQUOY GOLD PROJECT
TAILINGS MANAGEMENT FACILITY

PROJECT LOCATION:
HALIFAX COUNTY, NOVA SCOTIA

JOB No.:	SCALE:	DATE:
121619250	AS SHOWN	2021-02-23
DRAWN BY:	DESIGNED BY:	APPROVED BY:
JL	JG/RJ	PD

DRAWING TITLE:
WASTE ROCK STORAGE AREA
PHASE 3 DITCH DESIGN

DRAWING No.: **SK-86**
REVISION No.: **2**