APPENDIX 2A

Water Management Plan (Stantec)

Valentine Gold Project (VGP) Water Management Plan

Final Report

September 28, 2020

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Executive Summary

The mine site is broken up into three complexes, from northeast to southwest, the Marathon Complex, the Process Plant and TMF Complex, and the Leprechaun Complex. The major project facilities include the Leprechaun and Marathon open pit mines, process plant, TMF, mining services, Leprechaun and Marathon waste rock facilities and Low Grade Ore stockplies, topsoil and overburden piles, accommodations, access road, and accommodations camp. An overview of key features of the TMF water management plan is as follows:

- Water management infrastructure were designed under a decentralized water treatment framework, operating under gravity drainage to reduce pumping needs.
  - Surface runoff upstream of the project facilities will be diverted away to predevelopment catchments, where possible.
  - Perimeter ditches around the piles (i.e., waste rock, topsoil, and overburden stockpiles) will flow into water management ponds and discharge to the FDPs.
  - The Processing Plant Pad runoff will be directed to a water management pond prior to discharge to a watercourse.
  - Mine water from dewatering the open pit will be collected in sumps and pumped to a water management pond prior to discharge to the environment.
- Perimeter dams will be constructed in downstream raises to impound the tailings and provide flexibility in construction and distribute construction costs over the life of the facility, thus maintaining adequate storage and freeboard during operation.
- During operation, the TMF receives water from the processing plant via tailings slurry water, seepage collection pond discharge (intercepting tailings seepage from the TMF and pumping back into the pond for treatment), runoff from tailings pond un-diverted upstream catchment areas and direct precipitation.
- Water retained in the tailings pond will be exposed to sunlight to facilitate natural CN degradation and provide further sedimentation
- Losses from the TMF include reclaim water to the process plant, discharge to the polishing pond, water retained in the tailings matrix, deep groundwater seepage, and evaporation;
- Excess tailings water will be treated in an effluent treatment plant prior to discharge to the polishing pond during 8 months of the year
- The polishing pond will provide additional passive treatment and control the timing and amount of discharge. The polishing pond water will be released to a pipeline draining to Victoria Lake Reservoir
- Mining of the Marathon and Leprechaun pits will occur simultaneously until the end of Year 9. In Year 10, tailings deposition to the TMF as beaches will switch to subaqueous deposition in the Leprechaun Pit.
- Water withdrawal from Victoria Lake Reservoir is proposed as a freshwater make-up source for processing ore at the mill during operation, and to accelerate filling of the Leprechaun pit during closure. Water withdrawal from Valentine Lake is proposed to accelerate filling of Marathon pit during closure.

- Progressive rehabilitation activities will include adding a soil cover and vegetating waste rock pile benches as they are developed, stabilizing disturbed areas through vegetation, filling the Leprechaun pit with tailing and water during Years 10-12 to accelerate pit filling.
- Rehabilitation & Closure will involve activities to return the site to pre-development conditions, including, stabilizing through vegetation consumed topsoil, overburden, and Low Grade Ore stockpile areas, vegetating the tailings beach, dismantling and removing the buildings and, allowing the pits to fill with water.
- Sedimentation ponds and perimeter seepage collection ditches will be maintained until water quality meets objectives during Post-Closure & Monitoring phase of development.

Abbreviations

AEP	Annual Exceedance Probability
ARD	Acid Rock Drainage
CCME	Canadian Council of Ministers of the Environment
CEAA	Canadian Environmental Assessment Agency
CDA	Canadian Dam Association
Marathon	Marathon Gold Corporation
CWQG	Canadian Water Quality Guidelines
EA	Environmental Assessment
EEM	Environment Effects Monitoring
EIS	Environmental Impact Statement
ESC	Erosion and Sediment Control
ESCP	Erosion and Sediment Control Plan
FAL	Freshwater Aquatic Life
FDP	Final Discharge Point
GCDWQ	Guideline for Canadian Drinking Water Quality
km	Kilometres
LGO	Low Grade Ore
LOWL	Low Operating Water Level
MAC	Maximum Allowable Concentrations
MAF	Mean Annual Flow
MDMER	Metal and Diamond Mining Effluent Regulations
m	Meter
ML	Metal Leaching
Mt/a	Million tonnes per year
NL	Newfoundland and Labrador
NLDECCM	Newfoundland and Labrador Department of Environment, Climate Change and Municipalities
NOWL	Normal Operating Water Level
PAG	Potentially Acid Generating
PMF	Probable Maximum Flood
PMP	Probable Maximum Precipitation
POPC	Parameters of Potential Concern
QA/QC	Quality Assurance and Quality Control
ROM	Run-of-Mine
TMF	Tailings Management Facility
TSS	Total Suspended Solids
WMP	Water Management Plan

1.0 INTRODUCTION

This Surface Water Management Plan (WMP) supports and guides the construction, operation and closure of the Valentine Gold Project (the Project), located in the Central Region of the Island of Newfoundland, south of Valentine Lake. The proposed Project will include two open pits, waste rock piles, crushing and stockpiling areas, conventional milling and processing facilities (the mill), a tailings management facility (TMF), personnel accommodations, and supporting infrastructure, including roads, on-site power lines, buildings, and water and effluent management facilities. This live “working” WMP has been prepared by Stantec Consulting Ltd. (Stantec) for Marathon Gold Corporation (Marathon), as the Project proponent. Marathon is committed to reducing environmental effects through the implementation of mitigation measures, monitoring and adaptive water management for the Project. The current WMP version focuses primarily on water quantity and quality.

Closely integrated documents that supported the preparation of the WMP can be found in the references section of this document and are listed below:

- 2019 Baseline Hydrology and Surface Water Quality Monitoring Program (Stantec 2020a)
- Valentine Gold Project - Pre-Feasibility Study (Marathon & Ausenco 2020)
- Prefeasibility Study for Tailings Disposal at the Valentine Gold Project (Golder 2020), including the TMF water balance modelling report
- Basis of Design for Pre-Feasibility Level Water Management Design Input – Final (Stantec 2020b)
- Valentine Gold Project – Geochemistry Report (Stantec 2020c).
- Valentine Gold Project – Fish and Fish Habitat Valued Component Chapter (Stantec 2020d)
- Water Quantity and Water Quality Modelling Reports for the Leprechaun Complex and Processing Plant & TMF Complex, and Marathon Complex (Stantec 2020e,f)
- Valentine Gold Project – Assimilative Capacity Assessment (Stantec 2020g)
- Valentine Gold Project: Acid Rock Drainage/Metal Leaching (ARD/ML) assessment report (Phase II). (Stantec 2020g).

1.1 OBJECTIVES

The primary objectives of the water management design are to reduce operational risks and environmental effects of the Project. These objectives include:

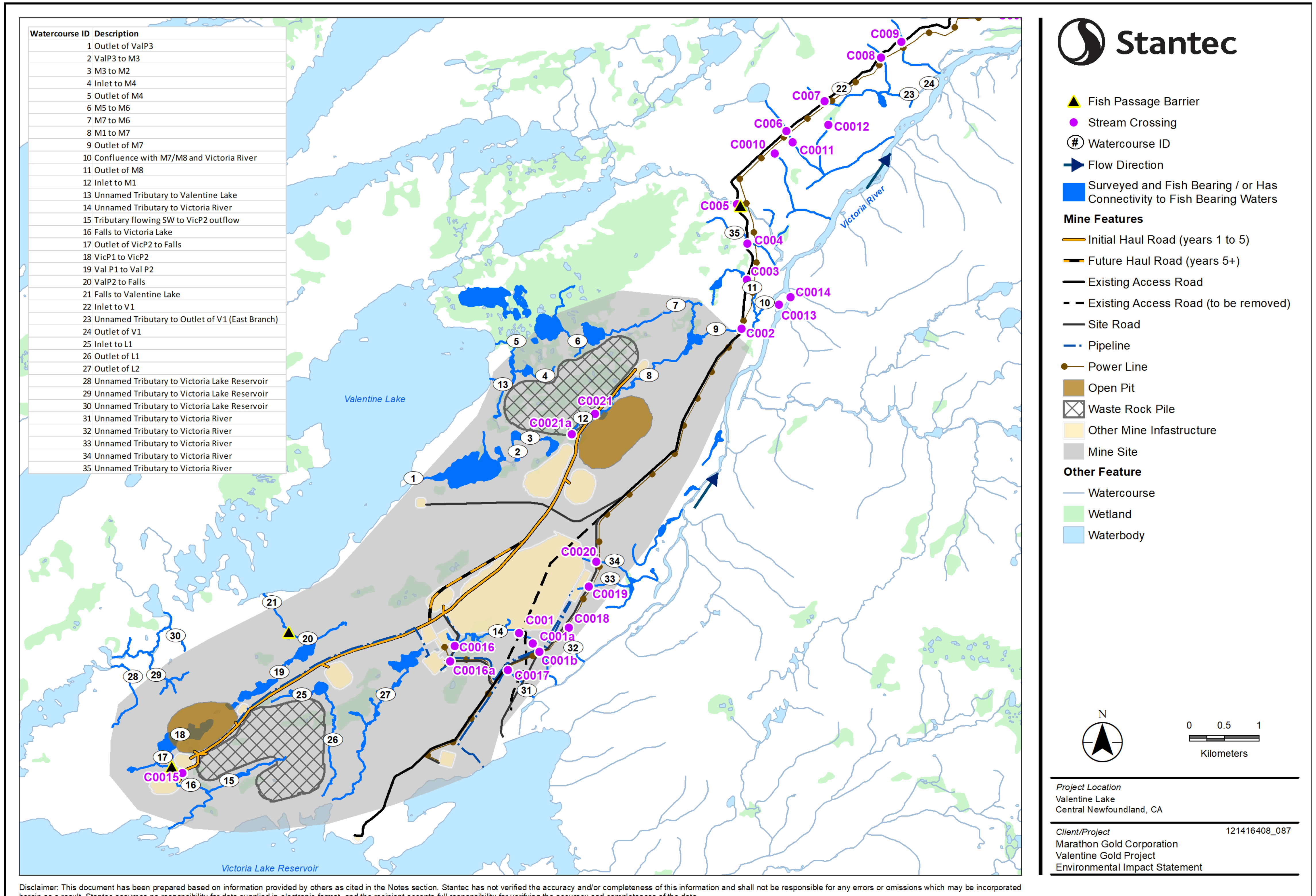
- Reduce water inventory requiring management through perimeter berms to divert external non-contact runoff
- Reduce the number of final discharge points (FDPs) through grading of ditches and construction of diversion channels to combine discharge points water management ponds
- Maintain flow to fish bearing streams and wetlands by maintaining pre-development catchments to the extent feasible
- Reduce water management costs during operation through grading and gravitational drainage and thereby reduce pumping requirements

1.2 BACKGROUND

1.2.1 Hydrology

The Project is centered on a topographic ridge that divides the drainage between the Valentine Lake watershed to the west and the Victoria Lake Reservoir and Victoria River watersheds to the south and east, respectively. A series of large waterbodies form the Exploits and Bay d'Espoir watersheds, which are two of the largest watersheds on the Island of Newfoundland and are significantly altered and controlled by hydroelectric developments. Valentine Lake historically drained north to the Victoria River to Red Indian Lake and then further downstream to the Exploits River. The construction of a series of dams and connecting channels associated with the Bay d'Espoir Hydroelectric facility, diverted Victoria Lake from the Victoria River toward the hydroelectric facility to the east.

The Project facilities are located at the headwaters of several watercourses, waterbodies, and wetlands, as presented in Figure 1.1 with Victoria Lake Reservoir to the south, Victoria River to the east, Valentine Lake, a headwater tributary Lake to the Victoria River to the west and Victoria River tributaries to the north. Streams denoted in in dark blue in Figure 1.1 have been field surveyed as fish bearing or having connectivity to fish bearing waters.



Watercourse ID	Description
1	Outlet of ValP3
2	ValP3 to M3
3	M3 to M2
4	Inlet to M4
5	Outlet of M4
6	M5 to M6
7	M7 to M6
8	M1 to M7
9	Outlet of M7
10	Confluence with M7/M8 and Victoria River
11	Outlet of M8
12	Inlet to M1
13	Unnamed Tributary to Valentine Lake
14	Unnamed Tributary to Victoria River
15	Tributary flowing SW to VicP2 outflow
16	Falls to Victoria Lake
17	Outlet of VicP2 to Falls
18	VicP1 to VicP2
19	Val P1 to Val P2
20	ValP2 to Falls
21	Falls to Valentine Lake
22	Inlet to V1
23	Unnamed Tributary to Outlet of V1 (East Branch)
24	Outlet of V1
25	Inlet to L1
26	Outlet of L1
27	Outlet of L2
28	Unnamed Tributary to Victoria Lake Reservoir
29	Unnamed Tributary to Victoria Lake Reservoir
30	Unnamed Tributary to Victoria Lake Reservoir
31	Unnamed Tributary to Victoria River
32	Unnamed Tributary to Victoria River
33	Unnamed Tributary to Victoria River
34	Unnamed Tributary to Victoria River
35	Unnamed Tributary to Victoria River

Figure 1.1 Overview of Project Watercourses, Waterbodies and Wetlands

1.2.2 Hydrogeology

Based on a review of geological maps and aerial photographs, the overburden material in the vicinity of the Project primarily consists of a discontinuous layer of till of variable thickness. Along with glacial deposits, areas of organic and peaty soils are present overlying either till or bedrock in areas of poor drainage. Areas of high ground in the Leprechaun and Marathon deposit areas are characterized by bedrock outcrop exposed within the till veneer and various other surficial deposits characterized by sandy silt. A well-defined northeast-trending regional fault (Valentine Lake Shear Zone) occurs immediately to the south of the Leprechaun deposit (Stantec 2017).

The prominent topographic ridge that underlies the Project is inferred to act as a regional flow divide for both surface water drainage and groundwater flow and defines an area of groundwater recharge. Overall, the direction of shallow groundwater flow is expected to follow topography and surface runoff, and discharge into the low-lying surface waterbodies that border the property.

Locally, groundwater flow from the Marathon deposit is expected to travel southeast towards the Victoria River and northwest towards Valentine Lake, which flows into Victoria River northeast of the Project, and ultimately discharges into the Exploits River, approximately 100 kilometres (km) to the north. Groundwater flow from the Leprechaun deposit is expected to primarily travel south-southeast towards Victoria Lake Reservoir, with a lesser component flowing north towards Valentine Lake.

As reported in the 2018 Hydrogeology Baseline Report (Stantec 2019), groundwater elevations vary across the site and generally reflect the topographic relief of the area, with higher groundwater elevations occurring in boreholes / wells located at higher topographic elevations (Stantec 2019). A groundwater elevation change of 100 meters (m) was observed between the topographic highs of the exploration corridor connecting the two pits (maximum elevation of approximately 420 m relative to the Canadian Geodetic Vertical Datum of 1928 (CGVD28) to Valentine Lake (elevation of 319 to 326 m CGVD28) and Victoria Lake Reservoir (elevation of 320 m CGVD28) (Stantec 2020b). Annual fluctuations of water levels collected in the five measured boreholes over the calendar year of November 2017 to November 2018 were less than 0.8 m (Stantec 2017) and no seasonal trend was observed.

1.2.3 Surface Water Chemistry

Regional water quality reported at the Environment and Climate Change Canada (ECCC 2020) managed sites (ID NF02YN0001 Lloyds River at Bridge, RTE 480, Burgeo Road and NF02YO0107 Exploits River Approx. 0.5km Downstream from Dam) between 2003 and 2019 includes metals, nutrients, and physical parameters. Total alkalinity (as $CaCO_3$) ranges from below detection limit 1.22 mg/L to 11 mg/L. Low alkalinity values suggest limited acid buffering potential in streams. Parameters were generally below the applicable Canadian Council of Ministers of the Environment (CCME) Canadian Water Quality Guidelines for the Protection of Aquatic Life (Freshwater) (CWQG-FAL; CCME 2010,2019,2020), with at least one reported exceedance of the maximum value for aluminum, cadmium, copper, iron, and lead reported at ECCC station NF02YO0107, and aluminum and selenium at station NF02YN0001.

As noted in the 2019 baseline water quality report (Stantec 2020a), surface water quality was monitored at 26 locations in the mine site between 2011 and 2019. The lab results indicated that pH ranged from 4.61 to 7.78 with a mean value of 6.94. A total of 18 of 26 water quality monitoring stations were lower than the CWQG-FAL lower limit of 6.5 for pH. Local water quality was found to be similar to regional water quality in that both were found to have low alkalinity, and therefore limited acid buffering potential. Some metals were also detected above CWQG-FAL guidelines at both the regional and local water quality monitoring locations (aluminum, cadmium, copper, iron, and lead). These results indicate that metals are found in naturally elevated levels both in local and regional surface water. Local water quality monitoring revealed consistent seasonal concentration trends, and that water quality in larger lakes such as Victoria Lake Reservoir and Valentine Lake was more dilute and lower in constituent concentrations than observed in tributary watercourses, ponds and wetlands.

1.2.4 Groundwater Chemistry

Baseline water quality testing to date (Stantec 2020c) indicates a calcium-sodium-bicarbonate-chloride-sulphate type groundwater that is characterized as clear (colour overall <15 Total Colour Units or TCU), slightly hard to hard (20.9 mg/L to 122 mg/L as Calcium Carbonate ($CaCO_3$)), slightly alkaline with moderate acid buffering potential and low conductivity, indicating fresh conditions. Langelier Saturation Index values for groundwater samples indicate groundwater is neither strongly corrosive nor scale-forming with respect to solid $CaCO_3$. Metals parameters were generally low with the exception of iron and manganese.

1.2.5 Local Water Users

The Victoria Dam and spillway are located at the north end of Victoria Lake Reservoir, just downgradient of the Project. This dam infrastructure is part of the Bay d'Espoir Hydroelectric Development. The Bay d'Espoir Hydroelectric Generating Facility is the largest hydroelectric plant in Newfoundland and includes three generating stations, six reservoirs, and associated dykes, dams, canals, and hydraulic structures. The generating stations comprising the Bay d'Espoir Development were built in stages beginning in 1967. There are four remote hydraulic structures associated with the Bay d'Espoir Development: Ebbegunbaeg Control Structure, Salmon River Spillway Structure, Victoria Control Structure (or Victoria Dam), and Burnt Dam Spillway (Newfoundland and Labrador Hydro 2012).

The Victoria Control Structure is a dam at the outlet of Victoria Lake Reservoir to the Victoria River, which naturally flowed north to Red Indian Lake. This dam raised the natural lake elevation from 290 to 325m and has a crest elevation of 326m. The low supply level of the lake, set by the Victoria Canal, was set at 319 m. In the late 1960s, Victoria Lake Reservoir was diverted to the Victoria Canal, which flows into the White Bear drainage basin to the south (Read & Cole 1972). The Victoria Canal was designed to convey between 34 m^3/s at low supply level and 170 m^3/s at full supply level (Read & Cole 1972).

2.0 PROJECT OVERVIEW

Water management within the Project Area is broken up into three complexes, from northeast to southwest, the Marathon Complex, the Process Plant and Tailings Management Facility (TMF) Complex, and the Leprechaun Complex. The overall site plan is presented in Figure 2.1 which depicts the major Project facilities of each complex, such as open pits, process plant, TMF, mining services, waste rock facilities, accommodations, access road, and accommodations camp. Access to the mine site is from the northeast via an existing access road extending south from the Town of Millertown.

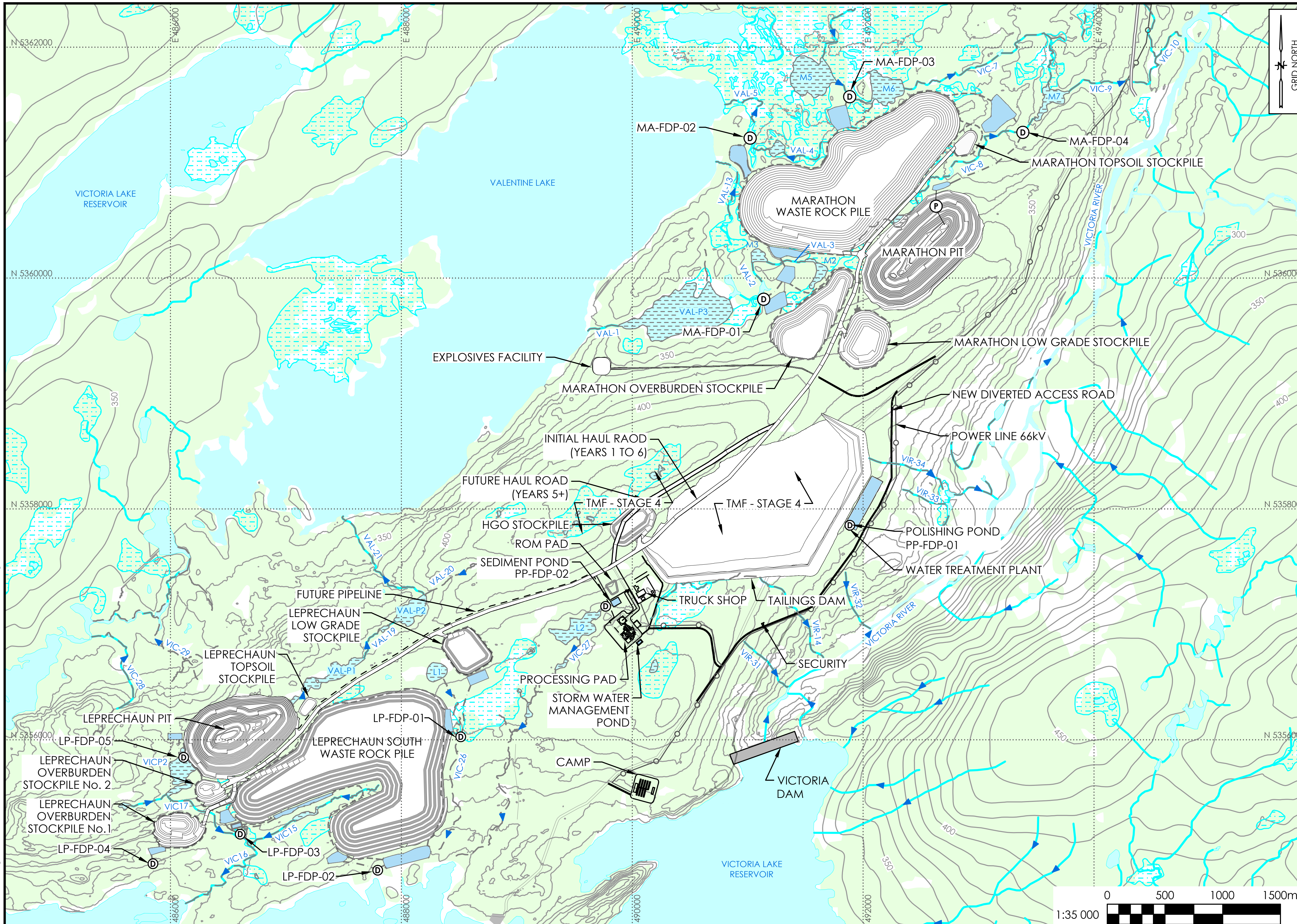
2.1 PROJECT FACILITITES

The Marathon complex Project facilities consist of the Marathon pit, waste rock pile, topsoil stockpile, overburden stockpile, low-grade ore stockpile and water management infrastructure. The Leprechaun Complex consists of the Leprechaun pit, a waste rock pile, low-grade ore stockpile, topsoil and overburden stockpiles, and water management infrastructure. The Processing Plant and TMF Complex consists of the TMF, polishing pond, water treatment plant, process plant, truck shop, wash-ROM pad, and high-grade ore stockpile.

Other Project facilities include the accommodation camp and an existing exploration camp that will be used until the accommodation camp is operational and maintained as overflow accommodations. An explosives facility will be constructed northwest of the TMF. Other site buildings include:

- Administration and security offices, change rooms and the plant lunchroom in separate building, warehouse building, laboratory building
- Vehicle maintenance and storage areas (4)
- Mine services including mine offices, a mine truck wash, a truck shop (maintenance), and a fuel station
- Diesel fuel storage tanks located with a bermed catchment area
- Prefabricated electrical buildings, plant main substation, and overhead power lines to supply power to the site

The Project will include haulage roads to accommodate haul truck loads, grades and passing of two-way traffic. The mine site is accessed by an existing public access road that extends south from Millertown approximately 88 km to Marathon’s existing exploration camp. Marathon will upgrade and maintain the access road from a turnoff approximately 8 km southwest of Millertown to the mine site, a distance of approximately 76 km.



LEGEND

- EXISTING CONTOURS, m
- PROPOSED MINE PIT / STOCKPILE
- WATERCOURSE
- FISH BEARING WATERCOURSE
- WOODED AREA
- WETLAND
- WATERBODY
- FISH BEARING WATERBODY
- PROPOSED POND
- PROPOSED DRAINAGE CHANNEL
- PROPOSED DITCH
- FUTURE PIPELINE
- POWER LINE
- FINAL DISCHARGE POINT

NOTE:
COORDINATES ARE NAD83(CSRS) UTM ZONE 21.

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

Reference:

1. EXISTING CONTOURS AND PROPOSED INFRASTRUCTURE: AUSENCO; PROJECT No. 104878-01; DRAWING No. 104878-0000-G-001; 2020-05-11; PRELIMINARY.
2. WATERCOURSES, WATERBODIES & WETLANDS: CANVEC DATABASE FROM NATURAL RESOURCES CANADA.
3. FISH BEARING WATERCOURSES AND WATERBODIES: SURVEYED FISH BEARING OR HAS CONNECTIVITY TO FISH BEARING WATER (STANTEC 2012, 2019, 2020).

OVERVIEW OF PROJECT COMPONENT AREAS - ULTIMATE FOOTPRINT

WATER MANAGEMENT PLAN

VALENTINE GOLD PROJECT, NL

Client:	MARATHON GOLD CORP
Job No.:	121416408
Scale:	1 : 35 000
Date:	28-SEP-2020
Dwn. By:	JL
App'd By:	NS
Fig. No.:	2.1

Stantec

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As presented in the Pre-Feasibility Study (Marathon & Auseco 2020), site selection and location of the Project facilities took the following factors into consideration:

- Locate the ROM pad between the two open pits, to minimize haul distance
- Utilize the natural high ground for the ROM pad as much as possible
- Separate heavy mine vehicle traffic from non-mining, light-vehicle traffic
- Locate the process plant in an area safe from flooding
- Locate the heavy equipment foundation on competent bedrock and utilize rock anchors for foundations design
- Upgrade and utilize the existing access road to reach the site
- Place mining, administration and processing plant staff offices close together to limit the footprint of the project facilities
- Reduce outdoor walking distances between buildings (important during extreme cold weather)
- Locate the ready line close to the mining admin/office area and change house
- Avoid known fish habitation areas

2.2 MINE PHASES OF DEVELOPMENT

The overall Project development schedule will consist of three phases: construction, operation, and decommissioning, rehabilitation and closure. For convenience, “closure” in this document refers to the first five years of the decommissioning, rehabilitation and closure phase, while “post-closure” refers to the remainder of this phase. Project activities within these phases are further subdivided for the purposes of this report as discussed in the following sections.

2.2.1 Construction

Construction activities will occur over 16 to 20 months, associated to mine Year -1.

Construction activities will include site preparation, earthworks, infrastructure construction, equipment and utilities installation and TMF construction.

Site preparation activities will include cutting and clearing of vegetation and removing organic materials and overburden on areas to be developed. Developing construction stage water and erosion control, such as ditching, water management ponds and construction access roads for the waste rock piles, stockpiles and TMF clearing will also be part of site preparation activities. Earthworks will include excavating, preparing excavation bases, placing structural fill, and grading; stripping and stockpiling organic and overburden materials for open pits for future rehabilitation; and use of open pit development rock (waste rock) for infrastructure such as structural fill and road gravels.

Infrastructure construction consists of placing concrete foundations and constructing buildings and Project infrastructure. Concrete will be primarily batched on-site and some pre-cast building footings may be poured off-site and transported to the site. Coarse aggregates will be crushed from mine waste rock and/or site rock quarries and fine aggregate materials such as sand will be sourced from local quarries. Equipment installation will include installing major Project infrastructure equipment such as the ROM hopper and water treatment equipment. Utility installations involve constructing and connecting power, water and fuel supply infrastructure. The TMF will be partially constructed during the construction phase and will be continually raised during operation to meet tailings and water storage requirements and Plant operating criteria.

The waste rock piles and overburden and topsoil piles will be constructed as follows:

- 10 m lift heights for overburden/topsoil
- 15 m lift heights for waste rock
- 1.5:1 horizontal to vertical (H:V) active slopes of overburden/topsoil lifts
- 1.3H:1V active slopes on waste rock lifts
- berm allowances push slopes out to approximately 3H:1V

Additional groundwater wells will be installed around the perimeter of Project facilities to support monitoring of project interactions.

2.2.2 Operation

Commissioning of the mine is planned to occur in year 1. The mine will be in operation for 12 years.

2.2.2.1 Marathon and Leprechaun Complex

Open Pits

Operation of the open pits will include drilling, blasting, loading and hauling of ore and waste rock to storage areas using conventional mining equipment. Leprechaun and Marathon pits will be mined simultaneously. Over the 9-year operation phase, there will be progressive expansion of the open pits, with associated vegetation clearing and overburden removal and storage. Explosives used in mining will be contained in an explosive storage facility onsite. The pits will be mined simultaneously with plans for the ore rock to be mixed and processed together. Mining of the open pits will cease at the end of Year 9 for both the Marathon and Leprechaun pits.

Ore extracted from the open pits will be hauled to stockpiles or the processing area. Ore grading between 0.33 and 0.50 grams per tonne (g/t) of gold (Au) will be stockpiled in the associated low-grade ore stockpiles. Cut-off grade optimization on the mine production schedule will also send ore above 0.50 g/t Au to a high-grade ore stockpile in certain planned periods. The processing plant will include a pre-processing period at a reduced milling rate of 2.5 Million tonnes per year (Mt/a) of ore material from open pit mines, increasing to 4 Mt/a in Mine Year 4.

The open pits will be dewatered throughout active mining operation. The collected contact water will be stored in a sump pit prior to being pumped to a water management pond at the surface. Water from the water management ponds will be used to supplement mill demand or discharged to the environment following treatment in the water management ponds as needed to meet discharge quality criteria.

Waste Rock and Overburden Piles

Waste rock from the milling process will be placed in the simultaneously active Marathon and Leprechaun waste rock piles, with one waste rock pile located in proximity to each open pit. These waste rock piles will be constructed from the existing ground surface and will be sloped and benched as they are developed, which will result in final safe slopes for closure of 3H:1V once the benches have been rehabilitated. In addition, the waste rock piles will be progressively rehabilitated during operation by covering benches with a vegetated soil cover to reduce infiltration into the piles.

The Marathon waste rock pile is located immediately northwest of the Marathon pit limits. The pile will be constructed to an ultimate crest elevation of 415 m. Topsoil from the Marathon pit will be stored in a topsoil stockpile 0.5 km north of the pit limits and overburden will be stored in a stockpile directly southwest of the pit limits. The Leprechaun waste rock pile is located just southeast of the Leprechaun pit limits and built up to a crest elevation of 430 m. Topsoil from the Leprechaun pit will be stored in a stockpile directly west of the pit limits and overburden will be stored in two stockpiles directly southwest of the pit limits. Piles are separated to avoid local natural water courses.

Ore will be hauled to a crusher 3.5 km southeast of the Marathon pit and 3.0 km northeast of the Leprechaun pit. Ore will be crushed to feed the process plant; waste rock will be deposited into waste rock piles adjacent to the pits, or used as rockfill to construct a tailings dam 2.0 km southeast of the Marathon pit and 4.5 km northeast of the Leprechaun pit.

Contact runoff from the piles will be managed by perimeter ditches and treated in water management ponds prior to release to the environment.

2.2.2.2 Processing Plant and TMF

The pre-production phase of operation will consist of crushing, semi-autogenous and ball milling, gravity recovery, leaching-absorption, carbon elution, and gold recovery. Leach-adsorption tails will be treated for cyanide destruction, thickened, and deposited in the TMF. The subsequent full production milling phase will consist of crushing and milling as before, but with the addition of a pebble crusher, gravity recovery, floatation, floatation concentrate thickening, floatation concentrate regrind, floatation concentrate leaching-adsorption, floatation tails thickening, floatation tails leaching-adsorption, carbon elution, and gold recovery. Reagents used in the milling process include quicklime, sodium cyanide, frother, promoter, hydrochloric acid, copper sulphate pentahydrate, sodium metabisulphite, sodium hydroxide, flocculant, activated carbon, and smelting fluxes.

Processing is divided into two periods of operation; the initial processing period and full production period. The initial processing period has a nominal throughput of 6,859 tonnes per day (t/d) or 2.5 Mt/a. As the mill feed grade decreases, and plant capacity is required to increase to maintain gold production, the mill will operate at full production rate of 10,960 t/d or 4.0 Mt/a. At full production, flotation equipment will be employed to recover the majority of the gold to a low mass concentrate stream, and ultra-fine grinding and cyanidation.

The TMF will receive precipitation and the process water discharged with the tailings slurry. Excess water from the open pit dewatering and runoff from waste rock piles are managed separately and do not report to the TMF. A water treatment plant and polishing pond allow for the treatment and discharge of the excess TMF site water to Victoria Lake Reservoir. Treatment and discharge will only occur for eight months each year. The tailings pond, with a maximum storage capacity of 1 Mm^3, has been sized to store the excess site water during the non-discharge period. The storage accounts for the environmental design flood and inflow design flood, while maintaining sufficient freeboard within the tailings pond. Reclaim water is pumped from a floating barge and pump in the TMF to the process plant.

The processing plant and TMF will operate as a circuit with tailings being deposited in the TMF as a thickened slurry (60% to 75% solids) and process water being reclaimed during thickening and via a pump and pipeline from a decant barge in the TMF. In year 10, active open pit mining ceases and processing will continue from stockpiled ore. At this point, tailings deposition is switched from the TMF to the Leprechaun pit, process water will continue to be supplemented by TMF reclaim water, in addition to the minimum of 8% freshwater make-up from Victoria Lake Reservoir.

As described below, freshwater make-up and elution water will be pumped from Victoria Lake Reservoir to the process plant, amounting to approximately 13% of process water during pre-production and 8% of process water during full production. Surplus water from the TMF will be discharged to a treatment plant, from which treated water will be sent to a polishing pond prior to discharge via a pipeline to Victoria Lake Reservoir. Ore rock will be stored on the run-of-mine stockpile and in the high-grade ore stockpile prior to processing.

A continuous downstream raise of the TMF dam will be constructed to meet requirements for water and tailings storage. The primary construction material for the TMF is the waste rock from the open pits. The first four stages will be constructed with a crest width of 20 m to facilitate the use of mine haulage equipment in dam construction. The final stage will have a crest width of 10 m and may require smaller earthmoving equipment for the final few metres of the dam raise. The average upstream slope flattens to about 3.5H: 1V accounting for the benches and a 2H:1V downstream slope. On the upstream slope, a 1 m thick (measured perpendicular to the slope) coarse filter/ transition layer will be placed on the prepared waste rock slope followed by a 1 m thick fine filter layer. A 1.5 mm thick linear low-density polyethylene (LLDPE) geomembrane will be installed, as the main water retaining element, on the fine filter layer. A 0.3 m thick layer of road surfacing will be placed and compacted along the dam crest to allow for light vehicle traffic during operations.

Dam runoff and seepage is captured in the perimeter seepage collection ditches and pumped back to the TMF. Water management onsite also includes diversion of non-contact freshwater around the Project and collection of contact water.

The polishing pond will be constructed with perimeter embankments above the natural topography; therefore, external run-off will be diverted away from the pond. The catchment of the pond is only the pond itself.

2.2.3 Decommissioning, Rehabilitation and Closure

This section outlines the general rehabilitation and closure plan for the Project. A formal Rehabilitation and Closure Plan will be developed, as required under the *Newfoundland and Labrador Mining Act* (Chapter M-15.1 Section 8, 9 and 10, Government of Newfoundland 2000).

The Project will have three key stages of rehabilitation activities that occur over the life span of the mine, which include:

- progressive rehabilitation
- closure & rehabilitation
- post-closure & monitoring

Progressive rehabilitation will occur over the nine years of active open pit mining and three years of stockpiled ore to reduce the amount of time runoff comes in contact with the mine facilities. Progressive rehabilitation involves activities that would otherwise be carried out during closure and will be completed proactively wherever possible and practical. Re-vegetation studies and trials will commence early in operation to support progressive reclamation activities. The following general proactive rehabilitation activities will be implemented in construction and operation activities, to further support progressive rehabilitations measures:

- Disturbances of terrain, soil, and vegetation will be limited to the areas necessary to complete the required work
- Organic soils, mineral soils, glacial till, and excavated rock will be stockpiled separately where practicable, and protected for future use
- Stabilization of disturbances will be completed to reduce erosion and promote natural revegetation
- Natural re-vegetation will be encouraged throughout the Project

Rehabilitation activities will continue during closure at the end of ore processing to restore the property as close to pre-development conditions as practicable, or to an alternate use or condition that is deemed appropriate and acceptable by Newfoundland and Labrador Department of Industry, Energy and Technology. Closure rehabilitation is anticipated to be completed over approximately five years.

General rehabilitation and closure activities include:

- On site wells will be decommissioned. This includes dewatering wells, groundwater monitoring wells, potable drinking water wells and/or industrial water wells. The decommissioning will comply with Government of Newfoundland and Labrador Guidelines for Sealing Groundwater Wells.
- Pre-mining site drainage patterns will be re-established to the extent practicable.
- Disturbed areas will be graded and/or scarified, covered with overburden and organic materials, where required, and seeded to promote natural re-vegetation.
- Demolishing and rehabilitation of construction or exploration-related buildings, roads, laydown areas, etc. will be conducted as part of progressive reclamation
- Hazardous chemicals, reagents, and similar materials will be removed for re-sale or disposal at an approved facility as per regulations.

- Equipment will be disconnected, drained and cleaned, disassembled, and where possible, sold for re-use to a licensed scrap dealer. If this is not achievable, equipment will be removed from site for disposal or recycled at an approved facility.
- Site buildings and surface infrastructure will be dismantled and removed for disposal or recycling at approved facilities.
- Concrete foundations will be demolished to a minimum of 0.3 m below the surface grade and covered with natural overburden materials to promote re-vegetation. Demolished concrete will be used as fill material for re-grading or removed from site for disposal in an appropriate facility.
- Fuel and explosive storage and dispensing facilities will be removed, and these areas rehabilitated. Phase I and potentially Phase II Environmental Site Assessments may be required to evaluate for potentially impacted soils and groundwater.
- Infrastructure footprint areas will be stabilized with vegetation.

2.2.3.1 Marathon and Leprechaun Complexes

The closure activities associated with the major components of the Project for the Marathon and Leprechaun Complexes are summarized below:

Waste Rock Piles and Stockpiles

- Slopes of waste rock piles will be constructed at 3H:1V requiring no adjustment in slopes for final closure
- Overburden and topsoil material will be progressively placed on benches and slopes and revegetated as the pile is developed during operation and the remaining areas of pile during closure
- Overburden and topsoil stockpiles will be depleted during the first two years of closure, and these areas will be seeded to promote natural re-vegetation
- The placement of a vegetated soil cover on waste rock piles and un-processed low grade ore stockpiles will effectively create two water streams: a non-contact surface runoff stream, and a contact seepage stream

Water Management Infrastructure

- Perimeter ditches will backfilled with overburden and covered with a vegetated soil cover as per the piles themselves creating the following conditions:
  - Non-contact runoff will drain down the pile slopes and benches, over the perimeter ditch footprints and overland to local receivers following natural drainage patterns
  - Contact seepage will be substantially reduced from the uncovered condition due the increase in runoff and evapotranspiration potential of the vegetated soil cover. The reduced volume contact seepage will migrate across the perimeter ditches and assimilate (attenuate naturally) with local groundwater to discharge into local receiving waters
  - In cases where natural attenuation of contact seepage will not be adequate to improve groundwater discharge quality at the local receiver to background or CWQG-FAL thresholds, further passive treatment systems may be required
  - Passive treatment systems could take the form of subsurface anaerobic units in the ditches or subsurface / surface units that utilize the water management pond basins as constructed wetland

features. In both cases seepage is transmitted in the ditch in a french drain subsurface arrangement

- Not withstanding the need for additional passive treatment, water management ponds will be breached to allow drainage to the natural ground and local receivers

Water quality monitoring during the operation and decommissioning, rehabilitation and closure phases will inform closure water management. Adaptive management will provide the flexibility to respond to emerging conditions proactively.

Open Pits

- Barricades and signage placed along the high-walls of the open pits as part of progressive rehabilitation
- Dewatering infrastructure will be removed
- Pits will be allowed to naturally fill with water accelerated by pumping water from Valentine Lake (Marathon Pit) or Victoria Lake Reservoir (Leprechaun Pit) Pit lake overflow discharge is expected to meet regulatory closure water quality discharge requirements

2.2.3.2 Processing Plant & TMF Complex

The closure activities associated with the major components for the Processing Plant and TMF Complex are summarized below:

Tailings Management Facility

- Grading and revegetation of completed tailings areas, when possible as part of progressive rehabilitation
- Further grading of the existing TMF downstream embankment slopes of 2H:1V will not be required, as the slopes already meet Canadian Dam Association (CDA) closure criteria
- The tailings solids within the TMF impoundment will be capped with overburden, topsoil and revegetated
- The surface of the TMF will be contoured as necessary to promote drainage towards the tailings pond
- Downstream slope of the TMF dam will be left as exposed rockfill to permit drainage of the downstream shell
- A larger closure spillway will be constructed to convey water from within the impoundment
- Reduce pond water storage in the TMF to classify the TMF as a landform and therefore alleviating the requirements for maintaining and inspecting the dams post-closure
- Decant pump will be decommissioned once water quality demonstrates that water collected in the pond is acceptable for direct release to the environment
- Seepage water collection system, including the pumps will be kept in service until water quality monitoring demonstrates that water collected in the system is acceptable for direct release to the environment. At that time, the pumping systems will be removed and the sumps will be backfilled.
- When no longer required, the seepage collection ditches and sump areas will be re-contoured to restore the original drainage course to the extent possible and to enhance the area for natural re-vegetation

- Similar to closure planning for the waste rock piles, surface runoff from the covered and vegetated tailings surface will be non-contact water not requiring further treatment. TMF seepage ditches and sump pits could be repurposed as passive water quality treatment systems if required during closure

2.2.4 Post-Closure

During the post-closure period, the site monitoring is carried out to demonstrate that closure strategies are performing as intended. The post-closure monitoring program will continue after final closure activities are completed for an estimated 6 to 10 years. However, the monitoring period could be adjusted based on the agreement of the regulatory bodies that all physical and chemical characteristics are acceptable and stable. When the project is deemed physically and chemically stable, the site can then be closed out or released by NLDNR and an application made to relinquish the property back to the Crown. NLDNR is currently reviewing and revising the approach to relinquishment respecting closed mining projects containing potential ARD/ML mine waste and/or dams.

3.0 WATER MANAGEMENT CRITERIA

3.1 SURFACE WATER QUANTITY CRITERIA

The Project will be registered under the *Metal and Diamond Mining Effluent Regulations* (MDMER). The MDMER pursuant to the federal *Fisheries Act*, comes into force on the first day that a mine releases more than 50 cubic metres (m^3) in a single day. The MDMER sets a daily flow volume monitoring requirement at each final discharge point. Therefore, a criterion in design was to combine points of water management pond effluent, where feasible, to reduce the amount of FDPs subject to MDMER.

Under MDMER, the deposit of mine waste into a fish bearing watercourse or water body will trigger a Schedule 2 application. A criterion in design of water management infrastructure was to not overprint fish-bearing watercourses or water bodies with mine waste. Mine effluent will be compliant with effluent criteria set out in MDMER.

Design criteria for water management ponds and ditching will refer to the Newfoundland and Labrador (NL) *Mining Act* and *Water Resources Act,* regulation requirements and guidance. Where a water management pond requires a dam meeting the definition of a dam in the NL *Water Resources Act* and CDA, then further criteria become relevant.

Water use is regulated by the NL Department of Environment, Climate Change and Municipalities (NLDECCM) through permitting requirements for activities within 15 m of a water body related to withdrawal of water, installation of intake structures, dams and culverts and discharge of wastewater. A 15 m setback from field identified fish bearing or assumed fish bearing streams and bogs/ponds was applied. This design criterion is in line with the Newfoundland and Labrador Policy on Flood Plain Management (NLDOEC 2014).

The Provincial EIS Guidelines (Government of NL 2020) requires that climate change be considered in design. The Representative Concentration Pathway 4.5 (RCP4.5) was applied to climate records to simulate rainfall over the next 20 years. This resulted in higher precipitation events and higher associated design flows.

Regulation of dam safety in Canada is primarily a provincial responsibility. Design criteria for the water management design was to meet the most stringent requirements of the CDA and the NL *Mining Act*. The CDA classifies a dam as an embankment of 2.5 m or greater from the toe of the downstream slope to the dam crest and 30,000 m^3 of liquid storage. A criterion was to design berms to avoid the CDA dam classification.

As part of the EIS, a maintenance flow to fish-bearing streams is required to reduce environmental effects to fish and fish habitat. Therefore, flow to fish bearing streams and wetlands was maintained in design, where possible by draining mine site components to pre-development catchment areas and using low flow outlet structures to augment baseflow to receiving waters.

Marathon's corporate direction was to also consider water management costs during operation. Consistent with industry best practice this was accomplished by reducing mine site water inventory as much as feasible through separation of groundwater and surface water flows by setting water management infrastructure at or above the groundwater table. Additionally, the mine site's water inventory will be reduced by construction of perimeter berms around the Marathon and Leprechaun pits to divert overland flow from entering the pits. Placement of infrastructure will reduce stranded areas of runoff and allow for diversion of overland flow of non-contact water away from the site.

Water management features were designed under a decentralized water treatment framework, operating under gravity drainage to reduce pumping needs. Water management design considered optimization of cuts and fills to reduce initial trucking cost and utilize local materials.

Water quantity control criteria applied in design of water management ponds, include:

- Store runoff from the Project component areas for storm events up to 1:100 Annual Exceedance Probability (AEP) with spring snowmelt and accommodate up to the 1:200 AEP in the spillway
- Slowly release water management pond effluent to the environment to provide flood attenuation and reduce downstream scour and erosion
- Augment baseflows through the installation of a low level outlet to maintain an environmental maintenance flow

3.2 SURFACE WATER QUALITY CRITERIA

The primary water quality criteria applicable to the Project is the following:

- CCME CWQG-FAL;
- Schedule 4 of the MDMER under the *Fisheries Act*; and
- Schedule C of NL Regulation 65/03 *Environmental Control Water and Sewage Regulations*, 2003 under the *Water Resources Act* (O.C. 2003-231).
- Environmental effects of mine effluent in relation to receiving watercourses or water bodies baseline water quality (CEAA, 2019 and NLDMAE 2020), to satisfy requirements of the EIS.

Schedule C of NL Reg. 65/03 states:

"A person primarily in the Metal Mining Industry shall comply with sections 3 and 19.1 and 20 and Schedule 4 of the Metal Mining Effluent Regulations (Canada) SOR/2002-222, including any changes or amendments to those sections of and that schedule to those regulations over time."

Therefore, as the Project is the proposed development of a metal mine, the CWQG-FAL and MDMER are the primary water quality criteria. The CWQG-FAL are those used to assess baseline quality and assimilative capacity and MDMER are those used to establish effluent limits. CWQG-FAL and MDMER criteria for parameters assessed in this study are presented in Table 3.1.

Table 3.1 Summary Regulatory Criteria and Reference Water Quality in Project Area

Parameter	Units	Regulatory Criteria		
		CWQG-FAL[1]	MDMER[2]	
			Max Monthly Mean	Max Grab
Alkalinity	mg/L			
Colour	TCU	Narrative		
Conductivity	µS/cm			
DO	mg/L	Variable (6.5 – 9.5 (cold water – life stage))		
pH	pH	6.5 – 9.0		
Turbidity	NTU	Narrative		
TSS	mg/L	Narrative	15	30
Calcium	mg/L			
Chloride	mg/L	120		
Flouride	mg/L	120		
Magnesium	mg/L			
Potassium	mg/L			
Sodium	mg/L			
Sulphate	mg/L	128 [a]		
Cyanide	mg/L	0.005 (as free CN)	1	2
DOC	mg/L			
Nitrogen (N)	mg/L			
Un-ionized Ammonia	µg/L	19	0.5 mg/L (expressed as N)	1.0 mg/L (expressed as N)
Nitrite	mg/L	0.06		
Nitrate	mg/L	13		
Phosphorus	µg/L	<4 - >100 (trophic status)		
Silica	mg/L			
Aluminum	µg/L	5 if pH < 6.5, 100 if pH > 6.5		
Arsenic	µg/L	5	500	1,000
Barium	µg/L			
Boron	µg/L	29,000 (short-term); 1,500 (long-term)		
Beryllium	µg/L			
Cadmium	µg/L	Hardness adjusted (range of 0.04 to 0.37)		
Cobalt	µg/L			
Chromium	µg/L			
Copper	µg/L	Hardness adjusted (range of 2 to 4)	300	600

Table 3.1 Summary Regulatory Criteria and Reference Water Quality in Project Area

Parameter	Units	Regulatory Criteria		
		CWQG-FAL[1]	MDMER[2]	
			Max Monthly Mean	Max Grab
Iron	µg/L	300		
Lead	µg/L	Hardness adjusted (range of 1 to 7)	200	400
Lithium	µg/L			
Manganese	µg/L			
Mercury	µg/L	0.026		
Molybdenum	µg/L	73		
Nickel	µg/L	Hardness adjusted (range of 25 to 150)	500	1,000
Selenium	µg/L	1		
Strontium	µg/L			
Silver	µg/L	0.25		
Thallium	µg/L	0.8		
Uranium	µg/L	33 (short-term), 15 (long-term)		
Vanadium	µg/L			
Zinc	µg/L	Hardness, pH, and DOC adjusted (30)	500	1,000
$Radium_{226}$	Bq/L		0.37	1.11

Notes:
[1] CWQG – Canadian Water Quality Guidelines for the Protection of Aquatic Life (CCME 2019)
[2] MDMER – Metal and Diamond Mining Effluent Regulations, values presented in the table are maximum authorized concentration in grab samples (MDMER 2019). The MDMER provides three effluent water quality limits including the maximum authorized monthly mean concentration, maximum authorized concentration in a composite sample and maximum authorized concentration in a grab sample. The Maximum Authorized Monthly Mean Concentration will be the MDMER effluent criteria carried forward in Project effects assessments.
[a] - Sulfate Guideline is for British Columbia Ministry of Environment and Climate Change Strategy 2017 for the protection of aquatic life

Water management ponds will be installed to treat runoff in contact with Project facilities for sediment removal. Additional measures to control erosion and prevent sedimentation into fish bearing watercourses or waterbodies of conveyance features was accomplished in design through ditch and berm lining for erosion protection and energy dissipation measures, such as sediment traps and energy dissipation pools.

Water quality control criteria applied in design of water management ponds, include:

- Runoff from the Project component areas for storm events up to 1:10 AEP to allow settlement of sediments to meet MDMER
- As particle size distributions were not available for the waste rock piles, the water management ponds were designed to treat a silt sized particle of 5.0×10^{-3} mm in diameter (BC MELP 1996), which is a typical particle size in design of a sedimentation pond.

- Ponds were designed primarily to meet the minimum residence time required for sediment to drop 1 m reaching a trapping efficiency of 80%.
- Runoff from the water quality design storm event will be detained in the water management pond for a minimum of 24 hours.
- A submerged type low-level outlet will also act as a hydrocarbon and Light Non-Aqueous Phase Liquids (LNAPL) containment feature as well as to reduce thermal discharge effects.
- A minimum length to width ratio of the water management pond of 2:1 to minimize short circuiting.

3.3 GROUNDWATER QUANTITY CRITERIA

The main potential effect to groundwater quantity during mine operation is potential dewatering of the surficial and bedrock aquifer surrounding the open pits. Groundwater quantity is also expected to be affected by reduced recharge in the vicinity of the waste rock piles and TMF. These changes in water level and recharge will be monitored in the groundwater monitoring network throughout the Project phases of development.

Effects on the nearest reported residential groundwater supplies during operations in the vicinity of Buchans and Millertown are negligible due to the distance between the Project and potential well users and the intervening lakes and watershed divides that would act as hydraulic barriers. In the absence of identified domestic well users, the primary receptor of dewatering induced by operation is the surface water receiver.

A regulatory threshold used to assess these changes in groundwater level and recharge rates is not applicable. However, changes will be compared to modelling predictions to trigger whether further assessment is required or to initiate contingency measures to assess the interaction of groundwater with nearby surface watercourse, waterbodies and wetlands. Should a change be identified greater than predictions, the nearby surface watercourse, waterbodies and wetlands will be monitored to identify an interaction with the Project and the resulting environmental effects.

3.4 GROUNDWATER QUALITY CRITERIA

Although groundwater resources in Canada are generally managed by provincial regulatory bodies as described above, the Guidelines for Canadian Drinking Water Quality (GCDWQ) published by Health Canada are also applicable to groundwater across Canada, and have been adopted by the government of NL for regulated public drinking water supplies. As the Project site is not near a current domestic water source, these regulations are not legally binding and will be compared to water quality results to assist in identifying elevated parameter concentrations. The GCDWQ are "*established based on current published scientific research related to health effects, aesthetic effects and operational considerations*" (Health Canada 2020).

Table 3.2 presents the groundwater monitoring guidelines (i.e., GCDWQ) of water quality parameters to be monitored for the Project.

Table 3.2 Groundwater Monitoring Criteria

Parameter	Units	Guideline for Canadian Drinking Water Quality (GCDWQ) (MAC unless otherwise noted)
Total Alkalinity	mg/L	-
Aluminum	mg/L	-
Ammonia	mg/L	-
Antimony	mg/L	0.006
Arsenic	mg/L	0.010
Barium	mg/L	2.0
Beryllium	mg/L	-
Bismuth	mg/L	-
Boron	mg/L	5
Conductivity	μSIE/cm	-
Calcium	mg/L	-
Cadmium	mg/L	0.007
Chloride	mg/L	≤250[AO]
Chromium	mg/L	0.05
Cobalt	mg/L	-
Colour	TCU	≤15[AO]
Copper	mg/L	2.0; ≤1[AO]
Cyanate	mg/L	-
Cyanide	mg/L	0.2
Fluoride	mg/L	1.5
Ion Balance	%	-
Iron	mg/L	≤0.3[AO]
Hardness	mg/L	-
Lead	mg/L	0.005
Potassium	mg/L	-
Magnesium	mg/L	-
Manganese	mg/L	0.12; ≤0.02[AO]
Nickel	mg/L	-
Nitrite (as N)	mg/L	1
Nitrate (as N)	mg/L	10
Nitrate + Nitrite (as N)	mg/L	10
Orthophosphate	mg/L	-
pH	unitless	7.0-10.5
Reactive Silica	mg/L	-

Table 3.2 Groundwater Monitoring Criteria

Parameter	Units	Guideline for Canadian Drinking Water Quality (GCDWQ) (MAC unless otherwise noted)
Silver	mg/L	-
Sodium	mg/L	≤200^{AO}
Reactive Silica	mg/L	-
Sulphate	mg/L	≤500^{AO}
Selenium	mg/L	0.05
Strontium	mg/L	7.0
Total Dissolved Solids	mg/L	≤500^{AO}
Total Organic Carbon	mg/L	-
Total Suspended Solids	mg/L	-
Turbidity	NTU	1
Thallium	mg/L	-
Thallium	mg/L	-
Thiocyanate	mg/L	-
Tin	mg/L	-
Titanium	mg/L	-
Uranium	mg/L	0.02
Vanadium	mg/L	-
Zinc	mg/L	≤5.0^{AO}
Notes MAC = Maximum allowable concentration (Health Canada 2020). AO = Aesthetic objective “-” = Not applicable.		

3.5 TMF WATER MANAGEMENT CRITERIA

The TMF water management features have been designed to withstand flooding events using criteria based on level of consequence of the impounding dam infrastructure.

The proposed TMF includes a tailings dam and polishing pond dam that incorporate current regulatory requirements in the design, including the Canadian Dam Association (CDA 2014, 2019) design standards. Design of the tailings dam crest and invert elevation of associated spillways are determined by considering the Inflow Design Flood, the Environmental Design Flood, the Normal Operating Water Level (NOWL), the Low Operating Water Level (LOWL), and freeboard. As reported by Golder (2020), the storage requirements of water retention structures are summarized in Table 3.3 and detailed in subsequent sections.

Table 3.3 Storage Requirements of Major Water Retention Structures

Dam	Selected Inflow Design Flood	Selected Environmental Design Flood	Selected NOWL	Selected LOWL
Tailings Pond Dam	2/3 between 1:1000 year storm event and the probable maximum flood (PMF) (generated from 309 mm 24 hr PMP at 390.3 m elev.) PMF (Passive Closure)	1:100 year storm in 24 hours, allotted up to1 m of depth in TMF	Climate Normal Conditions Dewatering Surplus	Inactive Storage Condition
Polishing Pond Dam	Not specified	Not Applicable	Climate Normal Conditions Discharge Surplus	Inactive Storage Condition

The Inflow Design Flood is the most severe inflow flood (peak, volume, shape, duration, timing) for which a dam and its associated facilities are designed (CDA 2014). The TMF dams were assessed as having a Very High Consequence classification (Golder 2020). As per the CDA requirement for a very high consequence classification, the Inflow Design Flood should be 2/3 between the 1:1000 year flood event and the probable maximum flood (PMF). The PMF is a flood that results from a precipitation event known as the probable maximum precipitation (PMP). The PMP is defined as the most extreme precipitation event physically possible in the area. The PMP was selected in a supporting prefeasibility level design of the tailings management area both completed by Golder (2020). For passive closure phase, the Inflow Design Flood is raised to the PMF.

The Environmental Design Flood is the most severe flood that is to be managed without release of untreated water to the environment (CDA 2014). Retention of water during the Environmental Design Flood requires storage capacity above the NOWL (CDA 2014). An emergency spillway will enhance the safe operation of the TMF by increasing the range of inflows that can be managed in extreme circumstances. The Environmental Design Flood can be defined by the required assimilation of water quality parameters of concern, such as total suspended solids, arsenic, and cyanide.

As defined by Golder (2020), for components that are temporary in nature or are only expected to service the operational period of the Project, the 1:25 year storm is used as the design criteria within the Processing Plant and TMF Complex. These features include the seepage collection ditch around the tailings pond and associated pumping capacity, mill site pond, and ROM Pad. However, the allotted freeboard height of most of this infrastructure would accommodate flows of a higher storm event. Design of the decant structure pumping capacity and pore spacing was based on the required capacity of the maximum water treatment plant treatment rate of 10,800 m^3/d and the average reclaim flows to the mill for process use.

4.0 WATER MANAGEMENT ANALYSIS

4.1 METHODS

Hydrologic conditions, runoff rate volume and flow hydrographs were simulated using the hydrologic model HEC-HMS (USACE 2000, 2010) for the 1:10 year, 1:100 year and 1:200 year storm events. The Soil Conservation Service (SCS) Curve Number method (USDA 2010) was used to simulate the runoff hydrograph in each Project component area. The watershed parameters required for simulation of the SCS Curve Number method included catchment areas, SCS Curve Number, Initial abstraction, lag time, and baseflow. Time lag was calculated as 0.6 of the time of concentration. Time of concentration was calculated using the SCS unit hydrograph equation and the travel time in the ditch was calculated using manning's equation for open channel flow. Initial abstraction (mm) was calculated as 0.2 of Storage (potential maximum retention, a measure of the ability of a watershed to retain storm precipitation). Toe seepage was assumed based on baseflow calculations to nearby streams and will be refined through modelling to support the environmental assessment.

As further discussed in the 2019 Hydrology Baseline Report (Stantec 2020a), the Intensity-Duration-Frequency curves developed for the Stephenville climate station (ECCC ID 8403820) were selected to represent precipitation at the site. The Stephenville Intensity-Duration-Frequency curves were developed based on 48 years of data (1967 – 2017), and have been adjusted to account for the effects of climate change for the 2011-2040 time horizon (2020s) for the IPCC RCP4.5 emissions scenario. The average increase of IDF rainfall amounts associated with the various projections are approximately 10% for the 2020s (CRA 2015). In the model, the storms were distributed using a 10-minute timestep over 24 hours based on the SCS Type II distribution.

Snow melt was estimated using an energy balance approach for melt during rainy periods (USDA 2004). Inputs to the calculations included average daily air temperature, wind spend, rainfall for the month of analysis. The SCS curve number was based on a rain on snow melt event assuming the pile was covered with snow and ice. The SCS curve number was assigned based on the proportion of precipitation plus snow melt for the month of April, as April corresponds to the month of the greatest amount of snow melt observed in the Stephenville climate normal record.

Sizing of water management infrastructure was completed in Microsoft Excel using theoretical relationships of geometry and flow. The diameter of low-level outlet structure of the water management pond was sized based on an orifice equation. The capacity of the ditches was estimated using the Manning's equation and the associated catchment for each significant change in ditch grade. The required dimensions of the water management pond emergency spillways were sized using a broad crested weir equation.

Water management pond water quality design was dependent upon the minimum size and specific gravity of the sediment / precipitate particle needed to be removed, outflow rate from the water management pond and design event runoff volume. Sediment pond detention time was based on the settling of a 5.0×10^{-3} mm diameter sediment particle to settle a minimum of 1 m depth. To meet this requirement, the relationship (Simon, Li & Associates Inc 1982) between pond surface area, Q10 Yr pond outflow rate and sediment particle settling velocity was satisfied. The particle settling velocity was estimated using Stoke's Law and is dependent upon water temperature, particle size, and specific gravity of the particle. The thickness of ice was estimated using Stefan's Equation (MOE 2003) to determine inactive storage volumes in water management ponds.

The design of the water management ponds accounted for climate change, ice thickness during the cold season, operating water levels, inactive storage to promote settling, and freeboard.

4.2 HYDROLOGIC CONDITIONS

Total precipitation for the 1:10 year,1:100 year, and 1:200 year storm events are 100.7 mm, 183.4 mm, and 198.6 mm, respectively. The 1:100 year and 1:200 year storms include the maximum daily snow melt for the month of April of 38.6 mm/d. Snowmelt was not added to the 1:10 year storm, as snowmelt was not considered to contribute to total suspended solids loading in the ponds and the 1:10 year storm was used to size the required sedimentation volume. It was assumed that approximately 50% of snow melt and 72% of rainfall would runoff and contribute to the ponds. Based on these assumptions, the resultant curve number of 85 is within the expected curve number range of 71 and 96 of total precipitation for coarse aggregate material (USDA 1986). The design runoff condition of the waste rock pile assumes that layers of ice within the snowpack limit infiltration into the pile during winter and result in additional runoff during winter melt conditions.

Table 4.1 presents the hydrologic model input values used to simulate each Project component hydrological characteristics. The predicted peak flows and runoff volumes from each area of the Project are presented in Table 4.2 for 1:10 year,1:100 year, and 1:200 year storm events. The model assumed an Initial Abstraction of 5 mm. Catchment areas do not include the areas of the pond and conveyance ditches. Water Management Pond IDs and Project areas are noted on Figure 1.1. and 2.1.

Table 4.1 Hydrologic Inputs

Sediment Pond ID	Area of the Project	Catchment Area (m^2)	Time Lag (min)		Toe Seepage (m^3/day)
			Pile	Ditch	
MA-SP-01A	LGO	164,940	14.9	5.7	131
MA-SP-01B	Overburden Stockpile	289,418	20.9	4.3	230
MA-SP-01C	Waste Rock Pile	220,350	25.7	2.3	175
MA-SP-02	Waste Rock Pile	388,120	29.1	2.5	308
MA-SP-03	Waste Rock Pile	302,385	15.2	2.2	240
MA-SP-04	Waste Rock Pile	518,280	15.2	3.7	412
MA-SP-04	Topsoil Stockpile	40,100	2.2	1.0	32
LP-SP-01A	LGO	115,080	2.4	2.8	91
LP-SP-01B	Waste Rock Pile	290,770	34.8	5.2	231
LP-SP-02A	Waste Rock Pile	471,100	35.3	3.1	374
LP-SP-02B	Waste Rock Pile	145,000	36.0	2.8	115

Table 4.1 Hydrologic Inputs

Sediment Pond ID	Area of the Project	Catchment Area (m^2)	Time Lag (min)		Toe Seepage (m^3/day)
			Pile	Ditch	
LP-SP-03A	Waste Rock Pile	444,700	47.3	2.1	353
LP-SP-03B	Waste Rock	224,540	34.8	2.9	178
LP-SP-03B	Topsoil Stockpile	45,150	1.9	1.2	36
LP-SP-03C	Waste Rock Pile	37,570	10.3	0.6	30
LP-SP-04	Overburden Stockpile	104,855	11.4	1.9	83
Note:					

Table 4.2 Hydrologic Outputs

Hydrologic Element	1:10 Yr Peak Discharge m^3/s	1:10 Yr Volume (1000 m^3)	1:100 Yr Peak Discharge m^3/s	1:100 Yr Volume (1000 m^3)	1:200 Yr Peak Discharge m^3/s	1:200 Yr Volume (1000 m^3)
MA-SP-01A	1.2	6.5	3.3	16.7	3.7	18.8
MA-SP-01B	1.8	11.5	5.2	29.3	5.9	33
MA-SP-01C	1.2	8.7	3.7	22.3	4.1	25.1
MA-SP-02	2	15.4	5.8	39.4	6.6	44.2
MA-SP-03	2.2	12	6.4	30.7	7.2	34.5
MA-SP-04	3.7	18.4	11.5	52.6	12.1	59.9
MA-SP-05	6	27.5	1.3	4.1	19.6	79.2
LP-SP-01A	1.2	4.5	3.5	11.6	3.9	13
LP-SP-01B	1.4	11.5	3.9	29.5	4.5	33.1
LP-SP-02A	2.2	17.5	6.4	46.6	7.2	52.5
LP-SP-02B	0.7	5.7	2	14.7	2.2	16.5
LP-SP-03A	1.7	16.9	5	44.4	5.7	50
LP-SP-03B	1.1	6	3.1	22.7	3.5	26.1
LP-SP-03C	0.3	1.5	0.9	3.8	1	4.3
LP-SP-04	0.8	4.2	2.3	10.6	2.7	12
LP-SP-05	4.5	20.6	12.9	52.7	14.7	59.3

4.3 WATER QUALITY

The water management ponds were sized to settle approximately 5 micron and coarser particles in the 1:10 year design flow. The analysis assumed that a particle has to settle approximately 1 m from the water surface to descend below the submerged outlet invert and thus have reached the effective sediment trapping depth. The assumed settling velocity of the particles was 2×10^{-5} m/s (assuming the temperature of the fluid in the pond is close to freezing). Given a minimum vertical settling zone of 1 m, it will take 14 hours for a particle to reach the trapped sediment zone below the outlet invert.

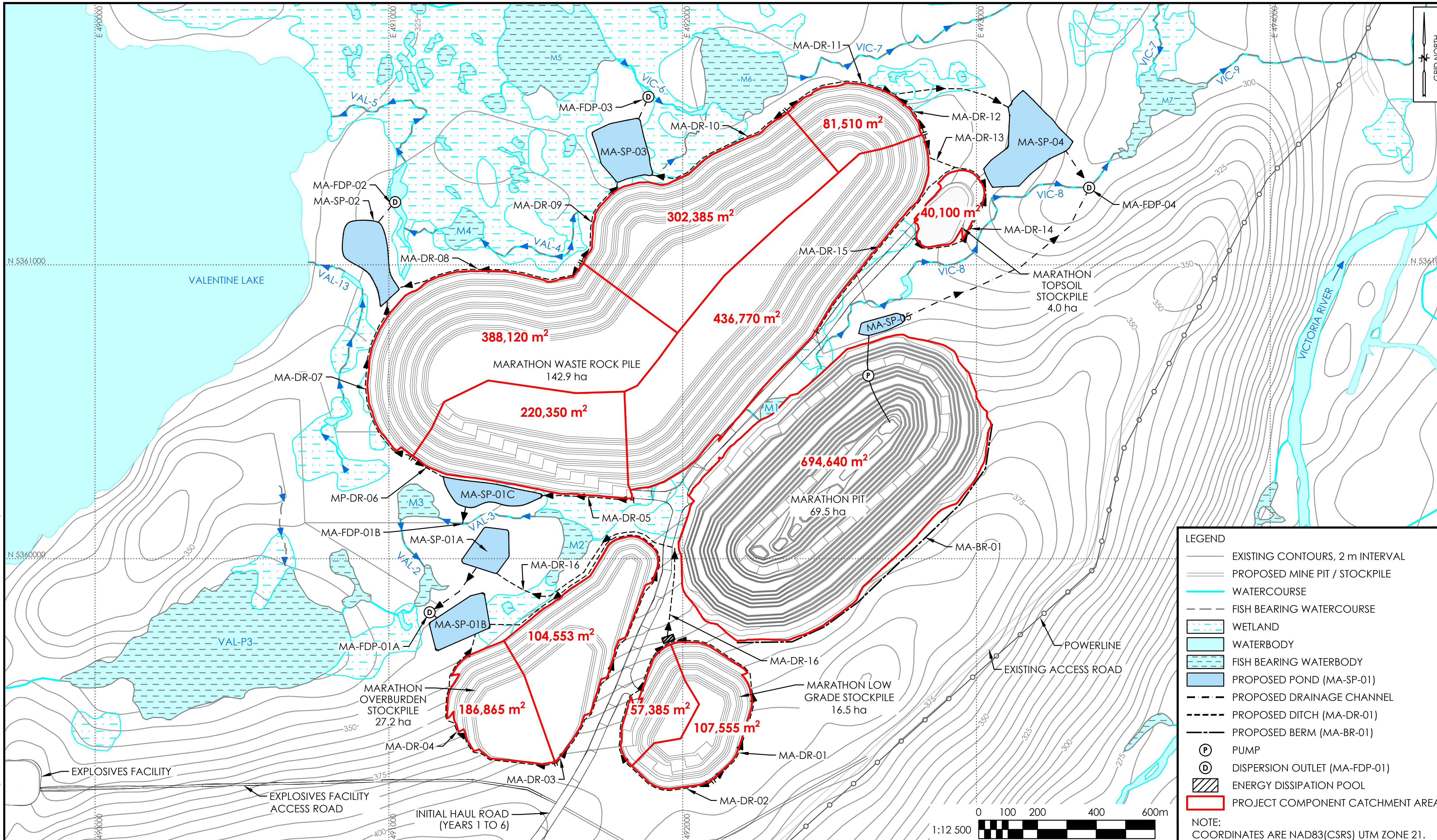
The invert elevation of the orifice pipe is set to provide an adequate pond volume for settling of the 1:10 year flood volume. However, it is recommended that the size analysis of the finer particles to be settled in the ponds should be tested to confirm water treatment requirements.

4.4 DRAINAGE ANALYSIS

The Project has a total of 12 FDPs. There is a total of five final discharge locations at the Marathon Complex that drain ultimately to the Victoria River either via Valentine Lake or direct tributaries to the river. There are five final discharge locations at the Leprechaun Complex that ultimately drain to Victoria Lake Reservoir, either directly to the lake or through tributaries. The Processing Plant and TMF Complex has an additional two final discharge locations that flow to Victoria Lake Reservoir, this includes the TMF effluent pipeline to Victoria Lake Reservoir and the processing area water management pond discharge to a tributary of Victoria Lake Reservoir.

The Project was designed to maintain predevelopment drainage conditions as close as possible throughout the phases of mine life. Water management ponds were designed to drain to pre-development catchments, where possible.

Catchment areas for the Project phases of construction are presented in Figures 4.1 – 4.6 based on the available Project LiDAR (Aethon 2019).



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1. EXISTING CONTOURS AND PROPOSED INFRASTRUCTURE: AUSENCO; PROJECT No. 104878-01; DRAWING No. 104878-0000-G-001; 2020-05-11; PRELIMINARY.
2. WATERCOURSES, WATERBODIES & WETLANDS: CANVEC DATABASE FROM NATURAL RESOURCES CANADA.
3. FISH BEARING WATERCOURSES AND WATERBODIES: SURVEYED FISH BEARING OR HAS CONNECTIVITY TO FISH BEARING WATER (STANTEC 2012, 2019, 2020).

MARATHON COMPLEX WATER MANAGEMENT CATCHMENT AREAS DURING OPERATION

WATER MANAGEMENT PLAN

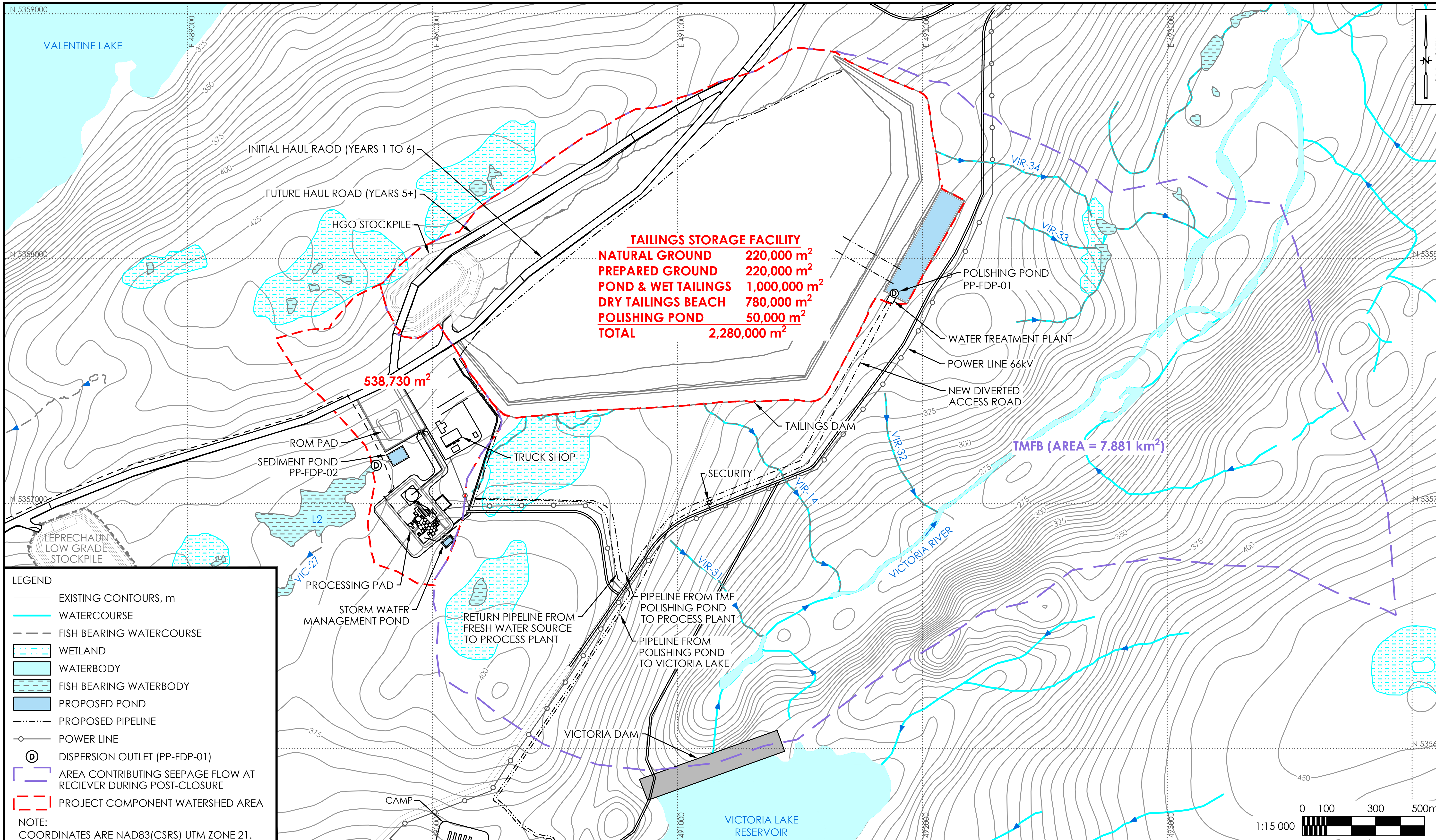
VALENTINE GOLD PROJECT, NL

Client: MARATHON GOLD CORP

Job No.:	121416408
Scale:	1 : 12 500
Date:	28-SEP-2020
Dwn. By:	JL
App'd By:	RJ

Fig. No.: 4.1

Stantec



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PROCESSING PLANT & TMF COMPLEX WATER MANAGEMENT DESIGN DURING OPERATION

WATER MANAGEMENT PLAN

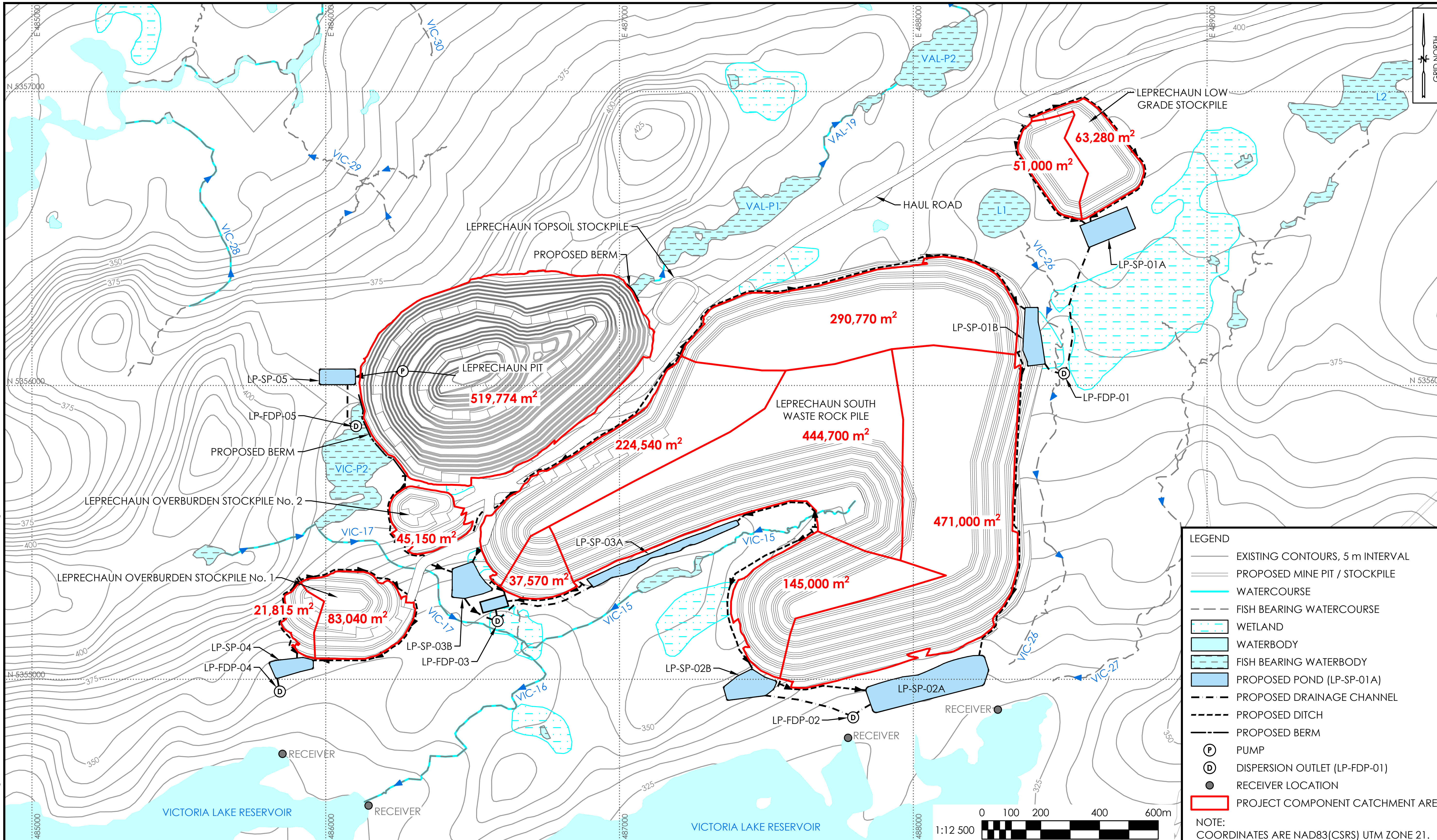
VALENTINE GOLD PROJECT, NL

Client: MARATHON GOLD CORP

Job No.:	121416408
Scale:	1 : 15 000
Date:	28-SEP-2020
Dwn. By:	JL
App'd By:	RJ

Fig. No.: 4.2

Stantec



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LEPRECHAUN COMPLEX WATER MANAGEMENT DESIGN DURING OPERATION

WATER MANAGEMENT PLAN

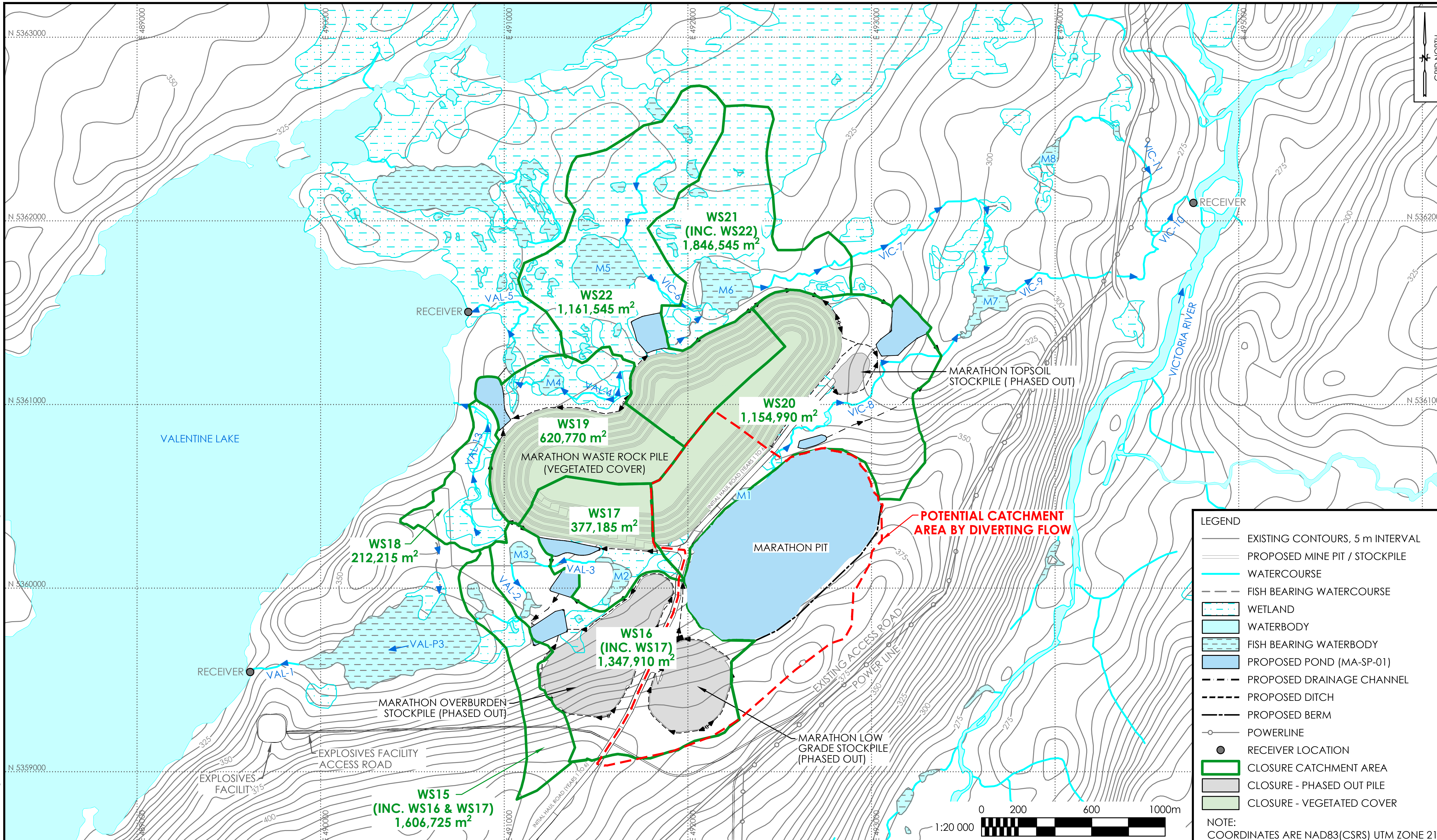
VALENTINE GOLD PROJECT, NL

Client: MARATHON GOLD CORP

Job No.:	121416408
Scale:	1 : 12 500
Date:	28-SEP-2020
Dwn. By:	JL
App'd By:	RJ

Fig. No.: 4.3

Stantec



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3. FISH BEARING WATERCOURSES AND WATERBODIES: SURVEYED FISH BEARING OR HAS CONNECTIVITY TO FISH BEARING WATER (STANTEC 2012, 2019, 2020).

MARATHON COMPLEX WATER MANAGEMENT CATCHMENT AREAS DURING CLOSURE

WATER MANAGEMENT PLAN

VALENTINE GOLD PROJECT, NL

Client: MARATHON GOLD CORP

Job No.:	121416408
Scale:	1 : 20 000
Date:	28-SEP-2020
Dwn. By:	JL
App'd By:	RJ
Fig. No.:	4.4

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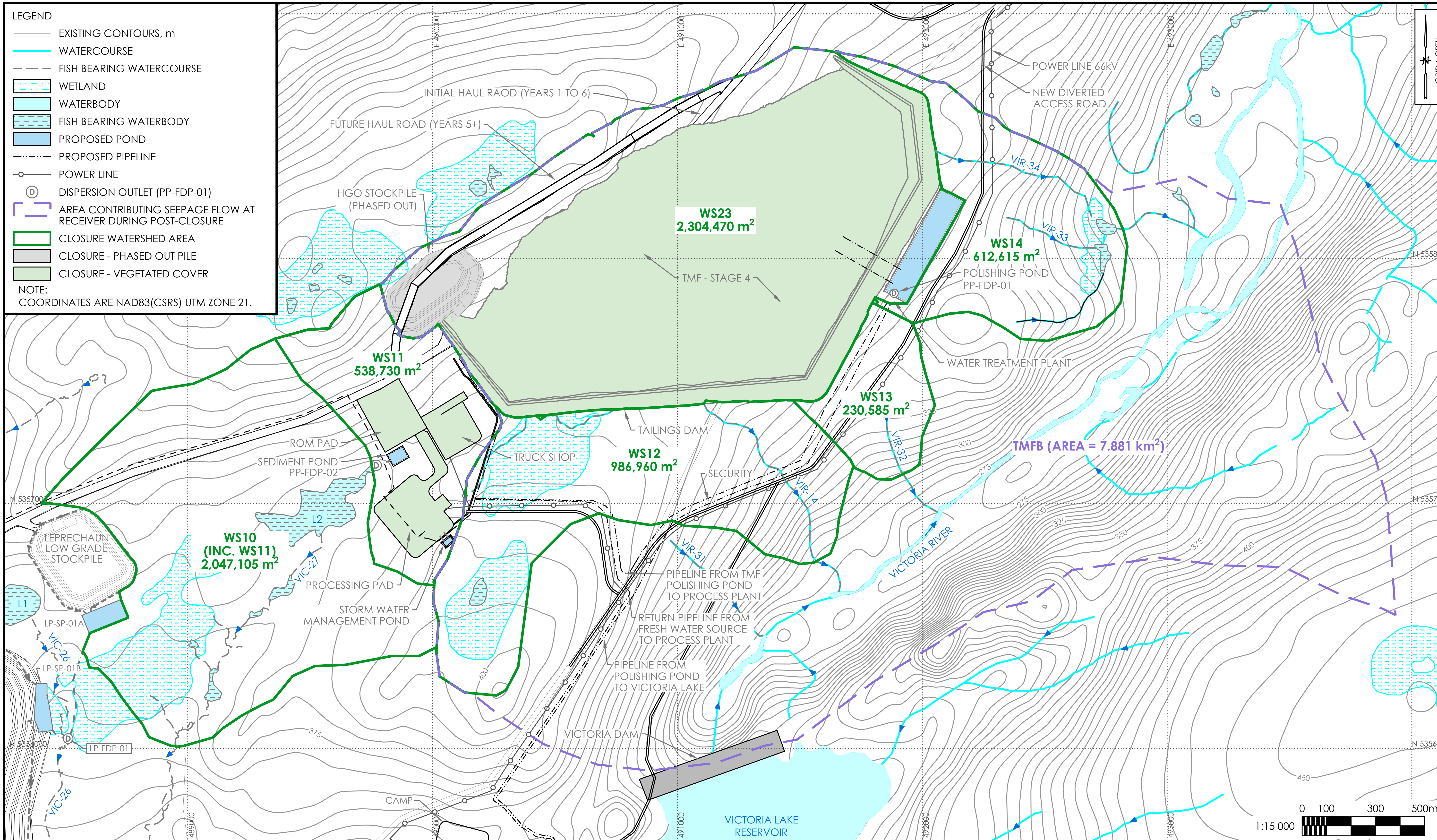

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PROCESSING PLANT & TMF COMPLEX WATER MANAGEMENT DURING CLOSURE

WATER MANAGEMENT PLAN
VALENTINE GOLD PROJECT, NL

Client: MARATHON GOLD CORP

Job No.:	121416408
Scale:	1 : 15 000
Date:	28-SEP-2020
Dwn. By:	JL
App'd By:	RJ

Fig. No.: 4.5

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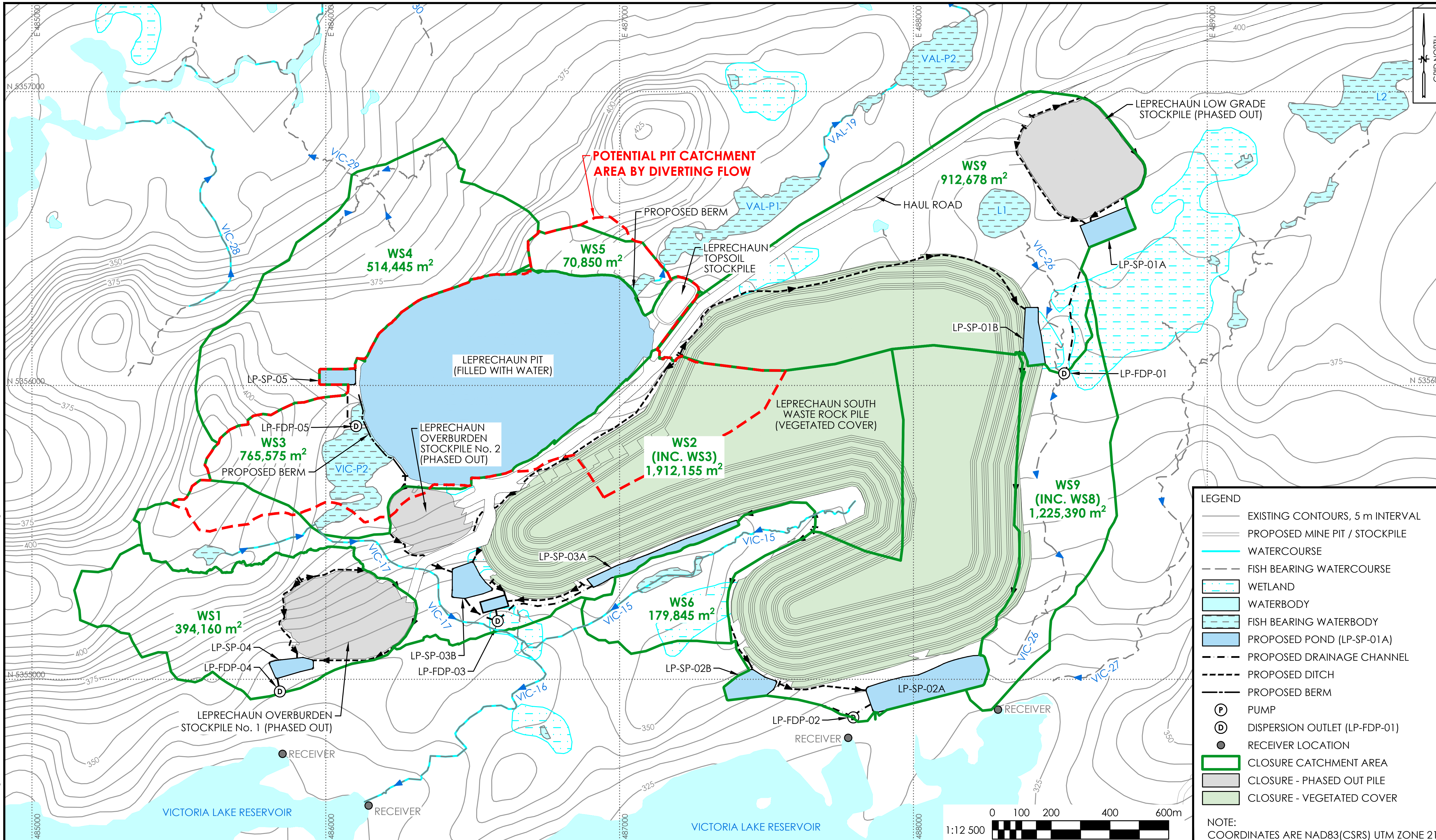

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LEPRECHAUN COMPLEX WATER MANAGEMENT DESIGN DURING CLOSURE
WATER MANAGEMENT PLAN
VALENTINE GOLD PROJECT, NL

Client: MARATHON GOLD CORP

Job No.:	121416408
Scale:	1 : 12 500
Date:	28-SEP-2020
Dwn. By:	JL
App'd By:	RJ

Fig. No.: 4.6

Stantec

5.0 WATER MANAGEMENT DESIGN

The Water Management Design is provided in the next sections at pre-feasibility level. The design will be progressed to feasibility level and detailed design stages. Design will be subject to change as additional information becomes available such as, geotechnical borehole/test pitting programs, regulator consultation, refinements in design of facilities, water quality modelling and detailed surveying.

5.1 CONSTRUCTION

The primary water management activity during construction will be erosion and sediment control (ESC) measures and mine excavation dewatering. ESC measures will be required for various construction phase activities outlined in Section 2.1 and include clearing, stripping and grubbing of vegetation, excavation and storage of topsoil and overburden, blasting and removal of mine rock and ore, dewatering of the pits and excavations. The primary water management activities during construction of the process plant are expected to include collection, treatment, and discharge of surface runoff from the construction area and surface runoff and groundwater inflow to foundation excavations. Other construction activities include construction of water management infrastructure, road construction, borrow area development and operation, and preparation of surfaces for major Project facilities.

ESC will be implemented to reduce environmental impacts involving earthwork activities during the development of the Project. The four basic principles to be adopted in implementation of ESC measures include:

- Direct runoff away from active work areas before construction commences, reducing the volume of sediment-laden water to be managed
- Limit the amount and timing of exposed soil to reduce the potential for erosion
- Control sediment-laden runoff leaving the site, following ESC measures put in place for the construction of the Project
- Protect sensitive receptors from sediment-laden runoff by directing untreated runoff away from these areas

Sensitive receptors on and adjacent to the site will require protection from sediment-laden runoff generated during site development activities. The most sensitive receptors, based on their proximity to active work areas where land disturbance will be encountered, include Victoria River, Valentine Lake, and Victoria Lake Reservoir and the associated tributaries and ponds. Many of these waterbodies and watercourses were identified as fish habitat (Stantec 2020d).

Standard sediment control features will be used during construction, including installation of silt fencing and construction of diversion ditches and berms to divert and/or collect surface water runoff. During construction, water from construction areas will be directed to temporary sediment ponds, energy dissipation pools, sediment traps or sediment filter bags or proposed operational water management ponds constructed early during construction. Water in the temporary sediment ponds will either be discharged overland or directly local receivers if water quality meets regulatory standards.

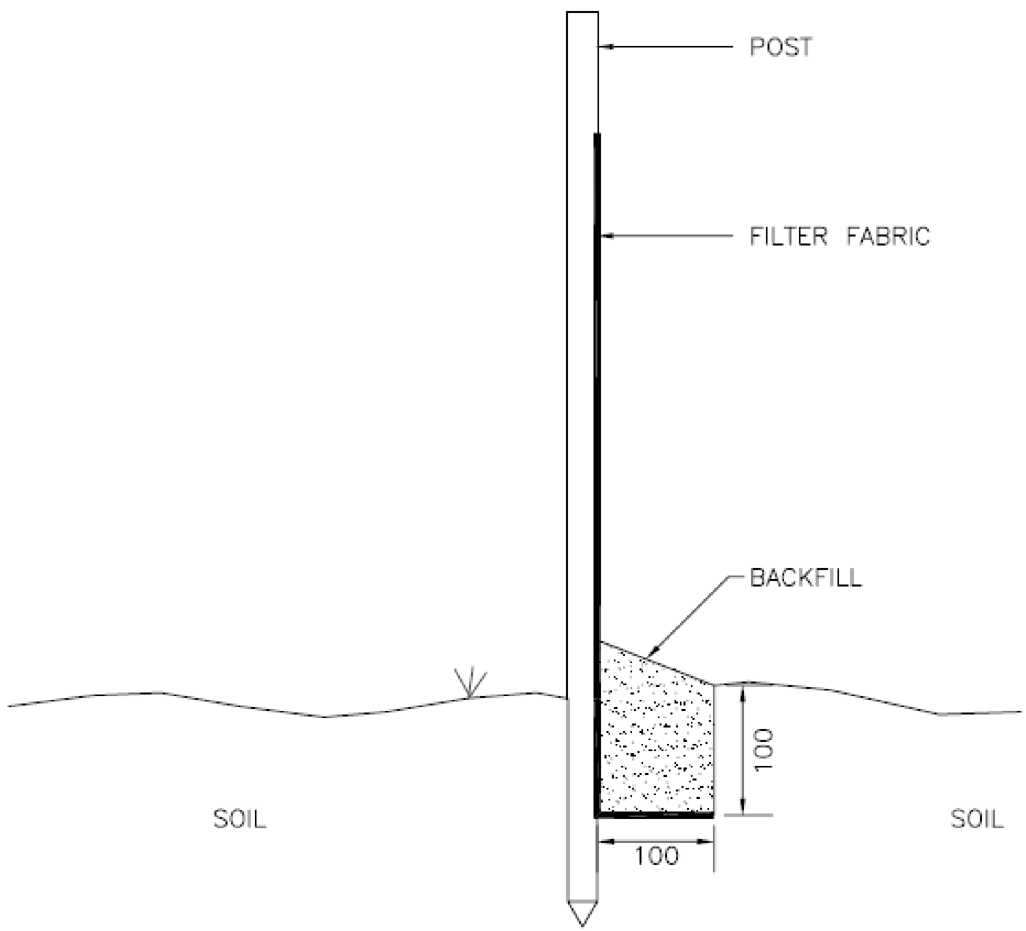
During construction, parameters of potential concern (POPC) in runoff are expected to be total suspended solids (TSS) and potentially elevated metal concentrations resulting from the storage of topsoil, overburden and waste rock.

Recommended water management best practices during construction are presented in Table 5.1 to manage surface runoff and reduce the erosion and sedimentation potential.

Table 5.1 Recommended Water Management Best Practices during Construction

General	Construction of perimeter (or ring) ditches around the footprint of Project facilities areas prior to clearing and grubbing on the site. The ditches will be constructed to collect and treat sediment-laden runoff inside the work area and divert runoff outside the work area offsite. Placement of flow checks consisting of clear stone in ditches to reduce the velocity of flow and deposit sediment load. Construct in the dry, dewatering areas prior to construction or installing temporary flow diversion measures to reduce the amount of sediment. Divert sediment-laden runoff to collection ponds or water management ponds for treatment prior to discharge. Periodic removal of excess sediment from the collection ponds may be required to reinstate the design storage capacity. During topsoil and overburden removal, surface water runoff and seepage will be collected in excavation sumps and pumped to either temporary water management ponds or discharged to the environment through filter bags.
TMF	Construction of the polishing pond in advance of the TMF dam to act as a sedimentation basin for upgradient construction of the TMF.
Piles	Topsoil, overburden, and bedrock removed for construction will be stored for rehabilitation and closure purposes in the designated pile areas at each complex. Prior to development of the waste rock piles, perimeter ditches and water collection ponds will be constructed to collect and store surface runoff. The drainage ditches will be constructed to drain by gravity to the sedimentation ponds, where practicable. In low areas where gravity flow to the sedimentation ponds is not practical, sumps will be constructed to collect water and pump it to the water management ponds. Water quality in the ponds will be monitored, treated via sedimentation as necessary, and discharged to the environment once water quality meets regulatory criteria.
Open Pits	During clearing and grubbing activities associated with the open pits, surface water runoff and seepage will be collected in excavation sumps and pumped to either temporary water management ponds (and discharged to the environment if discharge criteria are met) or further treated using additional ponds and/or filter bags prior to discharge. The footprint of the Leprechaun pit overlays VICP2 that flows both to the north and the south from the center of the pit, as the pit is located at a natural drainage divide. These headwaters including VICP2 will be dewatered and flow discharge further downstream. The footprint of the Marathon pit overlaps the existing Pond M1. Pond M1 and downstream reach will be dewatered as the proximity to the pit will result in loss of flows during active pit dewatering.
Access and Haul Roads	Temporarily divert flow in the watercourses, to replace existing bridges and culverts in the site access road and construct the haul road. Water will be discharged to a vegetated area through a perforated PVC pipe, located more than 60 m from any watercourse, or alternatively into a filter bag. Additional cross culverts under the site access and haul road may be installed as localized drainage dictates.

Implementation and maintenance of the ESC measures will be monitored on a daily basis and in more detail prior to and immediately following a precipitation event of 25 mm or more. ESC measures will be put in place to ensure that liquid effluent limits are met in the receiving watercourse. Maintenance and monitoring of the ESC measures are the responsibility of the contractor. Typical details for a silt fence installation, energy dissipation pool and sediment trap and provided in Figures 5.1 to 5.3, respectively.



INSTALLATION OF GEOTEXTILE SILT FENCE

1. EXCAVATE A 100mm X 100mm TRENCH IN A CRESCENT SHAPE ACROSS THE FLOW PATH WITH ENDS POINTING UPSLOPE.
2. DRIVE STURDY STAKES, SPACED 3000mm APART, INTO THE GROUND ALONG THE DOWNSLOPE SIDE OF THE TRENCH.
3. INSTALL THE FILTER FABRIC FROM A CONTINUOUS ROLL AND CUT TO REQUIRED LENGTH. THE FILTER FABRIC SHOULD BE STAPLED TO THE UPSTREAM SIDE OF THE STAKES, EXTENDING THE BOTTOM 200mm INTO THE TRENCH
4. BACK FILL AND COMPACT THE SOIL IN THE TRENCH OVER THE FILTER FABRIC.
5. CONTRACTOR TO MAINTAIN FENCE FOR THE DURATION OF THE PROJECT AND REMOVE AT THE COMPLETION OF THE PROJECT.

Figure 5.1 Installation of Geotextile Silt Fence (Government of NL 2004)

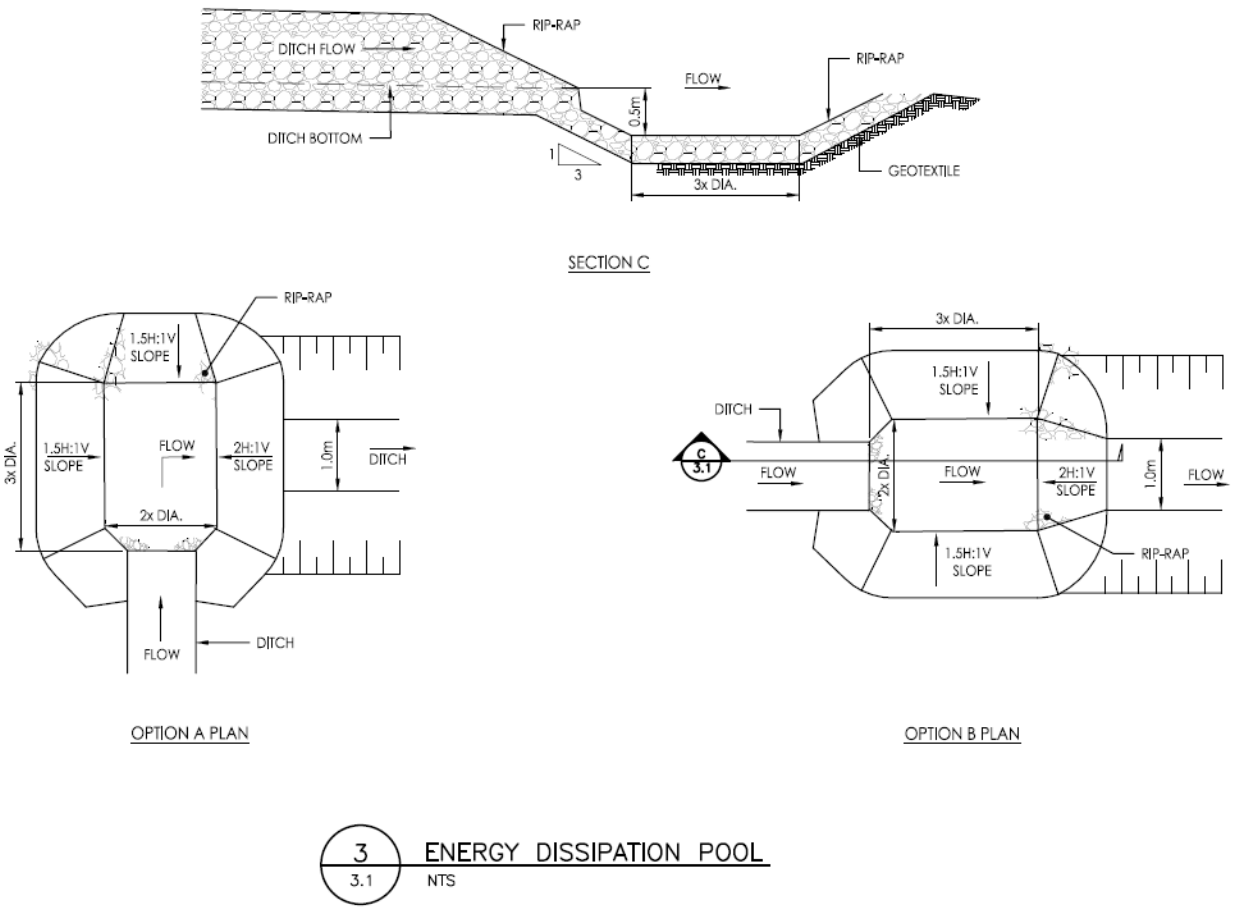


Figure 5.2 Energy Dissipation Pool Typical Detail

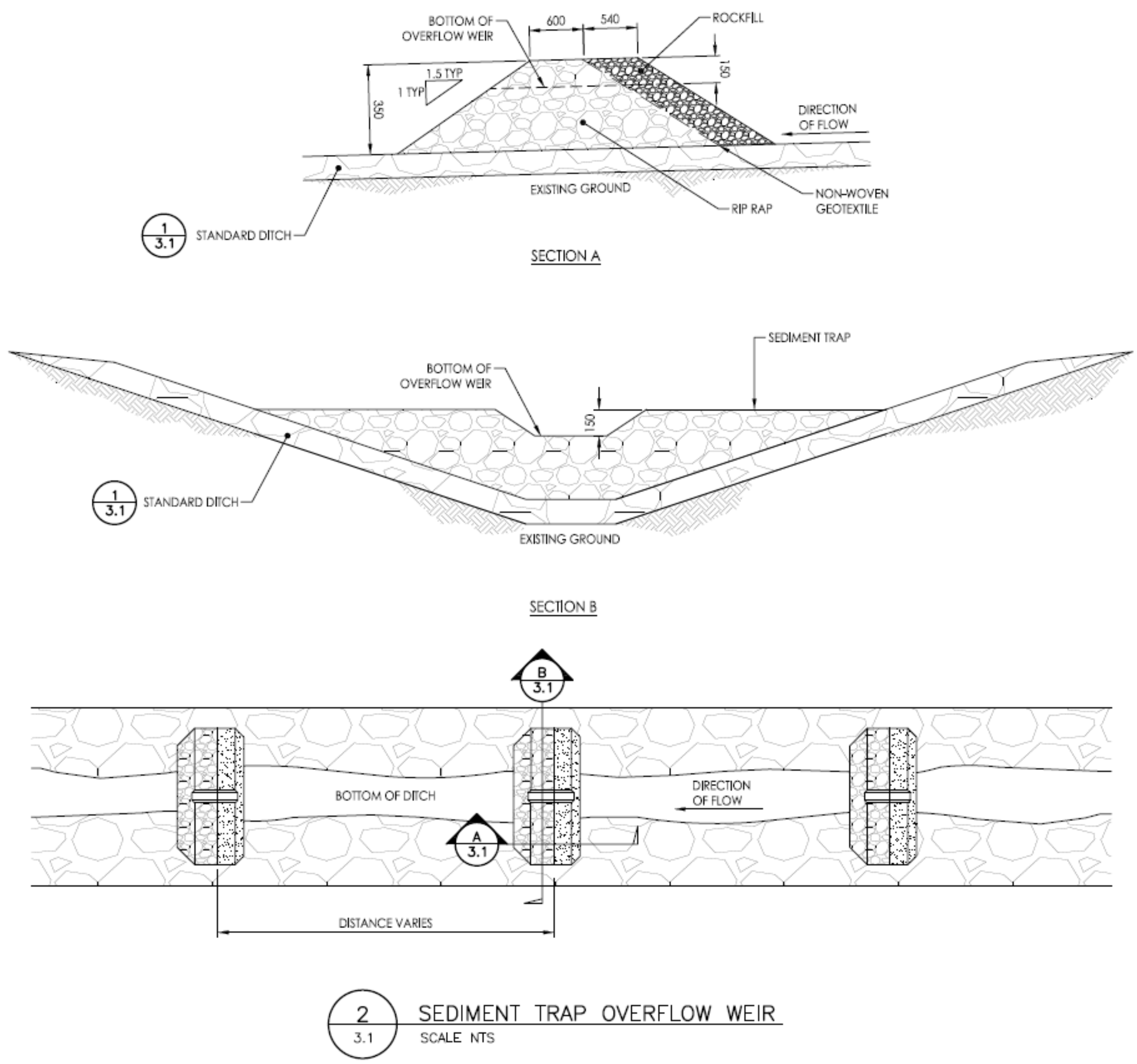


Figure 5.3 Sediment Trap Overflow Weir Typical Detail

5.2 OPERATION

Water management functions independently with decentralized treatment and control in each complex. To reduce the mine water inventory, non-contact runoff is proposed to be diverted using perimeter berms to allow runoff to naturally flow offsite. The water management design is presented in Figures 4.1 to 4.6 for the Marathon complex, Processing Plant and TMF Complex, and for the Leprechaun Complex, respectively.

The water management design diverts non-contact water from the mine facilities natural water drainage areas, where possible. Diversion of surface flows using berms constructed around the crest of open pits or up-gradient of waste rock piles and other developed areas to reduce the contact water inventory. Where possible, water collected in pits or in the water management ponds will be used for other purposes on site rather than discharged to the environment.

A buffer of approximately 15 m was maintained from fish bearing watercourses for the water management design. Flow to fish bearing streams and bogs were maintained by draining mine site components to pre-development streams and bogs and designing low flow outlets from ponds to receivers to augment baseflow. Flow to these fish bearing watercourses will be maintained by targeting pre-development flows, where feasible. MDMER limits will be met at FDPs prior to release.

The water management design includes 17 water management ponds. MDMER limits will be met at FDPs downstream of the prior to release to the receiver. Effluent will be released slowly to enhance baseflow augmentation and reduce the potential for downstream scour and erosion.

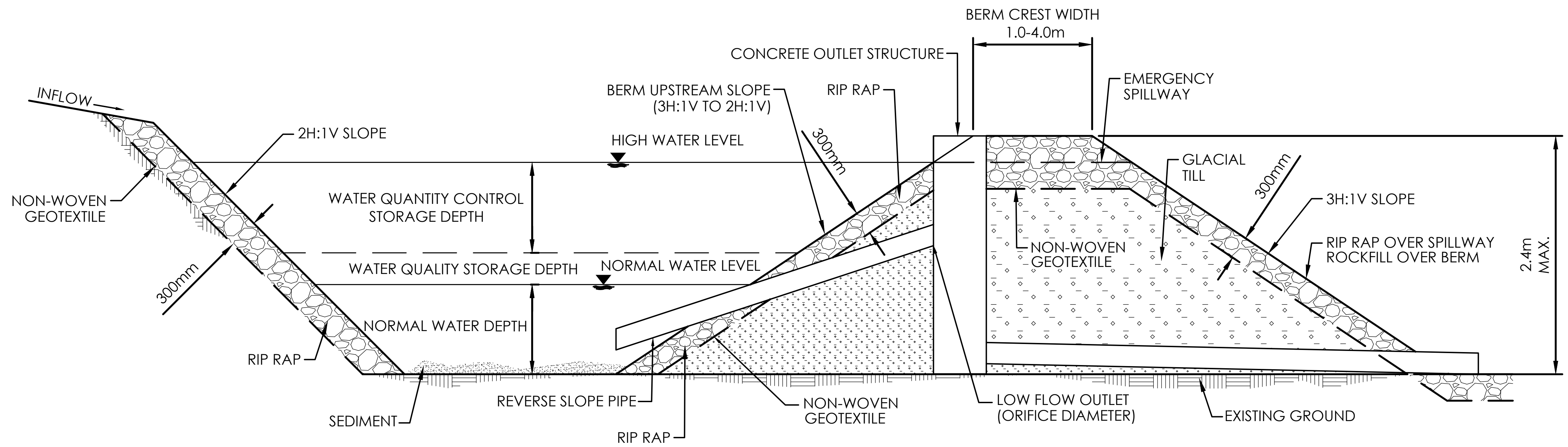
5.2.1 Ditch and Water Management Pond Design

Water management infrastructure, exclusive of pond outlet infrastructure and discharge channels, is summarized in Table 5.2 and Table 5.3. Catchment areas for mine site components were delineated in AutoCAD based on the available Project LiDAR (Aethon 2019) and have been included on Figures 4.1 to 4.6.

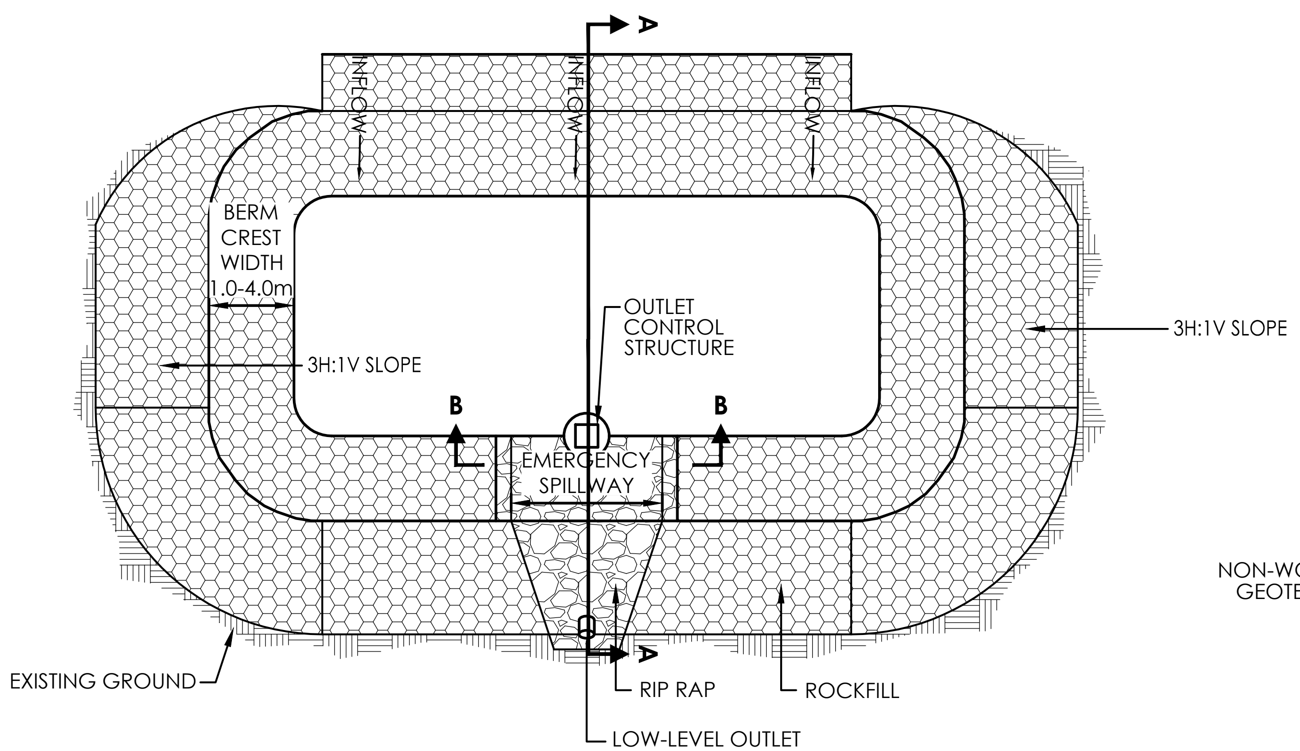
Water management ponds provide on-site storage of runoff with controlled releases permitted after appropriate residence time for particulate settling. All permanent pools will be excavated below grade, thus reducing the total berm height required to achieve the storage while improving dam safety. Water management ponds will include multi-stage outlet control through a low-level reverse flow submerged outlet and a spillway.

Water management pond embankments will be constructed out of locally sourced glacial till. Erosion protection will be provided through riprap lining of the berm and spillway and a scour pad at the toe of slope of spillways. A geotextile filter layer will be placed between materials to reduce the opportunity for piping. Where topography allows, the crest of the berm will be 4 m wide and have 3H:1V embankment slopes to allow for light vehicle access on top of the berm to facilitate maintenance and monitoring activities. Where topography is more constrained, a 1 m berm crest will be constructed with a 2H:1V embankment upstream slope. For the latter option, it was assumed that an access road would be constructed to the toe of the embankment to allow for maintenance access using a tractor or excavator on the 3H:1V embankment downstream slope. Typical water management pond design is presented in Figure 5.4.

Ditches will be constructed along the perimeter of piles to convey the 1:100 AEP surface runoff and toe drainage to water management ponds for water quality and quantity control. Ditches will be designed to convey gravity flow to reduce operational costs that would results from pumping. Ditches will follow a standard trapezoidal geometry with a maximum 2H:1V side slope tied into existing grade to reduce cost of construction and maintaining a minimum of 20 cm freeboard. Materials excavated from ditches will be sidecast, and berms constructed of the sidecast material to reduce cost of construction. Berms will be constructed on the outside bank of the ditches. No berms will be constructed between the ditch and its source stockpile. Ditches will be lined with rip-rap for erosion protection. In areas with ditch gradients steeper than 8%, sediment traps (i.e., check dams) will be installed at a spacing of 200 m per ditch grade % to provide energy dissipation and reduce erosional flow velocities in the ditch. For the same purpose, energy dissipation pools will be installed at the change in ditch gradient from slopes of 10% or higher to shallower slopes. Ditches are proposed in three general size categories to account for increases in collection drainage over the longer lengths of ditches. Figure 5.5 presents the typical section views for the three ditch size categories. Table 5.2 summarized details on water management pond and ditch design.



SECTION A



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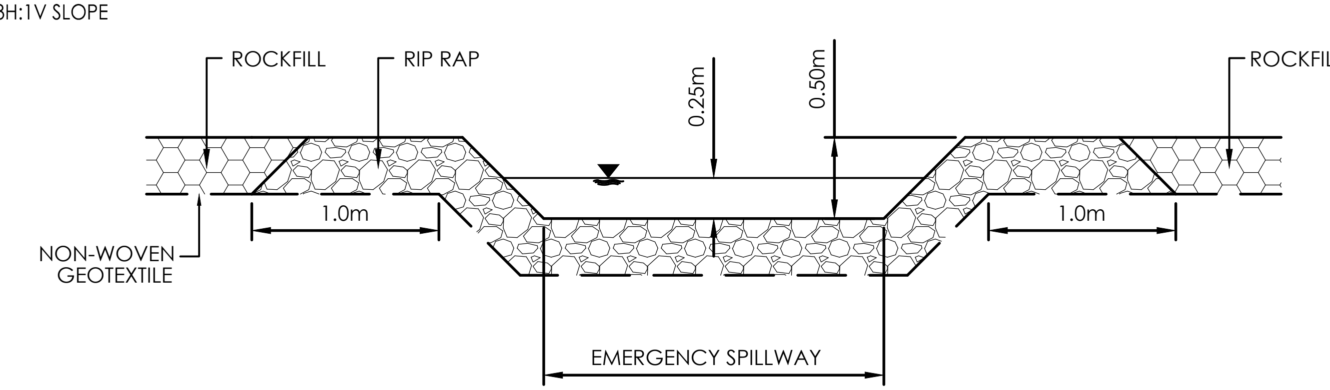

SECTION B

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Reference:

1. EXISTING CONTOURS AND PROPOSED INFRASTRUCTURE: AUSENCO; PROJECT No. 104878-01; DRAWING No. 104878-0000-G-001; 2020-05-11; PRELIMINARY.
2. WATERCOURSES, WATERBODIES & WETLANDS: CANVEC DATABASE FROM NATURAL RESOURCES CANADA.
3. FISH BEARING WATERCOURSES AND WATERBODIES: SURVEYED FISH BEARING OR HAS CONNECTIVITY TO FISH BEARING WATER (STANTEC 2012, 2019, 2020).

WATER MANAGEMENT POND TYPICAL
WATER MANAGEMENT PLAN
VALENTINE GOLD PROJECT, NL

Client: MARATHON GOLD CORP

Job No.:	121416408
Scale:	1 : 12 500
Date:	28-SEP-2020
Dwn. By:	DH
App'd By:	RJ

Fig. No.: 5.4

Stantec

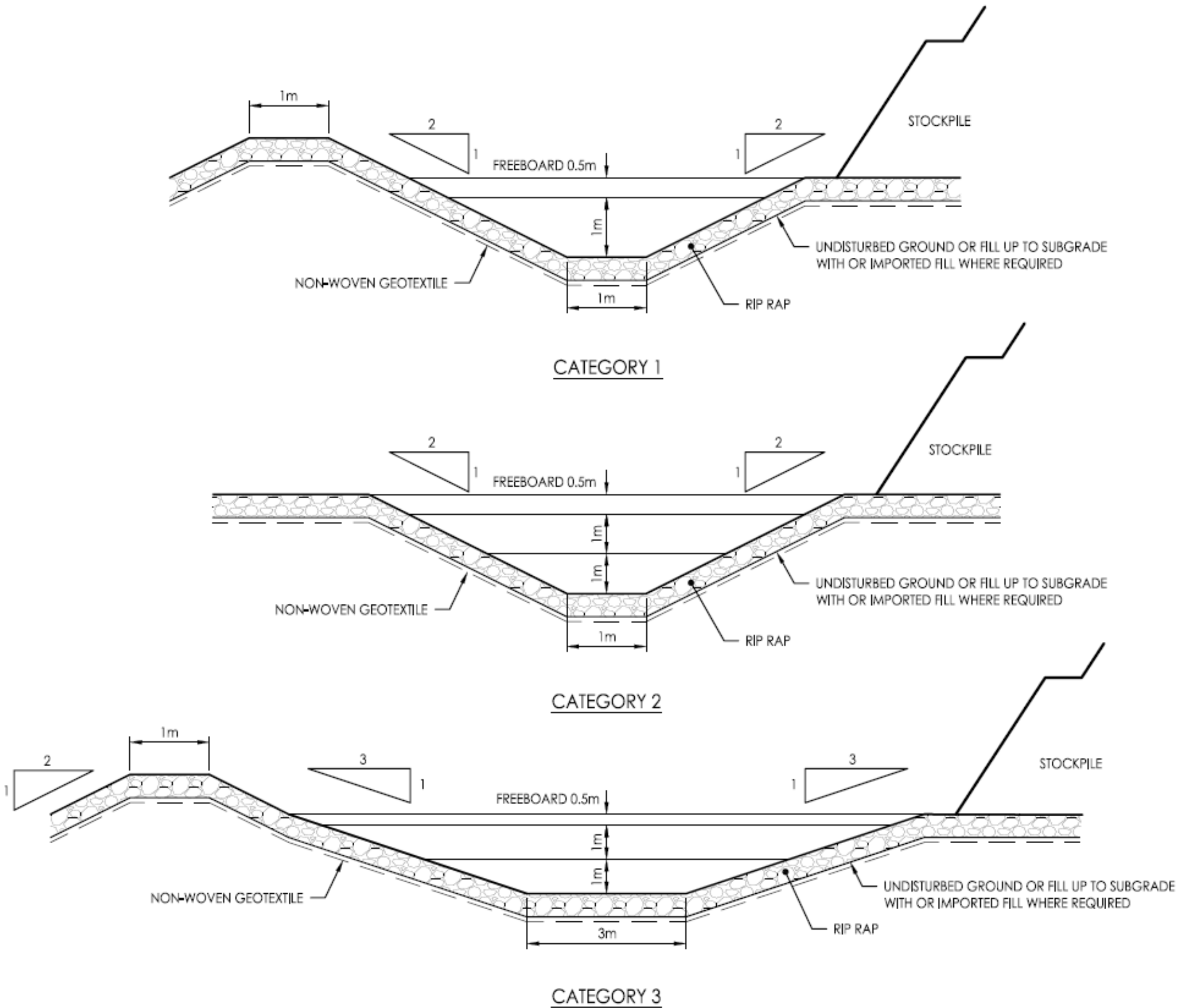


Figure 5.5 Ditch Typical

Table 5.2 Water Management Pond and Ditch Design Management Infrastructure

Mine Facility [Facility Area]	Ditch Run	Ditch Length (m)	Water Management Pond	Final Discharge Point	Discharge Location
Marathon Low Grade Stockpile [16.5 ha]	MA-DR-01	710	MA-SP-01A	MA-FDP-01A/B	Unnamed tributary that drains to Valentine Lake (VALP3)
	MA-DR-02	805			
	MA-DR-16	1165			
Marathon Overburden Stockpile [27.2 ha]	MA-DR-03	1515	MA-SP-01B		
	MA-DR-04	760			
Marathon Waste Rock Pile [142.9 ha]	MA-DR-05	330	MA-SP-01C		
	MA-DR-06	130			
	MA-DR-07	610	MA-SP-02	MA-FDP-02	Victoria Lake Reservoir
	MA-DR-08	655			
	MA-DR-09	310	MA-SP-03	MA-FDP-03	Wetland draining to Valentine Lake (Upgradient of M5)
	MA-DR-10	520			
	MA-DR-11	785	MA-SP-04	MA-FPD-04	Tributary to Victoria River (VIC8)
	MA-DR-12	160			
	MA-DR-13	315			
	MA-DR-15	365			
Marathon Topsoil Stockpile [4.0 ha]	MA-DR-14	735			
Marathon Pit [69.5ha]	MA-BR-01	1235	MA-SP-05		Tributary to Victoria River (VIC8)
Leprechaun Low Grade Stockpile [11.4 ha]	LP-DR-01	785	LP-SP-01A	LP-FDP-01	Unnamed tributary stream to Victoria Lake Reservoir (VIC-01)
	LP-DR-02	440			
Leprechaun Waste Rock Pile [161.5 ha]	LP-DR-03	1,370	LP-SP-01B		
	LP-DR-04	1,050	LP-SP-02A	LP-FDP-02	Victoria Lake Reservoir
	LP-DR-05	300			
	LP-DR-06	650	LP-SP-02B		
	LP-DR-07	345	LP-SP-03A	LP-FDP-03	Headwater stream that drains to Victoria Lake Reservoir (VIC17)
	LP-DR-08	270	LP-SP-03C		
	LP-DR-09	70			
	LP-DR-10	1,065	LP-SP-03B		
Leprechaun Topsoil Stockpile [4.5 ha]	LP-DR-11	495	LP-SP-03B		
Leprechaun Overburden Stockpile [10.5 ha]	LP-DR-12	325	LP-SP-04	LP-FDP-04	Unnamed tributary stream to Victoria Lake Reservoir
	LP-DR-13	885			
Leprechaun Pit [52 ha]	LP-BR-01		LP-SP-05	LP-FDP-05	VIC-P2
TMF	PP-PR-01			PP-FDP-01	Victoria Lake Reservoir
Process Plant Pad	PP-DR-01		PP-SP-01	PP-FDP-02	Victoria Lake Reservoir

As noted in Table 5.3, discharge channels were constructed at the outlet of water management ponds to combine effluent discharge points and associated MDMER monitoring requirements. Operational costs were reduced as flow was conveyed through additional ditching and grading rather than installing a pipe and pipeline.

Table 5.3 Sediment Pond Discharge Channel Lengths

Sediment Pond Name	Discharge Channel Length (m)	Sediment Pond Name	Discharge Channel Length (m)
MA-SP-01A	210	LP-SP-01A	445
MA-SP-01B	20	LP-SP-01B	70
MA-SP-01C	80	LP-SP-02A	60
MA-SP-02	35	LP-SP-02B	295
MA-SP-03	35	LP-SP-03A	350
MA-SP-04	145	LP-SP-03B	130
MA-SP-05	800	LP-SP-03C	30
---	---	LP-SP-04	35
---	---	LP-SP-05	20

Pond storage, geometry, and outlet configuration are summarized in Table 5.4. The inactive and 1:100 year active pond storage below the spillway are summarized for each sediment pond. Pond geometry includes the designed pond bottom elevation and berm crest elevation in addition to the pond width and length. Outlet configuration of the bottom draw pipes and associated orifice diameter needed to provide residence time and extended discharge attenuation and spillway width were also provided as these dimensions change for each sediment pond.

Pumps will be required to dewater the Marathon and Leprechaun pits. A pit dewatering pond was designed at a low-lying location adjacent to each pit. It was assumed that a pond volume of 11,190 m³ and 10,974 m³ for the Marathon and Leprechaun pits will be adequate to contain the pit dewatering rates based on the rates reported by Terrane (2019). Pit dewatering discharge directed to the pit dewatering ponds at the surface will be subsequently drained to pre-development catchments.

Table 5.4 Pond Storage, Geometry, Outlet Configuration

Sediment Pond Name	Inactive Pond Storage (m³)	Active Pond Storage (m³)	Total Pond Storage (m³)	Pond Bottom Elev. (m)	Pond Berm Crest Elev. (m)	Pond Width (m)	Pond Length (m)	Orifice Diameter (mm)	Spillway Base Width (m)
MA-SP-01A	12,400	13,500	25,900	*337.5*	*340.0*	*125.0*	*135.0*	300	2
MA-SP-01B	20,600	22,400	43,000	337.0	339.5	120.0	215.0	450	3
MA-SP-01C	17,100	19,300	36,400	338.0	340.5	90.0	330.0	300	2
MA-SP-02	26,160	28,400	54,560	326.0	328.5	270.0	160.0	450	4
MA-SP-03	27,600	29,600	57,200	326.0	328.5	170.0	200.0	450	4
MA-SP-04	44,700	47,600	92,300	312.0	314.5	250.0	300.0	450	6
MA-SP-05	5,070	4,100	9,170	330.5	333.0	165.0	55.0	450	10

Table 5.4 Pond Storage, Geometry, Outlet Configuration

Sediment Pond Name	Inactive Pond Storage (m^3)	Active Pond Storage (m^3)	Total Pond Storage (m^3)	Pond Bottom Elev. (m)	Pond Berm Crest Elev. (m)	Pond Width (m)	Pond Length (m)	Orifice Diameter (mm)	Spillway Base Width (m)
LP-SP-01A	9,570	11,600	21,170	377.0	379.5	75	160	300	2
LP-SP-01B	8,950	10,400	19,350	369.5	372.0	75	205	300	3
LP-SP-02A	30,000	45,000	75,000	326.0	328.5	115	420	450	4
LP-SP-02B	9,570	14,400	23,970	341.5	344.0	90	140	300	1
LP-SP-03A	10,200	13,500	23,700	352.0	354.5	35	550	450	3
LP-SP-03B	4,000	4,000	8,000	*347.0*	*349.0*	*40*	*100*	300	2
LP-SP-03C	9,570	3,800	13,370	349.0	351.5	110	120	300	1
LP-SP-04	4,790	7,200	11,990	338.5	341.0	60	145	300	2
LP-SP-05	4,390	2,400	8,890	335.5	338.0	60	130	450	8
PP-DR-01		3,000							

5.2.2 Processing Plant and TMF Complex

The tailings pond will collect direct precipitation, runoff from the tailings surface, water discharged from the mill with the tailings (Mine Years 1 to 9), and water pumped back from the seepage collection sumps around the facility. During the operation phase, water will be pumped from the tailings pond via a reclaim pump system for the operation of the processing plant. Excess runoff from the TMF is routed through a water treatment plant and polishing pond prior to discharge via a pipeline to Victoria Lake Reservoir. The pipeline extends into Victoria Lake Reservoir at the final discharge point PP-FDP-01. Clean make-up water required in the process plant will be supplied from Victoria Lake Reservoir. In Year 10, when tailings deposition is switched from the TMF to the Leprechaun pit, process water will continue to be supplemented by TMF reclaim water, in addition to the minimum of 8% freshwater make-up from Victoria Lake Reservoir.

Seepage collection ditches will be constructed at the downstream toe of the tailings dam. Seepage from the ditches will be directed to sump pits at various topographic low points around the dams; seepage and runoff collected in the sumps will be pumped back to the tailings pond. The tailings pond is designed to contain the Environmental Design Flood and Inflow Design flood. Excess water above the Environmental Design Flood in the polishing pond will spill through an emergency spillway and drain towards the Victoria River.

The process plant pad will be graded to allow surface runoff water to drain naturally to the internal network of collection swales and ditches sized to handle peak flow resulting from storm events. The pond will be designed to promote settling of solids and provide flow attenuation of peak storm events. The collection ditches will convey the water to a stormwater pond at 3,000 m^3 live capacity, west of the processing plant. The water in the water management pond will be pumped into the process water tank as make-up water and excess water will drain toward Victoria Lake Reservoir via a local tributary.

Raw freshwater will be pumped from Victoria Lake Reservoir to supply fire water, cooling water, gland water for pumps, reagent make-up, feed for potable water plant, and the freshwater make-up process water demand. Raw water for the process demand will be pumped from Victoria Lake Reservoir to the tanks and distributed to the required points in the plant and in addition supply the potable water treatment system. Demand for the process plant is 21 cubic metres per hour (m^3/h) in the pre-processing period (2.5 Mt/a) and 29 m^3/h at full production (4 Mt/a). The potable water plant satisfies the demand for the accommodation camps and other onsite building use. Sewage to be collected via an underground sanitary sewer network to an above-grade mechanical sewage treatment plant. Sanitary sludge will be disposed at an approved offsite facility by an appropriate contractor.

5.3 DECOMMISIONING, REHABILITATION AND CLOSURE

5.3.1 Rehabilitation and Closure

Water management during progressive rehabilitation and rehabilitation and closure will be consistent with operation. However, due to the ground disturbance associated with the rehabilitation activities, standard ESC measures for construction will also be implemented to supplement the existing water quality treatment infrastructure.

The duration of rehabilitation and closure activities provides adequate time for earthworks activities to be completed, vegetation to establish, and water quality to improve and the open pits to fill and eventually discharge to the environment. Not withstanding the need for additional passive treatment, water management ponds will be breached to allow drainage to the natural ground and local receivers. Water quality treatment of effluent discharge in water management ponds and TMF effluent in the treatment plant will continue during rehabilitation and closure until water quality monitoring demonstrates that water quality is acceptable to release to the environment and that closure activities are successful. At that time, all water management features will be removed and restored to natural, pre-development drainage conditions and the water treatment plant decommissioned. Perimeter ditches will be backfilled with overburden and covered with a vegetated soil cover as per the piles themselves creating the following conditions:

- Non-contact runoff will drain down the pile slopes and benches or beaches, over the perimeter ditch footprints and overland to local receivers following natural drainage patterns
- Contact seepage will be substantially reduced from the uncovered condition due the increase in runoff and evapotranspiration potential of the vegetated soil cover. The reduced volume of contact seepage will migrate across the perimeter ditches and assimilate (attenuate naturally) with local groundwater to discharge into local receiving waters
- In cases where natural attenuation of contact seepage will not be adequate to improve groundwater discharge quality at the local receiver to background or CWQG-FAL thresholds, further passive treatment systems may be required
- Passive treatment systems could take the form of subsurface anaerobic units in the ditches or subsurface/surface units that utilize the water management pond basins as constructed wetland features. In both cases, seepage is transmitted in the ditch in a french drain subsurface arrangement

Based on the results of the mixing zone assessment (Stantec 2020g), passive treatment systems may be required to be implemented during closure / post-closure at the Marathon waste rock pile, Leprechaun waste rock pile and TMF. As seepage quality at the ultimate receiver location was predicted to have exceedance of CWQG-FAL of copper, zinc, and cyanide. For example, perimeter ditches can be maintained to collect seepage and to treat it passively in a constructed wetland or permeable reactive barrier and discharge as surface water.

5.3.2 Post-Closure and Monitoring

During the post-closure period, site monitoring is carried out to demonstrate that closure strategies of Project facilities are performing as intended. Monitoring will be conducted at final discharge points of the water management facilities and at receiving locations (e.g., Victoria River, Valentine Lake, and Victoria Lake Reservoir) simulated in the groundwater model to intercept seepage from the pits, waste rock piles, and TMF. Post-closure monitoring and maintenance will be carried out at a reduced frequency from the operation phase or closure period.

The post-closure monitoring program will continue after final closure activities are completed. As stated in the project overview, post-closure monitoring will cease once the Project-related effects are deemed to be physically and chemically stable, and accepted by regulatory agencies. The site can then be closed out or released by NLDNR and an application made to relinquish the property back to the Crown.

6.0 SITE WIDE WATER BALANCE

A site-wide water balance was completed to estimate the quantity of mine site contact water expected to be managed for each Project complex. The water balance model was built to simulate flows to the water management infrastructure at the Marathon Complex, and at the Process Plant and TMF and Leprechaun Complexes. The models simulate flows for construction, operation, decommissioning, rehabilitation and closure phases. The water balance represents the mine site facilities at full development during operation.

6.1 METHODS

The water balance models were developed using the GoldSim software package a predictive water quantity and quality modeling tool (Stantec, 2020e,f). The water balance models accounted for the precipitation and groundwater gains and evaporation, transpiration, and infiltration losses of each identified mine facility, with the exception of the pits and TMF, discussed separately.

As presented in Figure 6.1 for a stockpile or waste rock pile, the percentage of precipitation that results in runoff of the Project facility areas was accounted for in the water balance model by the proportion of gains and losses to precipitation. The model assumed that a waste rock pile was fully wetted during operation and did not represent a loss in the accounting associated with the wetting of the pile. Equation 1 presents the accounting of runoff collected in the water management ponds based on the hydrological inputs:

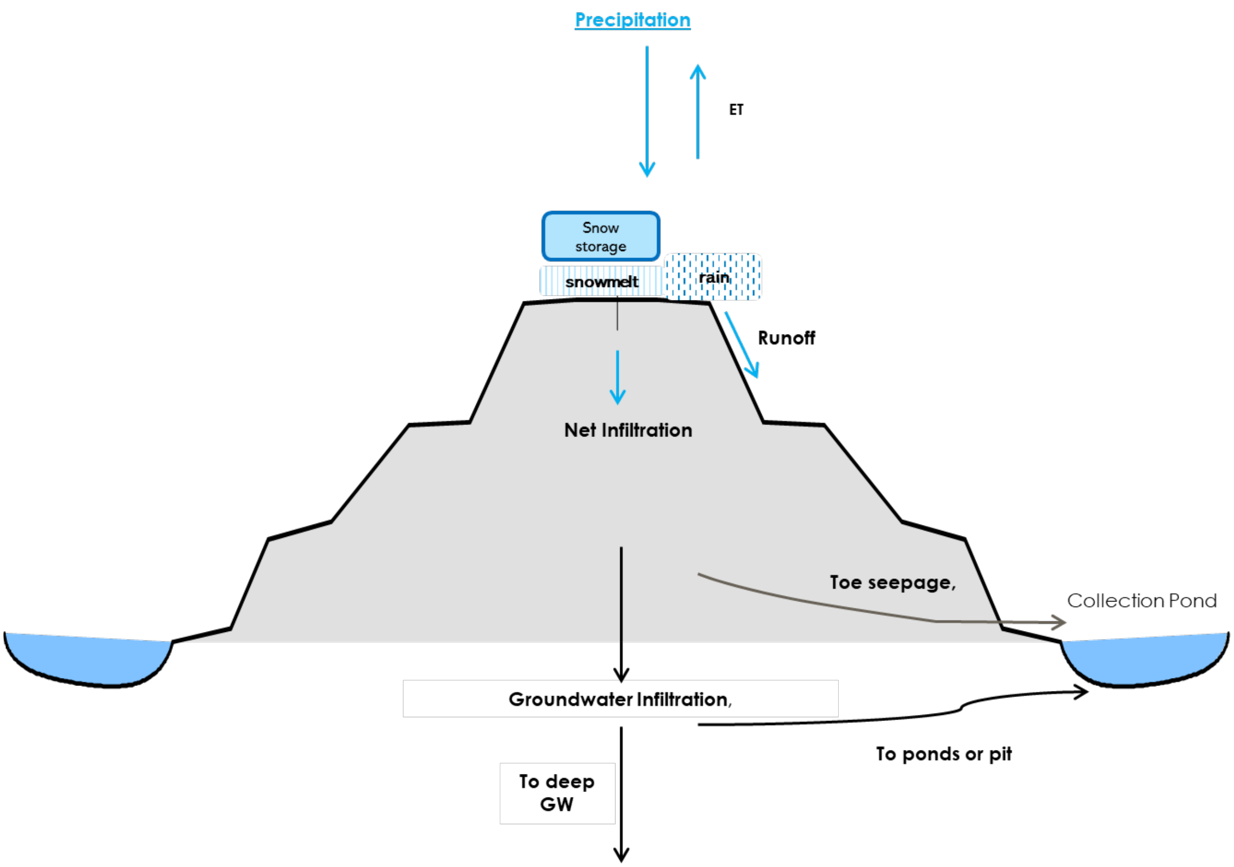


Figure 6.1 Waste Rock Pile Flow Pathways

Equation 1

Runoff to Water Management Ponds = Precipitation
– ET (%F)
- Snow Storage
+ Snow Melt Runoff (%F)
– Net infiltration
+ Toe Seepage
+ Shallow Groundwater Infiltration (%F)

where
%F = Adjustment factor applies as %
Net Infiltration = Toe Seepage + Shallow Groundwater + Deep Groundwater

Waste rock piles are comprised of a range of material grain sizes from fines to large boulders, with most material in the cobble to boulder size classes. Thus, when piles are open and not covered by snow, the pile surface is so coarse and has so much infiltration capacity that the piles do not generate runoff. The primary drainage activity during the open, non-snow covered period is infiltration. The infiltration that does not evaporate, referred to a net infiltration, seeps through the pile and routes either to toe seepage collected by the perimeter ditches or recharges deeper regional groundwater bypassing the perimeter ditch collection system. During snow-covered condition, rain of snow and snowmelt can produce runoff over snow and the ice lenses that develop within the snowpack.

The water balance of the TMF was based on a runoff coefficient approach. Runoff from the tailings and polishing ponds was estimated in the model based on the proportion of total precipitation (rainfall plus snow melt runoff) on the catchment multiplied by a runoff coefficient. This method is consistent with the prefeasibility level water balance model conducted by Golder for design (2020). The proportion of precipitation stored as snow was not accounted for at the TMF, as the snow storage in the pond will not be representative of the rest of the site.

As part of the design, runoff coefficients were assigned to different land use type, which includes:

- Natural or undisturbed ground upgradient of the TMF that will continue to gravity flow into the TMF during operation
- Prepared ground associated with areas that have been grubbed and/or graded, such as the perimeter haul roads and TMF embankments
- TMF dry tailings beach along the north dam and the water pond in the south

It was assumed that approximately 20% of the beaches were wet and the remaining 80% of the beaches were dry (Golder 2020). Natural ground runoff coefficient was based on an environmental water balance model based on assumptions of local climate and soil conditions and guidance provided by USGS (McCabe and Markstrom 2007), as presented in the 2019 Hydrology Baseline Report (Stantec 2019, 2020a).

Conceptual water management applied in the water balance models at the Marathon and Leprechaun Complexes for the operation phase is presented in Figure 6.2 through Figure 6.9.

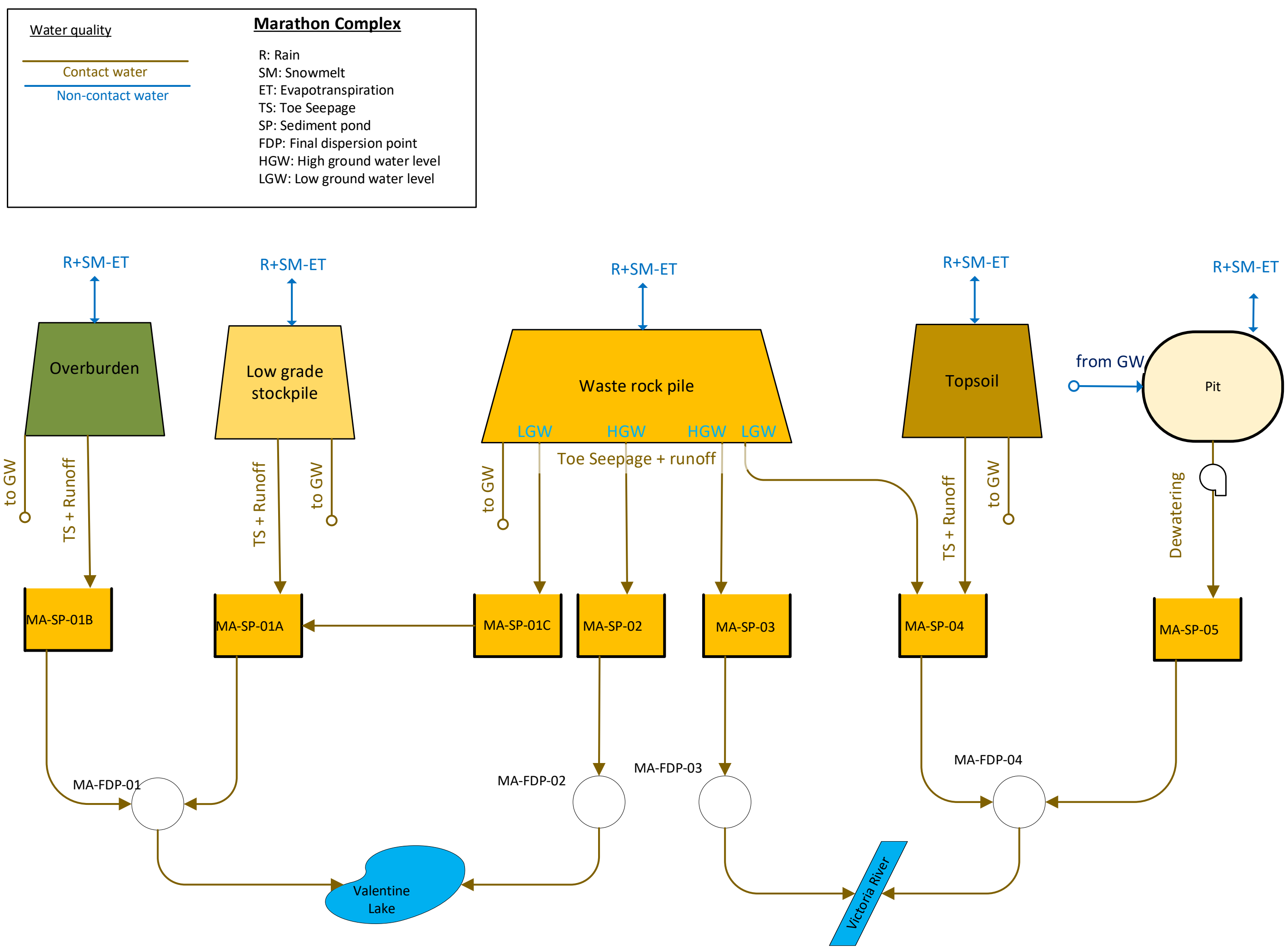


Figure 6.2 Conceptual Model of Mine Water Management – Marathon Complex Construction/Operation (Year -1 to 9)

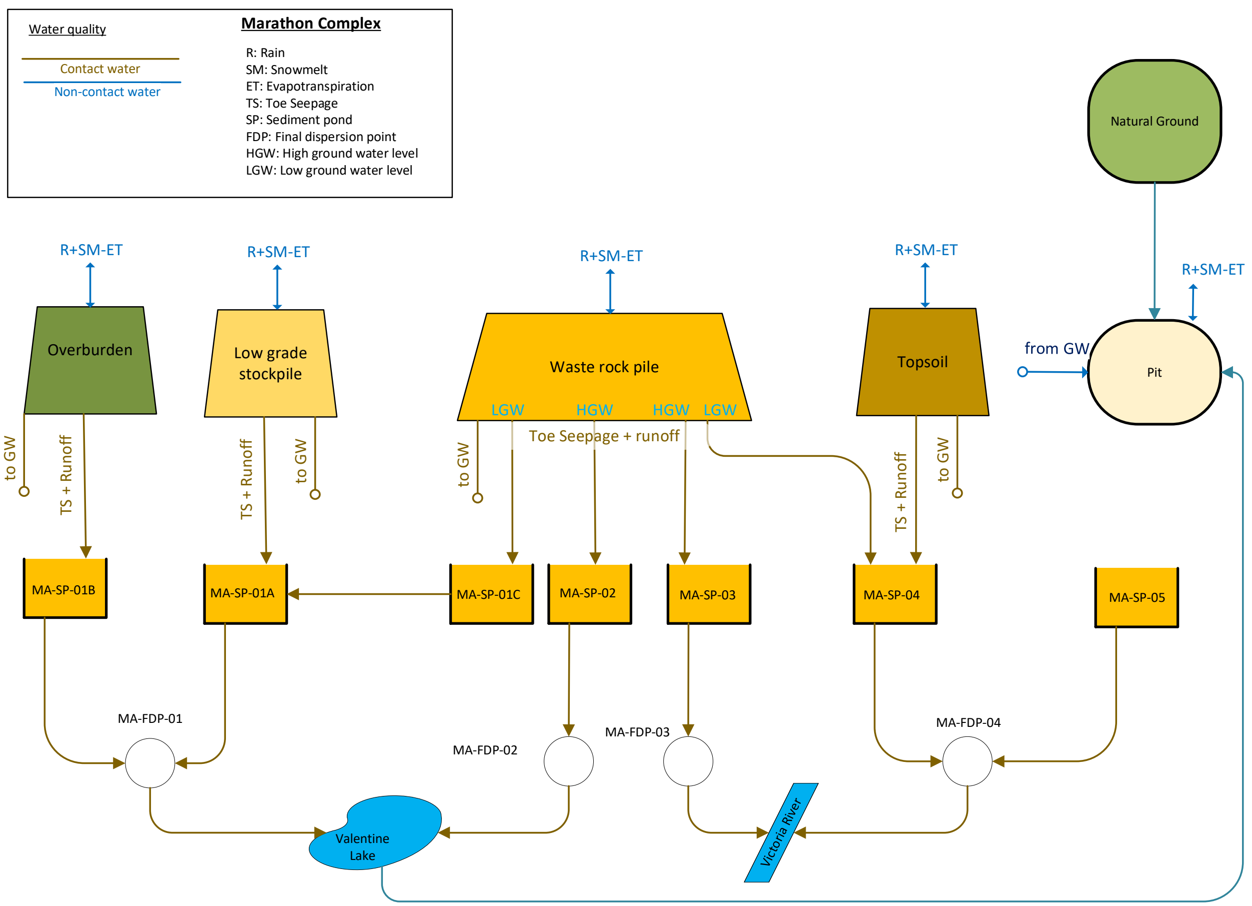
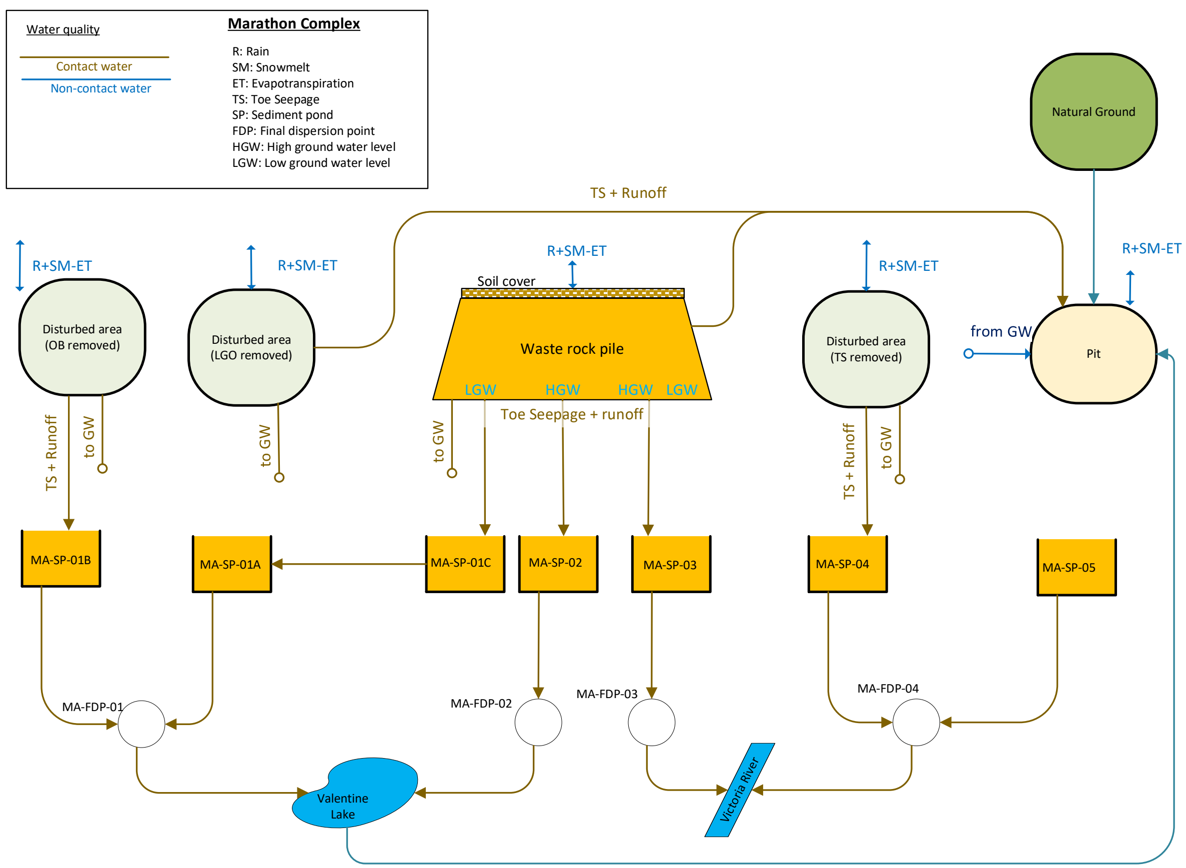


Figure 6.3 Conceptual Model of Mine Water Management – Marathon Complex Operation (Year 10 to 12)



Note: For simplicity in the model, water quantity and quality predictions were presented only for the combined flow at MA-FDP-01A from MA-SP-01A/B/C. Any dilution from flow between MA-FDP-01A/B is considered negligible.

Figure 6.4 Conceptual Model of Mine Water Management – Marathon Complex Closure (Year 13 until Pit is full)

Site Wide Water Balance
September 28, 2020

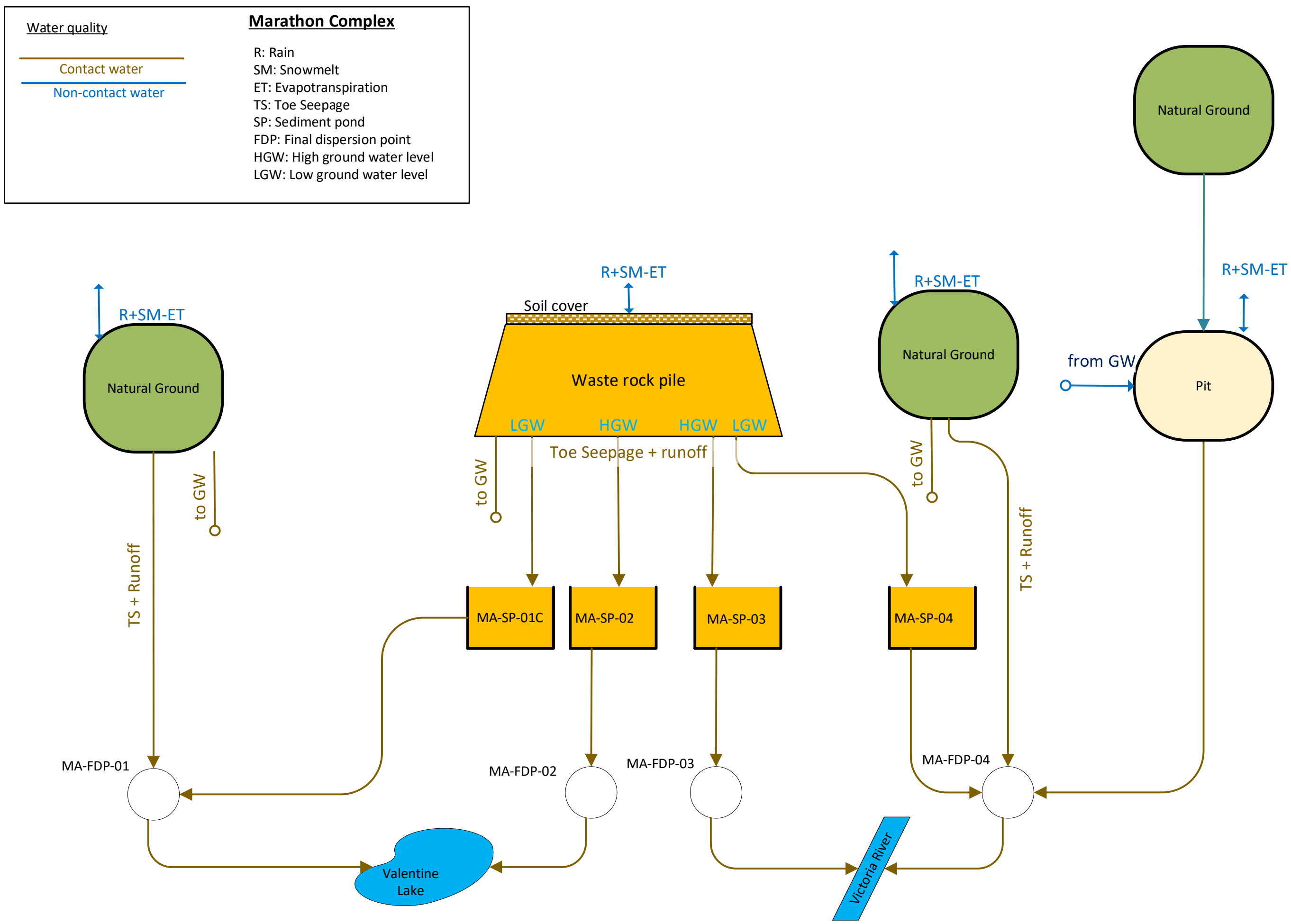


Figure 6.5 Conceptual Model of Mine Water Management – Marathon Complex Post-Closure (Pit is full)

Site Wide Water Balance
September 28, 2020

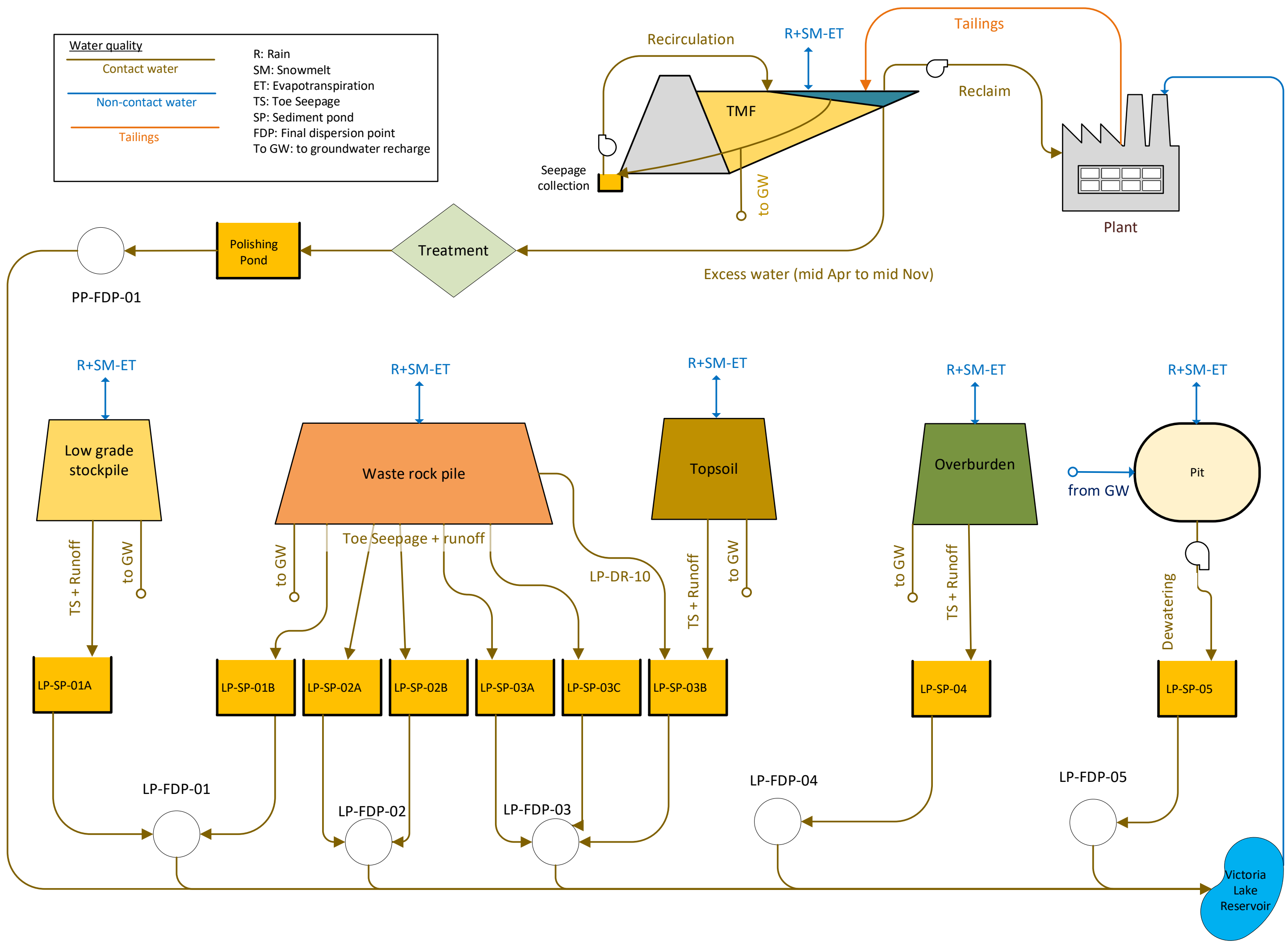


Figure 6.6 Conceptual Model of Mine Water Management – Leprechaun Complex Construction/Operation (Year -1 to 9)

Site Wide Water Balance
September 28, 2020

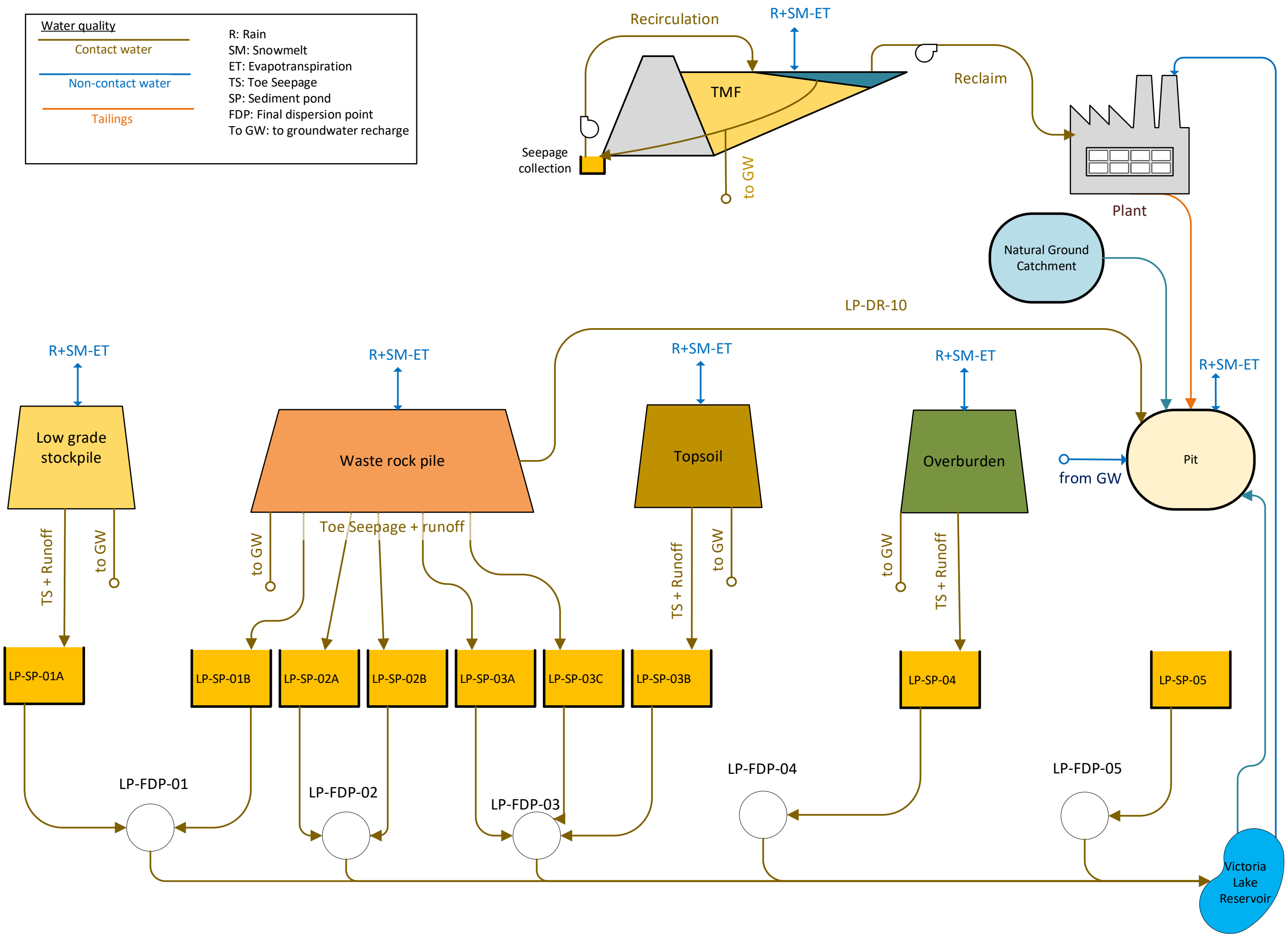


Figure 6.7 Conceptual Model of Mine Water Management – Leprechaun Complex Operation (Year 10 to 12)

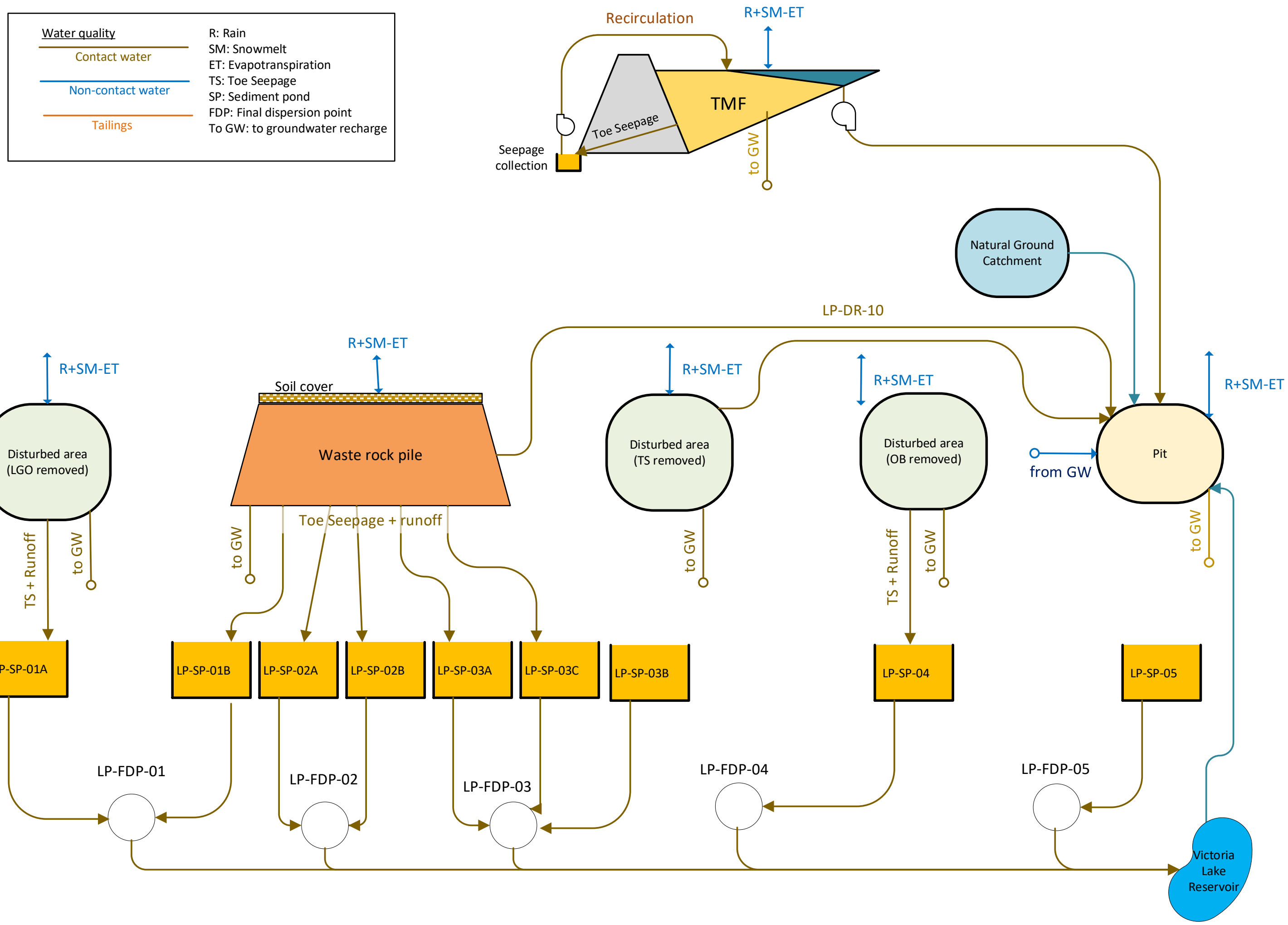


Figure 6.8 Conceptual Model of Mine Water Management – Leprechaun Complex Closure (Year 13 until Pit is full)

Site Wide Water Balance
September 28, 2020

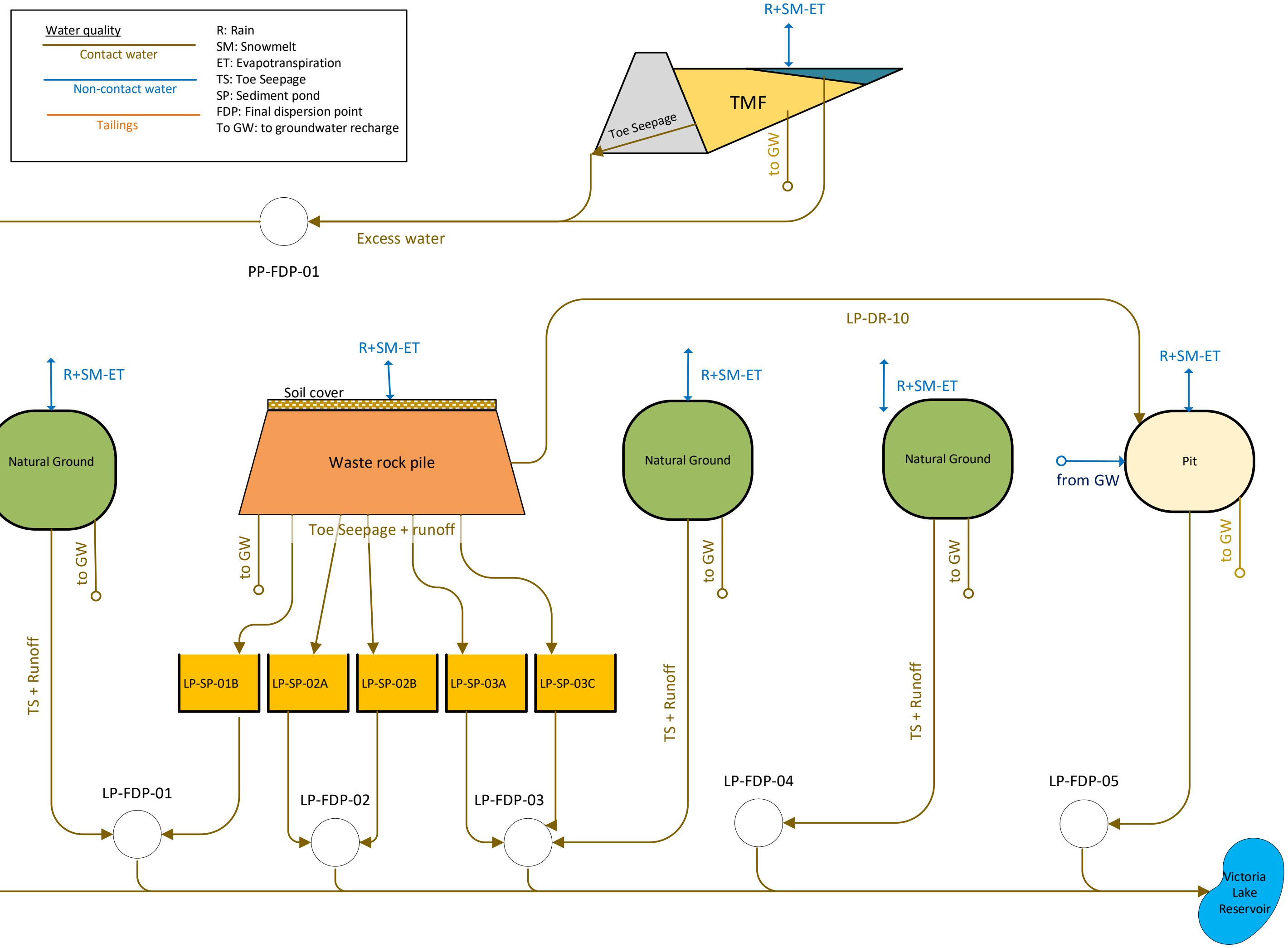


Figure 6.9 Conceptual Model of Mine Water Management – Leprechaun Complex Post-Closure (Pit is full)

6.2 WATER BALANCE RESULTS

Outflows and water quality from water management ponds were predicted in the water balance models, accounting for seepage and surface flow collected in the perimeter ditching of each Project facility and dewatering of the open pit.

As water management infrastructure is decommissioned during closure, runoff from Project facility areas during post closure is representative of the non-point discharge to the former water management pond. The water quality model shows that the ponds become full during freshet of the first year, and overflow to the FDPs thereafter. Table 6.1 and 6.2 presents the predicted water management pond outflows for the phases of development for the Marathon Complex, and Processing Plant and TMF and Leprechaun Complexes, respectively.

The magnitude of the flow from the water management ponds is dictated by pond volume, level, surface water flow and groundwater infiltration to the ponds.

Table 6.1 Monthly Average Flows/Outflows to/from Sediment Ponds (m^3/day) – Marathon

FDP	Period	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	Year
MA-FDP-01	Operations (Year 1 to 9)	917	1209	1490	3487	915	486	379	859	1034	1088	1407	1194	1205
	Operations (Year 10 to 12)	917	1204	1490	3487	915	486	379	859	1034	1088	1407	1194	1205
	Closure (Year 13 to 17)	771	973	1148	2705	751	361	231	708	919	985	1247	994	983
	Post-Closure (from Year 18)	651	822	973	2292	628	291	184	583	764	827	1052	841	826
MA-FDP-02	Operations (Year 1 to 9)	1115	1345	1554	3605	1435	1115	1044	1692	1787	1669	1894	1421	1640
	Operations (Year 10 to 12)	1115	1340	1554	3605	1435	1115	1044	1692	1787	1669	1894	1421	1639
	Closure (Year 13 to 17)	953	1186	1373	3260	915	429	231	873	1168	1243	1564	1225	1202
	Post-Closure (from Year 18)	943	1172	1360	3228	905	424	228	863	1155	1229	1548	1214	1189
MA-FDP-03	Operations (Year 1 to 9)	4516	5222	5951	11285	5367	4702	4711	5958	5935	5595	6151	5278	5889
	Operations (Year 10 to 12)	1428	1830	2244	5204	1631	1090	1010	1717	1868	1851	2257	1846	1998
	Closure (Year 13 to 17)	1168	1465	1715	4051	1099	470	258	1000	1357	1493	1903	2060	1503
	Post-Closure (from Year 18)	4545	5405	6236	13063	4536	2426	2109	4237	5070	5588	6762	5533	5459
MA-FDP-04	Operations (Year 1 to 9)	599	829	1069	2479	590	309	279	517	589	636	860	787	795
	Operations (Year 10 to 12)	599	825	1069	2479	590	309	279	517	589	636	860	787	795
	Closure (Year 13 to 17)	441	558	661	1554	407	159	91	353	485	550	711	589	546
	Post-Closure (from Year 18)	542	679	798	1880	507	210	117	456	621	691	883	699	674

Table 6.2 Monthly Average Flows/Outflows to/from Sediment Ponds (m^3/day) – Leprechaun

FDP	Period	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	Year
LP-FDP-01	Operations (Year 1 to 9)	518	694	876	2030	557	344	322	546	596	608	777	675	712
	Operations (Year 10 to 12)	518	692	876	2030	557	344	322	546	596	608	777	675	712
	Closure (Year 13 to 17)	540	687	816	1923	512	229	138	467	619	675	865	698	681
	Post Closure (from Year 18)	492	625	745	1754	462	201	119	416	556	611	787	635	617

Table 6.2 Monthly Average Flows/Outflows to/from Sediment Ponds (m^3/day) – Leprechaun

FDP	Period	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	Year
LP-FDP-02	Operations (Year 1 to 9)	1304	1625	1934	4484	1586	1153	1079	1776	1897	1820	2136	1673	22467
	Operations (Year 10 to 12)	1304	1619	1934	4484	1586	1153	1079	1776	1897	1820	2136	1673	22461
	Closure (Year 13 to 17)	1208	1515	1770	4191	1143	510	274	1062	1431	1560	1990	1575	18231
	Post Closure (from Year 18)	1276	1595	1863	4413	1211	545	293	1131	1523	1642	2083	1645	19220
LP-FDP-03	Operations (Year 1 to 9)	957	1276	1601	3715	1043	663	609	1045	1139	1143	1446	1247	1324
	Operations (Year 10 to 12)	957	1271	1601	3715	1043	663	609	1045	1139	1143	1446	1247	1323
	Closure (Year 13 to 17)	843	1069	1261	2981	789	342	184	717	970	1066	1373	1101	1058
	Post Closure (from Year 18)	888	1121	1321	3125	833	365	196	762	1029	1119	1433	1147	1112
LP-FDP-04	Operations (Year 1 to 9)	200	250	294	691	199	101	68	193	245	261	326	257	257
	Operations (Year 10 to 12)	200	249	294	691	199	101	68	193	245	261	326	257	257
	Closure (Year 13 to 17)	188	235	276	649	187	93	64	180	230	245	306	242	241
	Post Closure (from Year 18)	146	183	215	505	144	69	47	136	176	190	239	188	186
LP-FDP-05	Operations (Year 1 to 9)	2305	2533	2781	4607	2773	2648	2714	3128	3015	2796	2925	2570	2900
	Operations (Year 10 to 12)	42	52	64	145	34	0	0	18	33	51	70	54	47
	Closure (Year 13 to 17)	42	53	64	145	34	0	0	18	33	784	1399	1063	303
	Post Closure (from Year 18)	1783	2105	2443	4989	1618	504	448	1272	1663	2100	2624	2151	1975

Note: inflows are equal to outflows

The Marathon and Leprechaun pits will be mined for the first 10 years of the Project. In these years, flow components into the open pit include groundwater seepage, precipitation, surface runoff from natural areas, evaporation, and dewatering. As discussed previously, the Leprechaun pit will be operated as a tailings storage facility from Year 10 to the end of Year 12 and both pits will be filled with water to form pit lakes during closure.

To accelerate pit filling, the perimeter berms installed during operation to keep natural drainage from entering the pits will be removed and these flows will be directed toward the pits. In addition, reclaim water from the tailings pond (as tailings slurry via the processing plant) and freshwater from Victoria Lake Reservoir were simulated to be pumped to the Leprechaun pit during late operation, rehabilitation and closure, and into post-closure. Similarly, freshwater from Valentine Lake was simulated to be pumped to the Marathon pit. To fill the pits over a period of eight years post operation, a flow rate of 5.5 million cubic metres per year (Mm^3/year) or 178 Litres per second (L/s) from Valentine Lake for Marathon pit and a flow volume of 4.0 Mm^3/year from Victoria Lake Reservoir for Leprechaun pit is required. Accelerated pit filling will mitigate potential residual effects in that it will act to improve the water quality of the pit lake, reduce long term liability related to an extended period of natural pit filling, and expedites the submergence of PAG materials possibly exposed on the pit walls.

The source of water for the primary process plant is reclaim water from the TMF, supplemented with a freshwater make-up from Victoria Lake Reservoir. When water storage in the TMF is inadequate to supply normal reclaim flow to the process, additional water will be withdrawn from the Victoria Lake Reservoir. A water deficit in the TMF for reclaim was forecasted to occur in some months in Year 10 to the end of Year 12, associated with the start of tailings deposition in the Leprechaun pit, thereby decreasing the water (effluent) inflow to the TMF. Victoria Lake Reservoir will also be used as a water supply to fill Leprechaun pit directly during pit filling. The maximum flow rate from Victoria Lake Reservoir during Years 1 to 10 is predicted to be approximately 34 L/s and from Year 10 to 12, the maximum flow rate under accelerated pit filling is predicted to be 185 L/s.

The model was run iteratively to analyze the volume of excess water from the TMF requiring treatment prior to discharge to the environment. The tailings pond volume level at which the treatment is activated when the tailings pond level reaches 70% of its volume capacity. Operation of the pond the water treatment plant capacity during operation will not be exceeded for the 95^{th} percentile corresponding to a 1:25 year return period wet year. Results from the probabilistic analysis indicate no release of untreated water during operation (before Year 13) for the 95^{th} percentile. This condition could change depending on future operation management philosophy between the tailings pond and the water treatment plant.

7.0 WATER QUALITY TREATMENT & PREDICTIONS

7.1 WATER QUALITY TREATMENT

Water quality treatment for the tailings process water effluent involves the following:

1. Cyanide (CN) destruction circuit in the mill circuit, designed to reduce cyanide levels prior to discharging to the TMF
2. Sedimentation of suspended solids and supplemental natural cyanide degradation in the TMF tailings pond, with seasonal discharge to a process water treatment plant
3. Copper and ammonia removal and pH adjustment in the water treatment plant
4. Peak effluent flow equalization and sedimentation in the polishing pond

The tailings pond will have sufficient storage to facilitate the sedimentation and precipitation of suspended solids. Water will be stored in the tailings pond during open water conditions to promote natural degradation of residual cyanide, when possible. The cyanide degradation process in the tailings pond is primarily comprised of volatilization and UV light degradation. The tailings pond is predicted to have concentrations of unionized ammonia, total cyanide and copper above MDMER limits.

A water treatment plant will be situated below the tailings pond. The treatment process will be designed to remove ammonia, total cyanide, and copper. Additions of coagulant polymer will facilitate the removal of colloidal sized suspended matter. Effluent from the water treatment plant will be discharged to the polishing pond.

The polishing pond will further reduce the concentrations of contaminants to below the MDMER effluent limits, via further coagulation and sedimentation of copper and cyanide-metal solids and degradation of ammonia and cyanide. Water will be retained in the polishing pond for up to 5 days, with residence times developed to facilitate settling of coagulated particulate.

The water quality treatment chain including the mill cyanide destruction circuit, tailings pond, water treatment facility, and polishing pond, is designed to provide a final effluent that meets the MDMER effluent water quality criteria.

The water management design of other mine contact water treatment is focused on sedimentation. As sedimentation will reduce TSS concentrations and the particulate fraction of metals. Sedimentation for the treatment of Project facilities contact water will be mainly accomplished through the construction of water management ponds located near each FDP. The invert elevation of the water management pond outlet orifice pipe is set to provide an adequate pond volume for settling of the 1:10 year flood volume, the water quality design event.

Additional erosion and scour protection (e.g., sediment berms, rip-rap lining of ditches, energy dissipation pools) was designed in the collection ditches and downstream conveyance channel to further reduce TSS concentrations in the effluent.

7.2 WATER QUALITY PREDICTIONS

7.2.1 Sources of Potential Contaminants

As presented in the ARD/ML reports (Stantec 2020c,h), mine water from the open pit areas may contain suspended solids, explosive residuals (mainly ammonia, nitrite, and nitrate), and elevated levels of metals. Most of the pit walls and rubble on pit benches will be represented by waste rock, which has low ARD/ML potential in both deposits. Minimum ARD onset time is about six years after exposure of a small amount of potentially acid generating (PAG) materials based on kinetic testing (Stantec 2020h). These materials will be submerged during pit filling and therefore acidification of pit lakes water is not currently expected. Mine water discharged during operations and pit lake discharges are expected to meet MDMER limits.

Findings presented in the ARD/ML report (Stantec 2020h) are summarized below:

- Less than 0.5% of the approximately 50 Mm^3 of Leprechaun waste rock is classified as PAG. Overall, the waste rock pile is not expected to generate ARD due to the small amount of PAG material and significant excess of NP. Therefore, specific ARD management of waste rock is not required. For Marathon, approximately 14% of the 60 Mm^3 of waste rock is conservatively estimated to be PAG. Blending PAG and non-PAG rock with excess of neutralization potential and/or encapsulation of PAG waste by non-PAG rock is recommended to neutralize acidity potentially generated in PAG pockets. If these recommendations are followed, the final drainage from waste rock is not expected to be acidic. The waste rock pile will be covered by growth medium / overburden during rehabilitation, further reducing the risk of ARD/ML. There are no exceedances of MDMER limits observed in leachates from the waste rock humidity cells. Where waste rock will be used for site earthworks and grading during construction and operational development, necessary test work will be conducted to prevent PAG materials from being used in construction.
- About 10% of Leprechaun low-grade ore is estimated to be PAG, but overall is not expected to generate ARD. There are no exceedances of MDMER limits observed in these tests. For Marathon, approximately one-half of the low-grade ore is conservatively classified as PAG. The ARD onset time in PAG pockets of low-grade ore is approximately six years based on maximum laboratory leaching rates. The Marathon low-grade ore stockpile effluent has been segregated from other mine component flow streams in the overall mine design to facilitate collection and further ARD treatment, if required. There are no exceedances of MDMER limits observed in leachates from low-grade ore under neutral conditions.
- High-grade ore from the Leprechaun and Marathon deposits will be stockpiled together with 30% of the material originating from Leprechaun and the remainder from Marathon, on average. Approximately 13% and 67% of ore samples from Leprechaun and Marathon pits, respectively are conservatively classified as PAG. The overall mixture of Leprechaun and Marathon high-grade ores is non-PAG and the high-grade ore stockpile is not expected to generate ARD. Drainage from the high-grade ore stockpile flows to the TMF by gravity and any potential acidity will be neutralized in the decant pond or in the mill during pH adjustment required as a part of the gold recovery by cyanide process. No exceedances of MDMER are observed in results.

- Approximately 41 Mt of tailings will be produced from both high-grade ore and low-grade ore with about 38% of the material originating from the Leprechaun pit and the remainder from the Marathon pit.
- Composite samples of tailings from both deposits are classified as non-PAG and are not expected to generate ARD. During operation, tailings pond and pore water will likely exceed the MDMER limits for total cyanide (CN_T), un-ionized ammonia ($N\text{-}NH_{3\ UN}$), and copper (Cu) sourced from process water. Seepage from the TMF is conservatively predicted to exceed MDMER limits for CN_T, un-ionized ammonia, and Cu in post-closure. Requirement for treatment is further predicted by the water quality models and assimilative capacity assessment and discussed in the EIS.
- Surface runoff from areas immediately up-gradient of the tailings disposal area may contain suspended solids from wind-blown sources (i.e., process plant area, dry tailings beaches, and ROM pad). Process tailings water from the mill will contain suspended solids, be highly alkaline, and contain free and metal-complexed cyanide. Excess water produced by the TMF will be reclaimed to the process plant to offset process water demand and limit volumes of discharge from the tailings pond. TMF excess water not reused in ore processing will be treated via a water treatment plant and directed to a polishing pond prior to discharge to the environment.

7.2.2 Predicted Surface Water Quality

The monthly effluent water quality (mean and 95th percentile) at each FDP during operation was simulated during the Project phases of construction, operation, closure and post-closure periods as presented in the water quality and water quality modelling reports (Stantec 2020e,f). Water quality predictions were simulated using a GoldSim model integrating water balance and geochemical inputs. The major objective of the water quality model is to predict concentrations of potential contaminants in mine water collection facilities and at FDPs.

The parameters included in the model have criteria listed in CWQG-FAL and limits in the MDMER. Only the MDMER limits are directly applicable to the discharges. The CWQG-FAL guidelines are not applicable to discharges, as these guidelines are developed for the receiving environment and are used for screening and providing inputs to assimilative capacity assessments.

The predictions for discharge points to the environment can be summarized as follows:

Marathon Complex:

- Water quality model shows that there are no MDMER exceedances predicted at facilities and discharges in the Marathon Complex (waste rock pile, stockpiles, open pit, ponds and MA-FDP-01 to MA-FDP-04) during all mine phases at 95th percentile confidence level.
- At baseline conditions, phosphorus (P), chromium (Cr), and zinc (Zn) exceed the respective long-term CWQG-FAL in streams near the Marathon open pit.
- During construction and operation, long-term CWQG-FAL exceedances of copper (Cu), mercury (Hg), fluoride (F), nitrite ($N\text{-}NO_2$), Silver (Ag), un-ionized ammonia ($N\text{-}NH_{3\ UN}$), cadmium (Cd),

manganese (Mn), aluminum (Al), arsenic (As), total ammonia ($N\text{-}NH_{3\ T}$), selenium (Se), uranium (U), lead (Pb), iron (Fe), and nitrate ($N\text{-}NO_3$) are predicted to be above the respective long-term CWQG-FAL, in addition to the parameters exceeding at baseline conditions.

- The largest concentrations predicted for water quality during operation were for MA-FDP-02 and associated with seepage from waste rock.
- These parameters decline during closure and stabilize in post-closure with Cu, Hg, F, Ag, Cd, Mn, and Al remaining above CWQG-FAL. Exceedance for F could be a modelling artifact related to high detection limits scaled up to the full-size of the waste rock pile. Zn and Cr stabilize above the background levels in post-closure. The levels and trends for the parameters exceeding CWQG-FAL in MA-FDP-02 and MA-FDP-03 are similar.

Leprechaun Complex:

- Water quality model shows that there are no MDMER exceedances predicted at facilities and discharges in the Leprechaun Complex (waste rock pile, stockpiles, open pit, ponds and LP-FDP-01 to LP-FDP-05) during all mine phases at 95th percentile confidence level.
- Long-term CWQG-FAL exceedances of phosphorus (P), chromium (Cr), zinc (Zn), aluminum (Al), manganese (Mn), and iron (Fe) at baseline conditions and during construction.
- In addition to the parameters exceeding at baseline conditions, long-term CWQG-FAL exceedances of Cu, Hg, F, $N\text{-}NO_2$, Ag, $N\text{-}NH_{3\ UN}$, As, $N\text{-}NH_{3\ T}$, Cd, Pb, U, Se, and $N\text{-}NO_3$ are predicted to be above the respective long-term CWQG-FAL for LP-FDP-03.
- These parameters decline during closure and stabilize in post-closure with Cu, Hg, Ag, and F remaining above CWQG-FAL.
- During operation, the highest number of long-term CWQG-FAL exceedances were predicted for LP-FDP-03 and associated with seepage from waste rock. Seepage from waste rock and low-grade ore also affects LP-FDP-01 and LP-FDP-02, but these discharges have better water quality than LP-FDP-03 resulting in less exceedances of CWQG-FAL.

Processing Plant and TMF Complex

- During construction, water quality of the polishing pond PP-FDP-01 is similar to the chemistry of undisturbed runoff, which showed exceedances of the long-term CWQG-FAL for P, Zn, Cr, Mn, As, Al, Fe, and Cu considering 95th percentile concentrations.
- The model predicts exceedances of MDMER limits for $CN_{\ T}$, Cu, and $N\text{-}NH_{3\ UN}$ in the tailings pond, indicating that these parameters may require treatment in mine Years 1 to 10. At that time, the polishing pond receives treated effluent.
- During operation, Cu, $N\text{-}NH_{3\ UN}$, F, $N\text{-}NH_{3\ T}$, $CN_{\ WAD}$, Hg, $N\text{-}NO_2$, Se and Cd are predicted to be above the respective long-term CWQG-FAL, in addition to baseline exceedances. There is no

inflow from the tailings pond to the polishing pond starting in mine Year 10 and until end of the closure, and therefore, the discharge for the polishing pond returns to baseline conditions during this period. In post closure, Cu is predicted to exceed the MDMER limit due to an elevated concentration of this metal in tailings pond toe seepage. Therefore, a mitigation such as passive treatment of seepage should be considered. In addition to the MDMER exceedance for Cu and baseline indicated above, CN_{WAD}, $N\text{-}NH_{3\ UN}$, and $N\text{-}NH_{3\ T}$, are predicted to be above long-term CWQG-FAL in post-closure.

Results of the effluent/discharge water quality are further described in the Water Quantity and Quality Modelling Reports (Stantec 2020e,f).

7.2.3 Predicted Groundwater Quality

Significant adverse Project related effects on groundwater resources from the Project are not anticipated. The groundwater table in the area is near surface, which will inhibit inflow by maintaining a low gradient for groundwater flow. At the TMF, in addition to the low gradient, the low permeability of the tailings, and the presence of an upstream clay blanket will limit seepage into the groundwater. The low-permeability clay core of the tailings dam will also limit the amount of lateral seepage from the TMF to the perimeter ditches. Seepage through the dam will be low relative to average daily discharge rates at the FDP. Permeability through the clay core of the dam will minimize the passage of tailings water through the dam wall. Shallow seepage from the south of the tailings pond was assumed to run into the polishing pond, and seepage along the remaining perimeter of the dam is collected in ditches and recycled back into the tailings pond. Some groundwater is predicted to seep from the TMF and travel to the Victoria River and tributaries. Elevated concentrations of some metals (i.e., iron and manganese) are predicted to exceed the CWQG-FAL criteria, however, these elements exceed the concentrations in the baseline conditions. In addition, unionized ammonia is also predicted to exceed the CWQG-FAL criteria, but it is anticipated that the mill operations can be optimized to reduce arsenic, cyanide and ammonia.

7.3 PARAMETERS OF POTENTIAL CONCERN

Significant adverse Project related effects on groundwater resources from the Project are not anticipated. At the TMF, the low permeability of the tailings, and the presence of a synthetic liner on the upstream side of the dam will limit seepage into the groundwater and lateral seepage from the TMF to the perimeter ditches. Seepage through the dam will be low relative to average daily discharge rates at the FDP. The presence of the low permeability synthetic liner will minimize the passage of tailings water through the dam wall. Shallow seepage from the south of the tailings pond was assumed to run into the polishing pond, and seepage along the remaining perimeter of the dam is collected in ditches and recycled back into the tailings pond. Some groundwater is predicted to seep from the TMF and travel to the Victoria River and tributaries. Elevated concentrations of some metals (e.g., copper and zinc) are predicted to be below MDMER limits and to exceed the CCME FAL criteria, however, these elements exceed the concentrations in the baseline conditions. In addition, unionized ammonia is also predicted to exceed the

CCME FAL criteria, however it is anticipated that the mill operations can be optimized to reduce arsenic, cyanide and ammonia. The predicted water quality POPC for effluent treatment are listed in Table 7.1.

Table 7.1 Parameters of Potential Concern

Parameter	Units	MDMER Max Monthly Mean (µg/L)	CWQG-FAL Long-term
Aluminum (Total)	µg/L	-	100
Arsenic (Total)	µg/L	100	5
Cadmium (Total)	µg/L	-	0.04
Copper (Total)	µg/L	100	range of 2 to 4
Cyanide [a,b]	µg/L	500	
Cyanide (WAD) [a,b]	µg/L	-	5 (as free CN)
Fluoride [a]	µg/L	-	120
Iron (Total)	µg/L	-	300
Lead (Total)	µg/L	80	range of 1 to 7
Manganese (Total)	µg/L	-	210
Nitrite (as N)	µg/L	-	60
Total Ammonia (as N)	µg/L	-	689
Unionized Ammonia as N	µg/L	500	19
Phosphorus (Total)	µg/L	-	4
Sulphate	µg/L	-	-
Total Suspended Solids	mg/L	-	5
Zinc (Total)	µg/L	400	30

Notes
a- indicates parameters that did not have baseline water quality data. Mean and 95th percentile concentrations for these parameters outlined in the Water Quality Modelling Report (Stantec 2020e,f).
b- Cyanide is only a parameter of concern for the TMF effluent as it relates to processing of ore

Both nickel and radium-226 have MDMER effluent discharge limits, but the predicted water quality did not meet the threshold of a POPC. Similarly, concentrations of chromium, mercury, selenium, silver, and uranium predicted to exceed the CWQG-FAL at some FDPs also did not meet the threshold of a POPC. Elevated concentrations of sodium at all FDPs were also not considered a POPC, as there is no CWQG-FAL for this parameter.

7.4 ASSIMILIATIVE CAPACITY

An Assimilative Capacity (AC) assessment was completed for the operation phase and post-closure conditions of the Project. These phases are anticipated to represent the worst-case conditions with respect to effluent quality. The AC assessment was conducted to estimate the water quality of watercourse and waterbodies receiving discharges directly from FDPs, as well as the ultimate receivers.

An assimilative capacity assessment was conducted using the near-field mixing model Cornell Mixing Zone Expert System (CORMIX, Version 11.0) at 100 m and 250m downstream of the FDPs and for the three ultimate receivers of Valentine Lake, Victoria Lake Reservoir, and Victoria River. CORMIX model was used to model mixing zones at the three ultimate receivers under both the regulatory and normal operating conditions.

Water quality was assessed using a mass balance approach under two discharge conditions: regulatory and normal. The regulatory operating conditions are considered worst case and conservative while normal operating conditions are considered representative of the expected average discharge conditions. Input parameters for these two operating conditions were:

- Regulatory Operating Conditions:
  - MDMER limits for POPC listed parameters for effluent
  - 95th percentile for POPC not listed in MDMER, generated from water quality modelling
  - 75th percentile baseline water quality in the receiving watercourses
  - 7Q10 flow conditions (7-day low flow, 10-year return period)
- Normal Operating Conditions
  - Mean concentrations for POPC generated from water quality modelling
  - Mean concentrations for baseline water quality in the receiving watercourses
  - Mean annual flow (MAF) conditions

The results of the three CORMIX models provide an estimate of the POPCs within the effluent mixing zones under conservative conditions. The conservative conditions are based on maximum effluent concentrations, low flow (7Q10) conditions in the receiving environment and assuming no contaminant decay, sedimentation, and reduction/oxidation kinetics in the mixing zones.

Generally, for both of the regulatory and the normal operating scenarios, limited assimilative capacity is seen downstream of each FDP until reaching Victoria Lake Reservoir, Valentine Lake, or Victoria River, at which point mixing improves. The FDPs are shown on Figures 4.1 through 4.6 and discussed in detail below.

The mixing zones were determined in terms of assimilation or dilution ratios for the maximum effluent flow rate expected to enter each receiving waterbody. Expected water quality at the 100 m, downstream of the receiving point of the three ultimate receivers for POPC were determined.

The Marathon Complex, for the regulatory scenario, has exceedances for zinc at the 100 and 200 m mixing zone for MA-FDP-02, and MA-FDP-03/04. Also, exceedances for aluminum, iron, and manganese were observed in the combined effluent from MA-FDP-03 and MA-FDP-04. These exceedances are due to conservative assumptions of the effluent flow and low assimilative capacity of the watercourse. Additionally, the effluent concentrations were assumed at the MDMER limits, which is a very conservative assumption. Based on extrapolated dilution ratios for the regulatory scenario, it is expected that the ultimate mixing zone will extent approximately 300 m from the outfall, at which point all parameters will meet the CWQG-FAL.

For the Leprechaun Complex and Process Plant and TMF Complex, water quality at the end of the 100 m mixing zone for the regulatory scenario meets the CWQG-FAL for most FDPs, except for the combined effluent from LP-FDP-03 and LP-FDP-05, which has potential exceedances for arsenic, copper, lead, zinc and fluoride. These exceedances are due to the conservative assumption of effluent flow and low assimilative capacity of the watercourse. Additionally, the effluent concentrations were assumed at the MDMER levels, which is a very conservative assumption. Based on extrapolated dilution ratios for the regulatory scenario, it is expected that the ultimate mixing zone extends approximately 300 m from the outfall, at which point all parameters will meet the CWQG-FAL.

During the post-closure period of the decommissioning, rehabilitation and closure phase, the results of the mixing zone assessment (Stantec 2020g) include:

- Seepage quality at the discharge to the ultimate receiver to Victoria lake exceeded the CWQG-FAL for aluminum, copper and fluoride from the Marathon waste rock pile
- Seepage quality at the discharge to the ultimate receiver to Valentine Lake Reservoir exceeded the CWQG-FAL for zinc from the Leprechaun waste rock pile
- Seepage quality at the discharge to the ultimate receiver to Victoria River exceeded the CWQG-FAL for copper and cyanide from the TMF

Mitigation measures should be considered, such as maintaining perimeter ditching during closure / post-closure to convey seepage to a passive wetland treatment system

8.0 MONITORING PLAN

The objective of the monitoring program is to confirm compliance with regulatory requirements, support predictions of effects of the Project on water quality, identify changes in drainage patterns and surface water flow, and determine if additional mitigation or emergency response measures are required. All monitoring results will be submitted to NLDECCM within 30 days of each quarter of monitoring and will include the laboratory certificates of analysis and in spreadsheet format. An annual report is planned to be submitted to NLDECCM identifying relative trends in parameters and a discussion of the significance of the findings.

The proposed monitoring program will include surface and groundwater quality monitoring, surface water flow monitoring of nearby watercourses and effluent discharge locations, groundwater level monitoring of installed monitoring wells, and visual inspections of facility infrastructure. The proposed monitoring locations are preliminary, and will be reviewed and modified as design proceeds in consultation with regulators, and in accordance with permits and approvals.

Specific details of required effluent and exposure water quality monitoring under MDMER, such as toxicity testing, environmental effects monitoring and equipment calibration and testing are not part of the monitoring plan. These details will be developed in partnership with ECCC prior to operation.

As per the Provincial EIS Guidelines, the province requires real-time water monitoring. The following list of proposed water monitoring parameters and stations is recommended as the station inventory from which a select sub-set of stations with be instrumented with telemetry and linked to a real-time monitoring network. Marathon will engage with NLDECCM regarding the establishment of the real-time water monitoring network for the Project.

Quality Assurance and Quality Control (QA/QC) is an integral component of proper field and laboratory procedures. As stated in the MDMER (Schedule 5, Section 7(e)), water quality monitoring is to be conducted by *implementing quality assurance and quality control measures that will ensure the accuracy of water quality monitoring data* (MDMER 2019).

8.1 SURFACE WATER QUANTITY MONITORING

As part of routine operation, effluent discharge, mine water, tailings water reclaim, freshwater makeup, process water and potable water volumes will be recorded on a daily basis. Gauges will be installed in distribution lines for process reclaim water, spigoted tailings, and process water discharge to facilitate the monitoring of flows. Records will include a monthly total and average volumes. Fresh water make-up and potable water withdrawal will be gauged and recorded.

Hydrometric monitoring will be conducted at the FDPs at a minimum accuracy of 15% of the total discharge, according to the flow measurement requirements outlined in the MDMER guidance document (ECCC 2020). In addition, flow monitoring will be conducted at existing streams that are adjacent to the pits. To satisfy the precision required in MDMER, pressure transducers will be installed at discharge locations to measure water level hourly. Water level will be translated to flow through an established stage-discharge curve. In addition, water levels will be manually measured on a staff gauge at the time of data logger retrieval for comparison to the automated level in order to detect measurement drift in pressure transducers and make any required data adjustments

Flow monitoring of all pumping equipment on site will be conducted using flow totalizing meters, this include the open pit dewatering rates, water withdrawal rates from Victoria Lake Reservoir, water treatment plant rates, effluent discharge from TMF, reclaim and tailings deposition rates. Water levels in all water management ponds will be monitored using pressure transducers to estimate the daily flow volume discharge from the water management ponds.

8.2 SURFACE WATER QUALITY MONITORING

Surface water quality will be impacted by runoff that comes in contact with the mine. While no formal limits are assigned in permitting or approval, parameters listed in Table 8.1 must be monitored at the surface water quality monitoring sites during construction, operation, and decommissioning, rehabilitation and closure of the Project. Additionally, pursuant to the MDMER (Subsections 5, 14, and 17), monthly acute toxicity and bi-annual sublethal toxicity testing must be completed for effluent from the FDPs.

Preliminary surface water monitoring locations are described in Table 8.1 and shown on Figures 8.1 to 8.3. The locations of these stations may require some adjustments in the field post-construction, where applicable. These stations can be partitioned by those identified to characterize water quality at background stations, and those selected as downstream stations (i.e., reference and exposure stations).

The sampling frequency at FDPs may be decreased from monthly to quarterly if the MDMER parameter concentrations are found to be less than 10% of the value set out in column 2 of Schedule 4 for 12 consecutive months. Water quality monitoring stations that are not associated with an FDP will be reevaluated after the first year of operation.

Table 8.1 Surface Water Monitoring Stations and Requirements

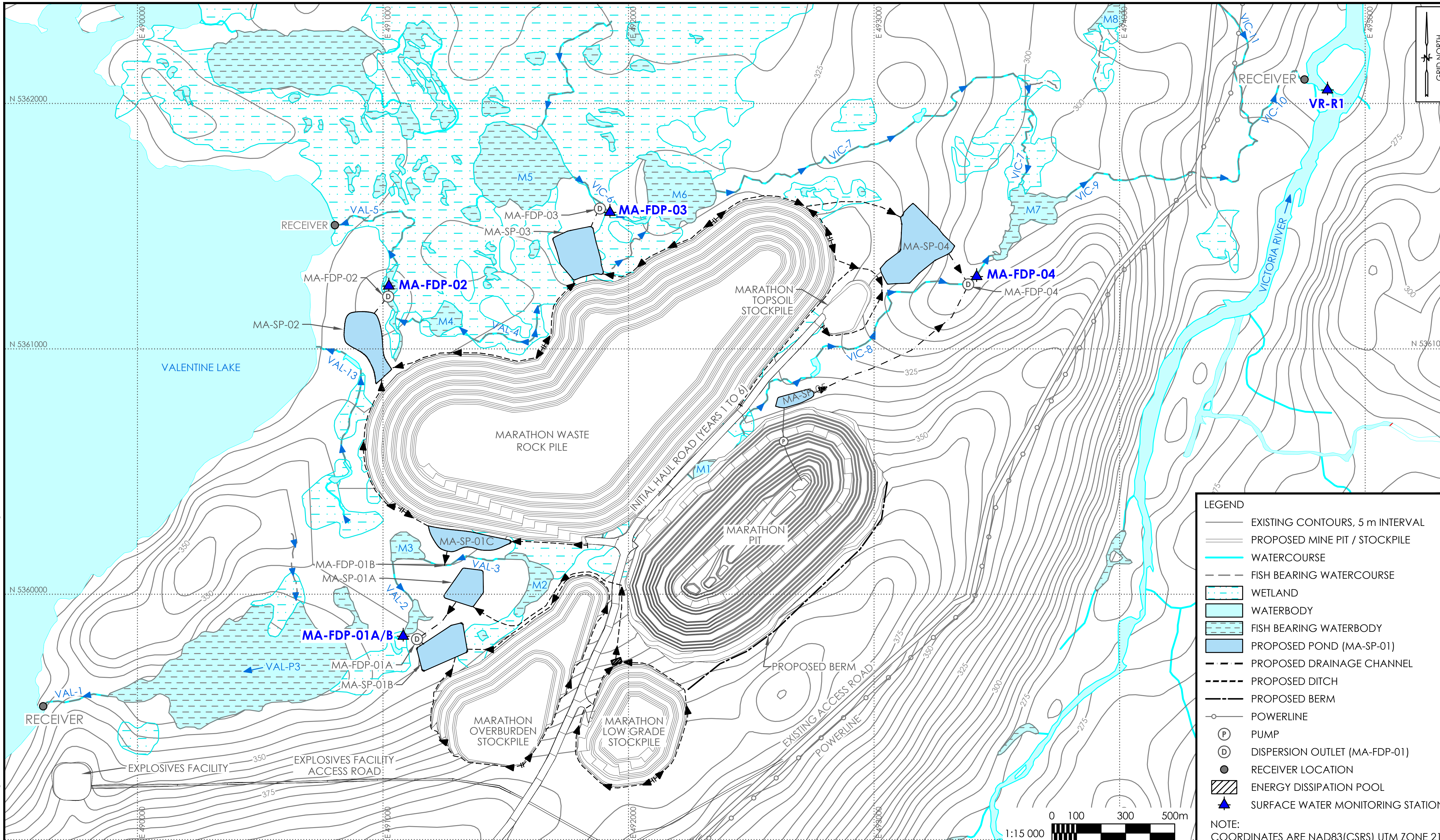
Site	Rational	Description	Water Quality Parameters	Monitoring Frequency
MDMER Required Monitoring Stations				
MA-FDP-01A/B	FDP	Stream Val-3 and Stream Val-2 at FDP	General Acute Toxicity Sublethal Toxicity Flow pH	Weekly Monthly Bi-Annually Daily Weekly
MA-FDP-02	FDP	Stream Val-5 at FDP	General Acute Toxicity Sublethal Toxicity Flow pH	Weekly Monthly Bi-Annually Daily Weekly
MA-FDP-03	FDP	Stream Val-6 at FDP	General Acute Toxicity Sublethal Toxicity Flow pH	Weekly Monthly Bi-Annually Daily Weekly

Table 8.1 Surface Water Monitoring Stations and Requirements

Site	Rational	Description	Water Quality Parameters	Monitoring Frequency
MA-FPD-04	FDP	Stream ViR-8 at FDP	General Acute Toxicity Sublethal Toxicity Flow pH	Weekly Monthly Bi-Annually Daily Weekly
LP-FDP-01	FDP	Wetland connected to Stream VIC-26 at FDP	General Acute Toxicity Sublethal Toxicity Flow pH	Weekly Monthly Bi-Annually Daily Weekly
LP-FDP-02	FDP	Effluent Discharge to Stream VIC-26 at FDP	General Acute Toxicity Sublethal Toxicity Flow pH	Weekly Monthly Bi-Annually Daily Weekly
LP-FDP-03	FDP	Conveyance Channel to Victoria Lake Reservoir at FDP	General Acute Toxicity Sublethal Toxicity Flow pH	Weekly Monthly Bi-Annually Daily Weekly
LP-FDP-04	FDP	Stream Vic-17 at FDP	General Acute Toxicity Sublethal Toxicity Flow pH	Weekly Monthly Bi-Annually Daily Weekly
LP-FDP-05	FDP	Pond VIC-P2 at FDP	General Acute Toxicity Sublethal Toxicity Flow pH	Weekly Monthly Bi-Annually Daily Weekly
PP-FDP-01	FDP	Polishing Pond	General Acute Toxicity Sublethal Toxicity Flow pH	Weekly Monthly Bi-Annually Daily Weekly
PP-FDP-02	FDP	Pond L2 at FDP	General Acute Toxicity Sublethal Toxicity Flow pH	Weekly Monthly Bi-Annually Daily Weekly
To Characterize Background and Reference Water Quality				
VR-R1	Reference	Victoria River – 100 m upstream of receiver location	General	Monthly
Val-R1	Background	Valentine Lake Reservoir – upstream	General	Monthly
VIC-R1	Background	Victoria Lake Reservoir – upstream	General	Monthly
VR-R2	Background	East Tributary of Victoria River at headwaters	General	Monthly

Table 8.1 Surface Water Monitoring Stations and Requirements

Site	Rational	Description	Water Quality Parameters	Monitoring Frequency
To Assess Environmental Effects of Mine				
C001a	Downstream	Downstream of TMF site in stream ViR14	General	Monthly
TMF3	Downstream	Downstream of TMF site	General TSS	Monthly
SCD1	Downstream	Seepage Collection Ditch – East side of TMF	General Flow	Monthly Daily
SCD2	Downstream	Seepage Collection Ditch – South side of TMF	General Flow	Monthly Daily
SCD3	Downstream	Seepage Collection Ditch – South side of TMF	General Flow	Monthly Daily
SCD4	Downstream	Seepage Collection Ditch - West side of TMF	General Flow	Monthly Daily
VIC-27	Downstream	Downstream of FDP PP-FDP-01	General	Monthly
VAL-19	Downstream	East of Leprechaun Pit	Flow	Daily
VIC-29		North of Leprechaun Pit	General Flow	Monthly Daily
VIC-25	Proximity to roadway	Adjacent to Haul Road – Leprechaun Complex	TSS	
General Water Quality – Parameters to be Monitored				
Total Aluminum, Total Arsenic, Total Cadmium, Total Copper, Cyanide, WAD Cyanide, Flouride, Total Fluoride, Total Iron, Total Lead, Total Manganese, Nitrite, Nitrogen Ammonia, Unionized Ammonia, pH, Phosphorus, Sulphate, Total Suspended Solids, Total and Dissolved Zinc, Hardness, and Sodium.				
Acute and Sub-lethal Toxicity Testing (DFO 2016)				
Rainbow Trout Acute Lethality Test (non-acutely lethal at all times), *Daphnia magna* Acute Lethality Test, Sublethal Testing				
Note: Monitoring locations are preliminary and will be reviewed and revised as design progresses in consultation with regulators and in accordance with permits and approval.				



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PRELIMINARY SURFACE WATER MONITORING STATIONS - MARATHON COMPLEX
WATER MANAGEMENT PLAN
VALENTINE GOLD PROJECT, NL

Client: MARATHON GOLD CORP

Job No.:	121416408
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App'd By:	RJ

Fig. No.: 8.1

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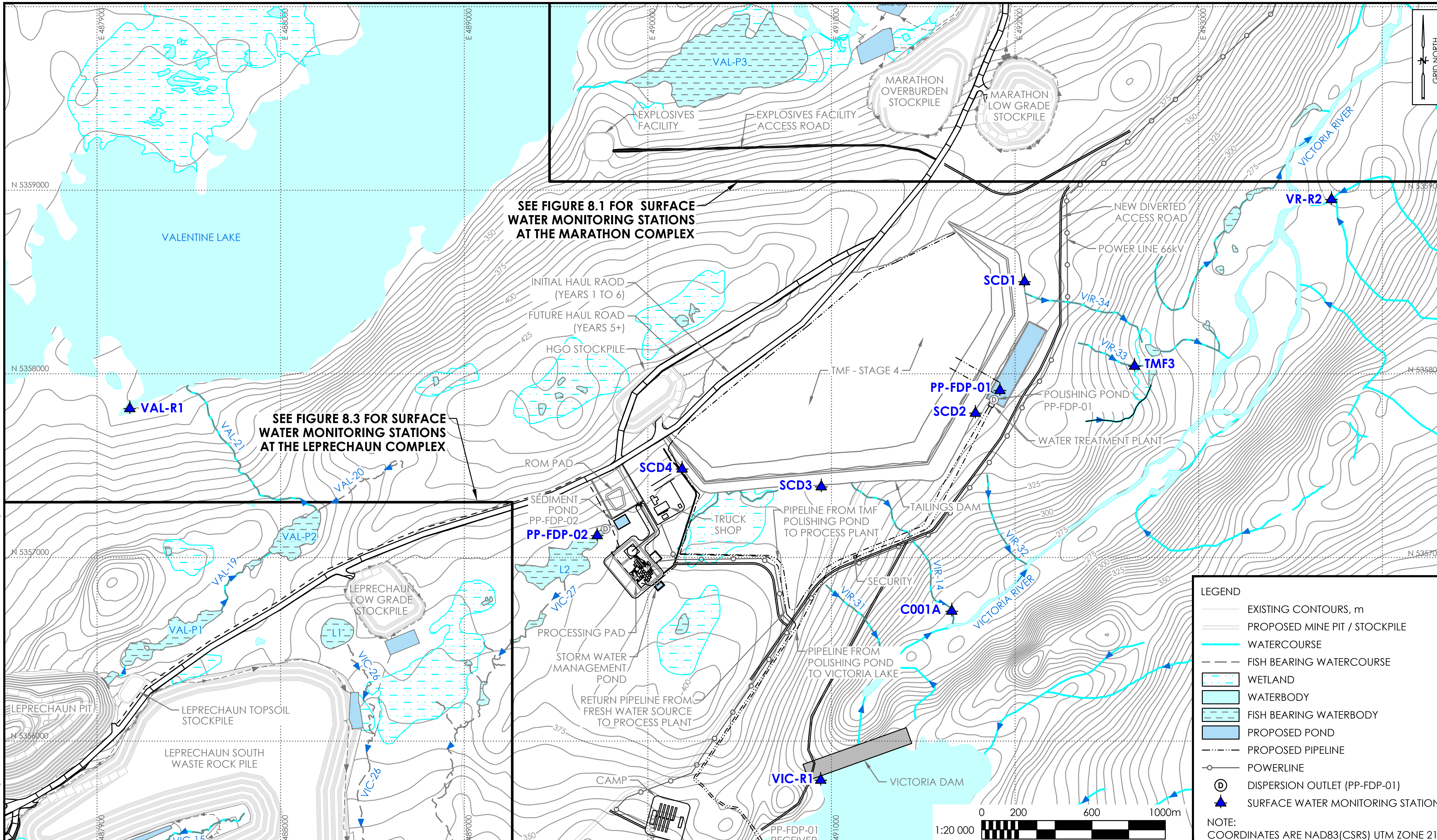

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PRELIMINARY SURFACE WATER MONITORING STATIONS - PROCESSING PLANT & TMF COMPLEX

WATER MANAGEMENT PLAN

VALENTINE GOLD PROJECT, NL

Client: MARATHON GOLD CORP

Job No.:	121416408
Scale:	1 : 20 000
Date:	28-SEP-2020
Dwn. By:	JL
App'd By:	RJ

Fig. No.: 8.2

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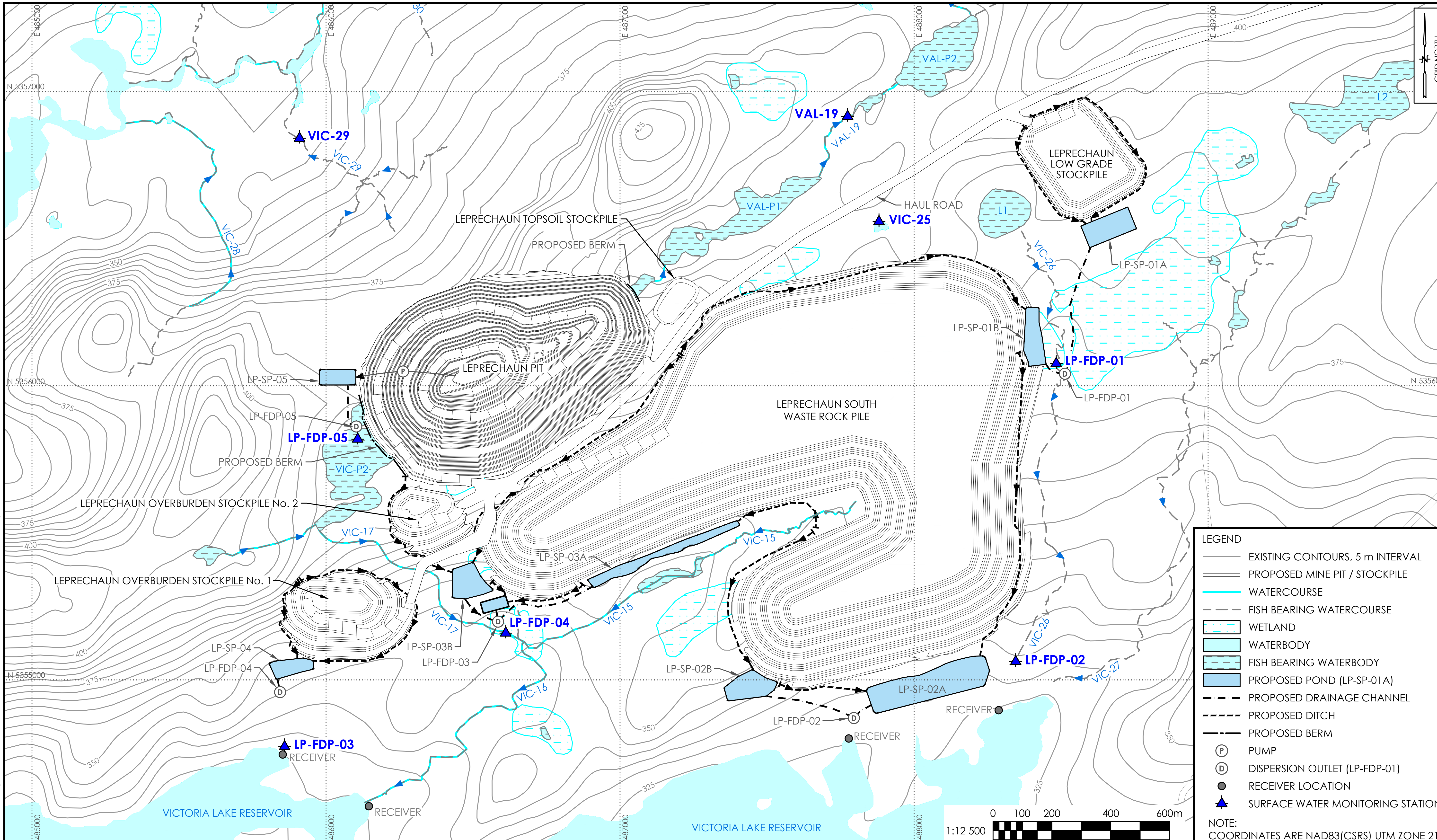

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PRELIMINARY SURFACE WATER MONITORING STATIONS - LEPRECHAUN COMPLEX
WATER MANAGEMENT PLAN
VALENTINE GOLD PROJECT, NL

Client: MARATHON GOLD CORP

Job No.:	121416408
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App'd By:	RJ

Fig. No.: 8.3

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8.3 GROUNDWATER MONITORING

8.3.1 Ground Water Quantity Monitoring

Groundwater is at or close to ground surface over most of the Process Plant and TMF Complex and a layer of glacial till is present over the bedrock. Apart from flow along discrete structural features in the bedrock (e.g., faults, jointing, etc.), it is anticipated that groundwater flow is likely to occur within the overburden, the bedrock/till contact, and within the upper slightly weathered bedrock zone. It is therefore suggested that the monitoring wells be installed to monitor these potential flow pathways. Prior to well installation, the locations and design of the monitoring wells should be reviewed on the basis of information obtained during construction. As necessary, the detected seepage can be directed back into the TMF via pump-back wells.

Potential groundwater interactions at the open pits include seepage into the pit through water-bearing fractures intersecting the pit walls, and gradual lowering of the static water table in bedrock surrounding the open pit due to progressive mine dewatering. Groundwater monitoring wells will be monitored for static water levels on a monthly basis to assess effects to groundwater quantity. Some groundwater monitoring wells will be monitored more frequently in order to develop a relationship of groundwater and surface water. The frequency of water level monitoring will be reviewed after a year of data has been collected and analyzed.

The groundwater levels will identify if the resultant depressed groundwater table is as predicted and if the depression has an influence on stream flows of adjacent waterbodies. Should groundwater monitoring identify impacts to nearby surface water tributary flows, groundwater contingency measures will be implemented to maintain flow. A contingency plan will be developed that outlines emergency response.

8.3.2 Ground Water Quality Monitoring

Groundwater in contact with the mine site will have changes in quality. Some seepage through and under the dams at the TMF can be anticipated. It is expected that the majority of the seepage from the dams can be collected in ditches and conveyed to small sumps and, if necessary, pumped back into the TMF. The remainder would be lost to the groundwater flow regime.

The potential for vertical seepage pathways and thrust faults may result in groundwater in contact with the open pit to interact with adjacent surface water bodies. Other potential groundwater effects at the site may include accidental release of petroleum hydrocarbon or mill processing chemicals into groundwater. Therefore, groundwater quality will be monitored to identify changes in water quality in down-gradient wells due to rechange of runoff from the site, identify interaction with surface water body, identify areas of seepage and/or to support calibration of the seepage models, identify an accidental release of petroleum hydrocarbon or mill processing chemicals into groundwater, or to identify low grade ARD impacts to groundwater.

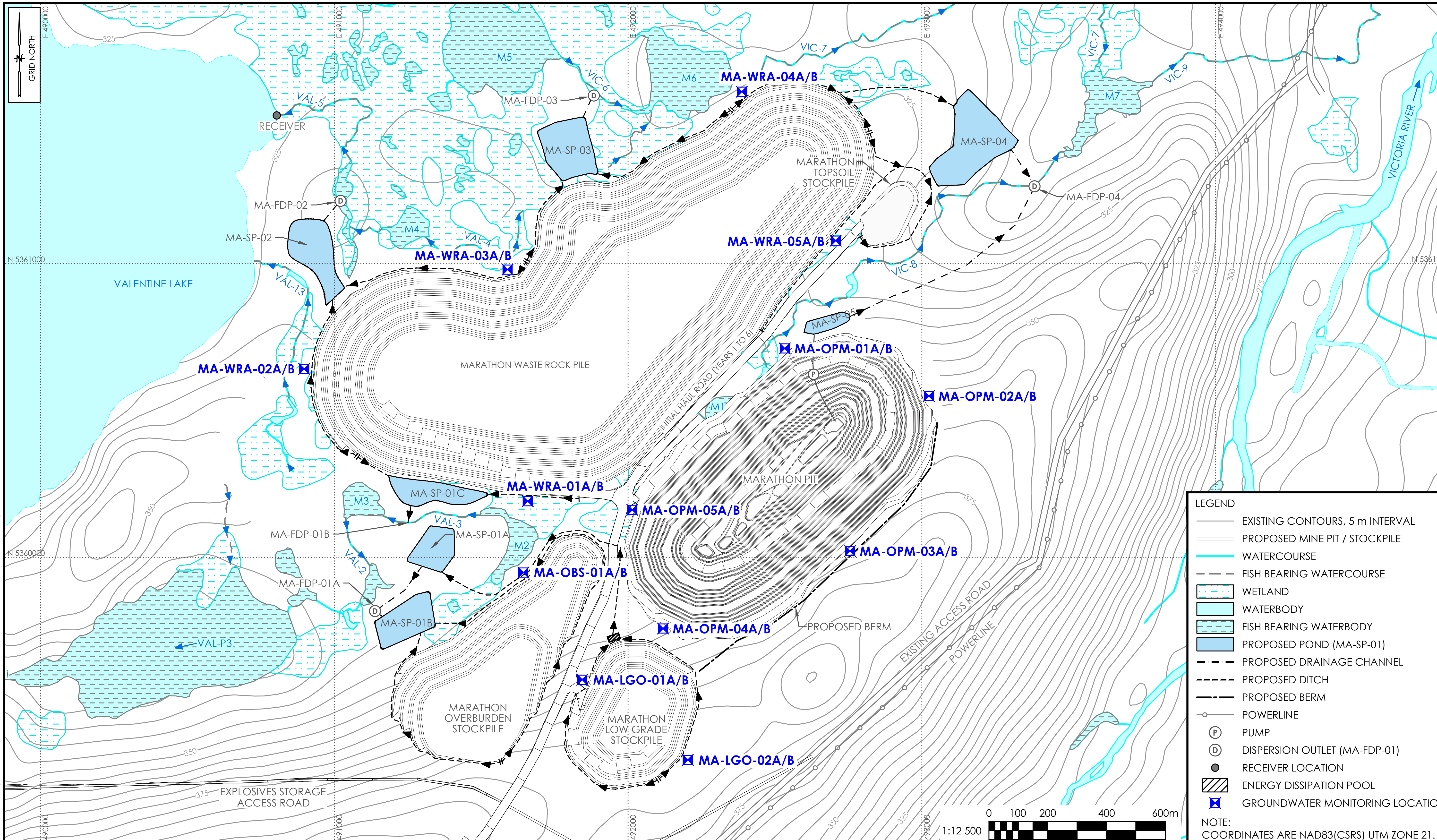
A network of groundwater monitoring stations will be located around the perimeter of each Project facility during the initial construction program. It is recommended that an additional monitoring well be installed outside the TMF to serve as an indicator of background groundwater quality. The location and number of additional monitoring wells to be installed in the future shall be determined based on the performance and results of the initial monitoring wells. The proposed groundwater monitoring network is described in Table 8.2 and presented on Figures 8.4 to 8.6 and will be refined as on-going groundwater monitoring and intrusive investigations continue.

Groundwater monitoring will be conducted during pre-development, construction, operation, and closure stages. Monitoring and maintenance of the reclaimed facilities will be carried out during operations and into closure. It is anticipated that monitoring and maintenance will be carried out during the active closure stage at frequencies similar to those required during operations. Post-closure monitoring and maintenance will be carried out at a reduced frequency depending on the results of the monitoring and the measures of success selected for closure.

Periodic environment effects monitoring (EEM) as required under MDMER will be conducted.

Table 8.2 Preliminary Groundwater Monitoring Stations

Area	Groundwater Monitoring Station	Monitoring Frequency
Marathon Pit	MA-OPM-01A/B, MA-OPM-02A/B, MA-OPM-03 A/B, MA-OPM-04 A/B,	Monthly
Marathon Waste Rock Pile	MA-WRA-01 A/B, MA-WRA-02 A/B, MA-WRA-03 A/B, MA-WRA-04 A/B, MA-WRA-05 A/B	Quarterly
Marathon Low Grade Ore Stockpile	MA-LGO-01A/B, MA-LGO-02 A/B	Quarterly
Marathon Overburden Stockpile	MA-OBS-01A/B	Quarterly
Leprechaun Pit	LP-OPM-01 A/B, LP-OPM-02 A/B, LP-OPM-03 A/B, LP-OPM-04 A/B, LP-OPM-05 A/B	Monthly
Leprechaun Waste Rock Pile	LP-WRP-01A/B, LP-WRP-02A/B, LP-WRP-03A/B, LP-WRP-04A/B, LP-WRP-05A/B,	Quarterly
Leprechaun Low Grade Stockpile	LP-LGO-01A/B, LP-LGO-02 A/B	Quarterly
Leprechaun Overburden Stockpile	LP-OBS-01A/B	Quarterly
Leprechaun Topsoil Stockpile	LP-TSS-01A/B	Quarterly
Process Plant	PP-PPP-01A/B, PP-PPP-02A/B, PP-PPP-03A/B, PP-PPP-04A/B	Quarterly
Tailings Management Facility	PP-TMF-01A/B, PP-TMF-02A/B, PP-TMF-03A/B, PP-TMF-04A/B, PP-TMF-05A/B	Quarterly
High Grade Ore Pile	PP-HGO-01A/B, PP-HGO-02A/B	Quarterly
Groundwater Water Quality – Parameters to be Monitored		
Total Aluminum, Total Arsenic, Total Cadmium, Total Copper, Cyanide, WAD Cyanide, Flouride, Total Fluoride, Total Iron, Total Lead, Total Manganese, Nitrite, Nitrogen Ammonia, Unionized Ammonia, pH, Phosphorus, Sulphate, Total Suspended Solids, Total and Dissolved Zinc, Hardness, and Sodium. In situ field measurements: Temperature, pH, Conductivity		
Note: Monitoring locations are preliminary and will be refined as design progresses in consultation with regulators and in accordance with permits and approval.		



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PRELIMINARY GROUNDWATER MONITORING LOCATIONS - MARATHON COMPLEX

WATER MANAGEMENT PLAN

VALENTINE GOLD PROJECT, NL

Client: MARATHON GOLD CORP

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Fig. No.: 8.4

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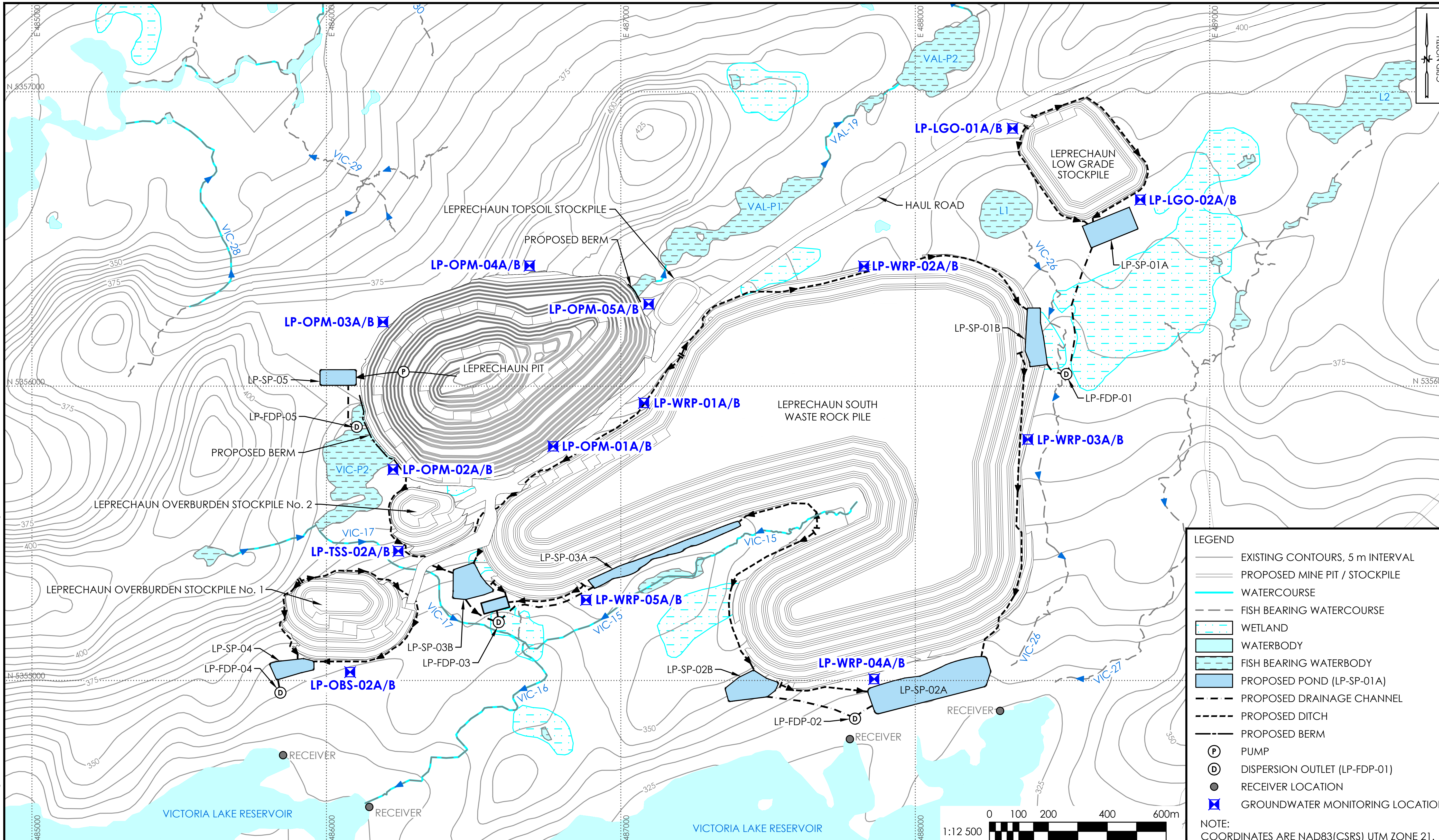

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PRELIMINARY GROUNDWATER MONITORING LOCATIONS - LEPRECHAUN COMPLEX

WATER MANAGEMENT PLAN

VALENTINE GOLD PROJECT, NL

Client: MARATHON GOLD CORP

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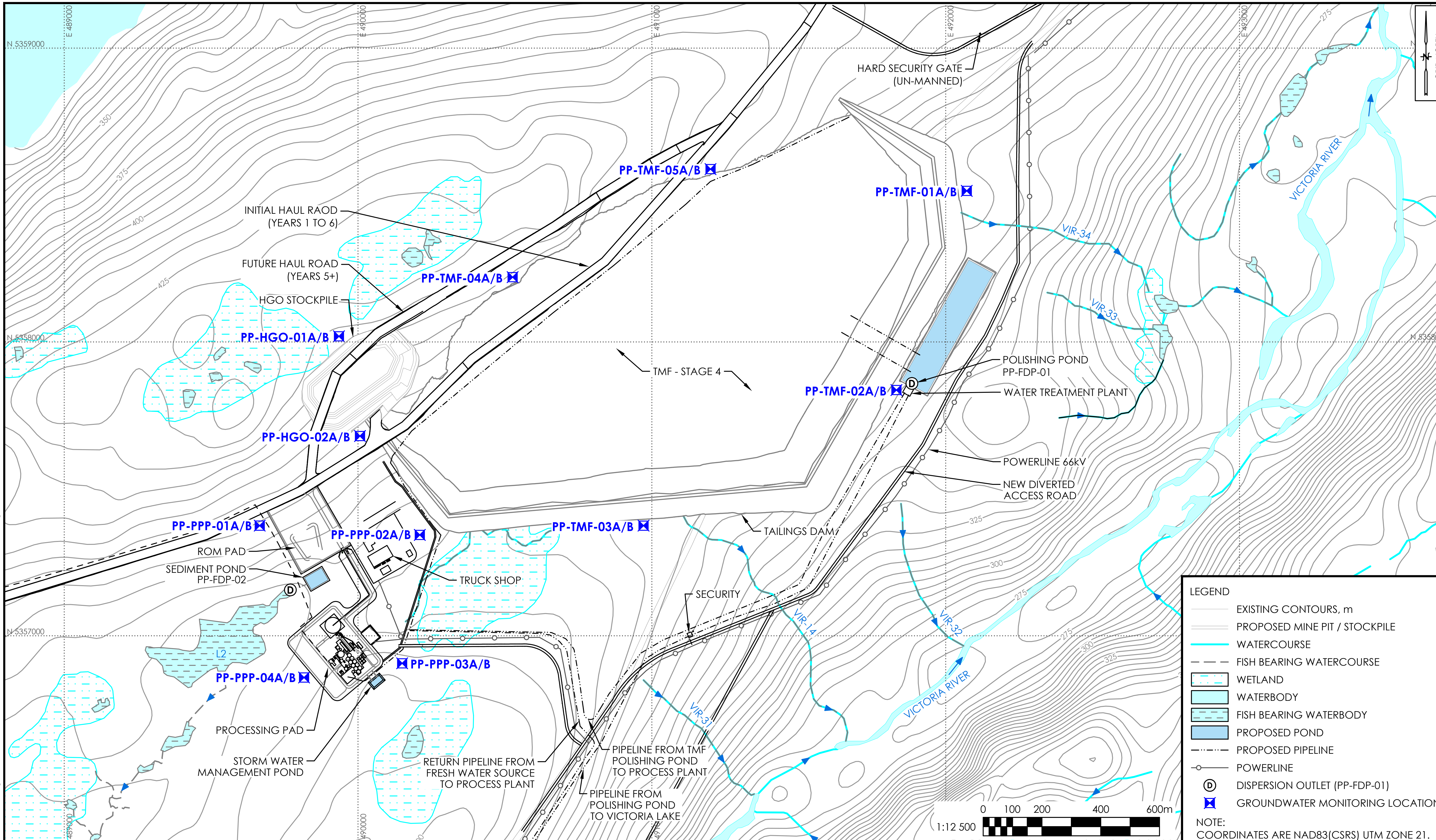

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PRELIMINARY GROUNDWATER MONITORING LOCATIONS - PROCESS PLANT AND TMF COMPLEX

WATER MANAGEMENT PLAN

VALENTINE GOLD PROJECT, NL

Client: MARATHON GOLD CORP

Job No.:	121416408
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8.4 CLOSURE MONITORING

Surface water and groundwater monitoring will continue into closure and post-closure. The objective of the monitoring will be to determine if the rehabilitation measures were successful and the Project produces stable runoff and seepage quality compliant with regulatory closure regulations. The monitoring frequency will continue as per operation and will be revisited one year into closure.

The proposed closure monitoring and maintenance activities include visual inspections of reclaimed areas to identify unstable areas, maintain all facilities and equipment to be used during closure until they are no longer required, install instrumentation at selected facilities for monitoring of the reclaimed areas, and test surface and groundwater quality and measure water volumes at select locations to confirm that the closure measurements are performing as predicted and are not adversely affecting the environment as required by the Newfoundland and Labrador Mine Regulation 42/00.

9.0 REFERENCES

BC Ministry of Environment and Climate Change Strategy (BCMELP). 2017. "British Columbia Approved Water Quality Guideline: Aquatic Life, Wildlife, & Agriculture - Summary Report." https://www2.gov.bc.ca/assets/gov/environment/air-land-water/water/waterquality/water-quality-guidelines/approved-wqgs/wqg_summary_aquaticlife_wildlife_agri.pdf.

Canadian Council of Ministers of the Environment (CCME). 2010. Canadian water quality guidelines for the protection of aquatic life: Ammonia. In: Canadian environmental quality guidelines, 1999, Canadian Council of Ministers of the Environment, Winnipeg.

Canadian Council of Ministers of the Environment (CCME). 2019. Canadian water quality guidelines for the protection of aquatic life: Manganese (Dissolved) 2019. In: Canadian environmental quality guidelines, Canadian Council of Ministers of the Environment, Winnipeg.

Canadian Council of Ministers of the Environment (CCME). 2020. Canadian water quality guidelines for the protection of aquatic life: Summary Table. In: Canadian environmental quality guidelines, 1999, Canadian Council of Ministers of the Environment, Winnipeg, MB. Accessed from http://st-ts.ccme.ca/en/index.html in January 2020.

Doneker, R.L. and Jirka, G.H. 2017. CORMIX User Manual: A Hydrodynamic Mixing Zone Model and Decision Support for Pollutant Discharges into Surface Waters, Report prepared for U.S. Environmental Protection Agency.

Environment and Climate Change Canada (ECCC). 2020. Metal and Diamond Mining Technical Guidance for Environmental Effects Monitoring Government of Canada (SOR/2002-222).

ECCC. 2019. Engineering Climate Datasets. Retrieved from Environment and Climate Change Canada: https://climate.weather.gc.ca/prods_servs/engineering_e.html

Environment and Climate Change Canada (ECCC). (2019). Canadian Climate Normals 1981-2010 Buchans. Retrieved from Climate Normals and Averages: https://climate.weather.gc.ca/climate_normals/results_1981_2010_e.html?stnID=6579&autofwd=1

ECCC (Environment and Climate Change Canada). 2020. National Long-term Water Quality Monitoring Data. Retrieved from Environment and Climate Change Canada: https://open.canada.ca/data/en/fgpv_vpgf/67b44816-9764-4609-ace1-68dc1764e9ea

Government of Newfoundland and Labrador. 2000. Newfoundland and Labardor regulation 42/00 - Mining Regulations under the *Mining Act.*

Government of NL. 2004 – Typical Silt Fence Installation Detail -1260. Available at: https://www.gov.nl.ca/mae/capital-works/standard-drawings-previous/ . Cited July 2020.

Government of NL. 2020. Environmental Impact Statement Guidelines for the Valentine Gold Project. Prepared by the 2015 Environment Assessment Committee for the Minister of the Department of Municipal Affairs and Environment, Government of Newfoundland and Labrador.

Golder Associates Ltd. (Golder). 2020. Marathon Gold Prefeasibility Study for Tailings Disposal at the Valentine Gold Project, Newfoundland. Mississauga, Ontario.

Marathon Gold & Ausenco. 2020. N.I. 43-101 Technical Report & Pre-Feasibility Study on the Valentine Gold Project, Newfoundland and Labrador, Canada.

McCabe, Gregory J, and Steven L. Markstrom. 2007. A monthly Water-Balance Model Driven By a Graphical User Interface. Open-file report, Reston, Virginia: US Geological Survey.

MDMER. 2019. "Metal and Diamond Mining Effluent Regulations." Fisheries Act - Registration SOR/2002-222. 06 25. https://laws-lois.justice.gc.ca/eng/regulations/sor-2002-222/FullText.html.

Meays, C., and R. Nordin. 2013. "Ambient Water Quality Guidelines for Sulphate. Technical Appendix, Water Protection and Sustainability Branch, Environmental Sustainability and Strategic Policy Division." BC Ministry of Environment.

MOE (Ontario Ministry of Environment). 2003. Stormwater Management Planning and Design Manual, Ministry of Environment, Ontario.

Newfoundland and Labrador Department of Municipal Affairs and Environment (NLDOEC). 2014. "Policy for Flood Management." Newfoundland and Labrador Department of Municipal Affairs and Environment. March 19. https://www.mae.gov.nl.ca/waterres/regulations/policies/flood_plain.html.

Read, W.S. & Cole, L.J. 1972. The Bay D'Espoir Hydro Electric Development. Newfoundland and Labrador Power Commission.

Stantec Consulting Ltd. 2017. Valentine Lake Project: Preliminary Baseline Hydrogeology Assessment. Project #: 121415923.

Stantec Consulting Ltd. 2019. Valentine Lake Project: Preliminary Baseline Hydrogeology Assessment, Water Level Data. Project #: 121415923.

Stantec. 2020a. Valentine Gold Project: 2019 Baseline Hydrology and Surface Water Quality Monitoring Program. Markham, Ontario: Stantec Consulting Ltd.

Stantec. 2020b. Valentine Gold Project: Basis of Design for Pre-Feasibility Level Water Management Design - Final.: Stantec Consulting Ltd.

Stantec. 2020c. Valentine Gold Project (VGP): Valentine Gold Project: Acid Rock Drainage/Metal Leaching (ARD/ML) assessment report. September 2020.

Stantec 2020d. Valentine Lake Gold Project – Fish and Fish Habitat Valued Component Chapter.

Stantec 2020e. Valentine Lake Gold Project – Marathon Water Quantity & Water Quality Modelling Report

Stantec 2020f. Valentine Lake Gold Project – Leprechaun Water Quantity & Water Quality Modelling Report

Stantec 2020g. Valentine Lake Gold Project - Assimilative Capacity Assessment

Stantec. 2020h. Valentine Gold Project (VGP): Valentine Gold Project: Acid Rock Drainage/Metal Leaching (ARD/ML) assessment report (Phase II). September 2020.

Simons, Li & Associates Inc. 1982. Design Manual for Sedimentation Control Through Sedimentation Ponds and Other Physical/Chemical Treatments, prepared for U.S. Department of the Interior Office of Surface Mining.

USACE (United States Army Corps of Engineers). 2000. Hydrologic Modeling System HEC-HMS, Technical Reference Manual, US Army Corps of Engineers, Institute of Water Resources, Hydrologic Engineering Center, California, US.

USDA (United States Department of Agriculture). 2004. Chapter 11 Snowmelt. Part 630 Hydrology National Engineering Handbook. Natural Resources Conservation Service.

USACE (United States Army Corps of Engineers). 2010. Hydrologic Modeling System HEC-HMS, User's Manual, US Army Corps of Engineers, Institute of Water Resources, Hydrologic Engineering Center, California, US.

APPENDIX 2B

Prefeasibility Study for Tailings Disposal at the Valentine Gold Project, Newfoundland (Golder)

GOLDER

REPORT

Marathon Gold

Prefeasibility Study for Tailings Disposal at the Valentine Gold Project, Newfoundland

Submitted to:

Marathon Gold

10 King St E, Suite 501, Toronto, ON, M5C 1C3

Submitted by:

Golder Associates Ltd.

6925 Century Avenue, Suite #100, Mississauga, Ontario, L5N 7K2, Canada

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June 2020

Distribution List

1 e-copy to Marathon Gold

1 e-copy to Ausenco

Executive Summary

Marathon Gold engaged Golder Associates Ltd. to complete a pre-feasibility level TMF design for the Valentine Gold Project in south central Newfoundland. The following studies and analyses were completed to support the PFS level design:

- TMF site selection assessment including a fatal flaw screening of sites, option development and trade-off study (supported with process design and cost estimation from Ausenco).
- Qualitative dam hazard classification and establishment of applicable design criteria.
- Hydrology review and TMF water balance considering wet and average conditions.
- Dam design including slope stability and seepage / leakage rate assessment.
- Tailings deposition planning for various development stages of the TMF.
- Water management strategy for the TMF including hydraulic design of the conveyance structures (seepage and run-off collection ditches, emergency spillway, etc.).
- Construction quantities and cost estimates for the TMF over the life of mine

The options trade-off study considered operational, technical, environmental, social and financial aspects. The preferred site was selected between the Marathon and Leprechaun pits, south of the thrust fault and to the east of the process plant. Thickened tailings was selected as the preferred tailings deposition method.

The TMF has a preliminary design to accommodate 30 Mt of tailings material that will be produced over the initial 9 years of the mine life. Tailings will subsequently be deposited in the mined-out Leprechaun Open Pit from Year 10 to Year 12. The design is based on the annual Mill throughput which ramps up from 1.875 Mtpa in Year 1 to 4.0 Mtpa in Year 5.

The dam safety program established in Newfoundland requires that dams must be designed, operated and maintained to meet the requirements of Canadian Dam Association (CDA) Dam Safety Guidelines. In accordance with the dam classification methodology presented in the CDA Dam Safety Guidelines, the proposed TSF dams have been provisionally classified as a “Very High” consequence of failure, based on the potential environmental impact and population at risk. The design of the TSF was carried out to meet minimum allowable factors of safety under static and pseudo-static loading conditions recommended in the current CDA Dam Safety Guidelines. Seepage and stability analyses were carried out as part of the design. Based on the model results, the dams are expected to be stable under the assumed loading and expected foundation conditions.

Preliminary and limited geotechnical investigations have been completed to date at the proposed TMF site. A review of available surficial geology mapping for the project area indicates that the dominant subsurface material in the TMF footprint is glacial till occurring mainly as hummocky and blanket deposits with thicknesses up to 15 m or as thin discontinuous veneer (typically less than 1.5 m thick) overlying bedrock. The area of high ground along the crest of the ridge at the north limit of the TSF is characterized by bedrock outcrop either exposed above the till veneer or concealed by vegetation. Bogs are present in poorly drained areas. Finite sampling from borehole drilling suggests the till is primarily granular and non-cohesive in nature, comprising silt, sand, and gravel containing cobbles and boulders. It is a requirement that the TMF dams are founded on compact to dense native tills and/or bedrock.

The overall design objective of the TMF is to protect the regional groundwater and surface water resources during both operations and long term (post-closure), and to achieve effective reclamation upon mine closure. A staged TMF development with embankments raised in a downstream direction was selected. Mine waste rock from the pit developments will be the primary embankment construction material. The rockfill will be placed as part of the mining operation and for the intermediate stages a crest width of 20 m was selected to allow for mine vehicle and equipment access during construction. The embankment has a 3.5H:1V upstream slope and 2.0H:1V downstream slope with a maximum height of 49 m. The upstream embankment will be lined with a geomembrane to minimize potential seepage and underlain by a geotextile, a fine filter layer and a coarse filter layer. The filter zones will be processed material sourced from local borrow areas. Excess water within the TMF will be controlled, collected and recycled to the process plant to the maximum practical extent. An emergency discharge spillway and runout channel are provided for each embankment raise. Rip-rap lined seepage collection ditches are provided along the toe of the embankment and these report to a downstream settling pond. Closure will include a vegetated overburden cover over the tailings

The operational plan for the TSF is to deposit slurry via spigots primarily from the natural high ground on the north west side of the TSF and secondly from the perimeter embankment. This will allow the tailings pond to be located on the east side of the TSF and a tailings beach will form that slopes from the deposition points along the high ground down to the perimeter embankment. The TSF will be monitored to demonstrate performance goals are achieved and design criteria and assumptions are met. The perimeter embankment will be raised in stages to provide the necessary storage.

The accumulation of water in the TSF has been modelled for the mean and 25-year wet annual precipitation conditions. The site has a positive water balance i.e. rainfall exceeds evaporation. The TSF will receive rainwater and the process water discharged with the tailings slurry. Excess water from the open pit dewatering and run-off from waste rock stockpiles are managed separately and do not report to the TSF. A water treatment plant and polishing pond allow for the treatment and discharge of the excess site water to Victoria Lake. Treatment and discharge will only occur for 8 months a year. The TSF pond, with a maximum storage capacity of 1 Mm^3, has been sized to store the excess site water during the non-discharge period. Reclaim water is pumped from a floating barge and pump in the TSF to the process plant.

The following activities are recommended to support the design of the TMF as it advances to the feasibility level study:

- Geotechnical site investigations at the preferred TMF site to characterize the foundation conditions associated with the proposed infrastructure.
- Geotechnical investigations within the property boundary to identify potential borrow sources and requirements for development of the borrow areas.
- In-situ permeability tests of the overburden soils and bedrock beneath the proposed dam foundations. The results of the investigation shall be used to evaluate the proposed dam design and seepage cut-off requirements (i.e. bedrock grouting).
- A site-specific seismic hazard assessment to inform the input parameters for dam stability assessment.
- Develop a groundwater model to evaluate the impacts of the TMF on the local environment. The model should also address the impacts of in-pit disposal of tailings in the mined-out Leprechaun pit during the latter years of operation.

- Tailings testing to determine the geotechnical properties to understand the settlement, permeability and deposition characteristics.
- Optimization and further evaluation of the proposed dam alignment, deposition planning, and construction staging based on the findings of the geotechnical site investigations and other project developments.
- A dam breach and inundation study to support the dam classification.
- Further refinement of the TMF and site wide water balance.
- Optimize the location and design of the Polishing Pond.
- Advancement of the closure cover design criteria and success attributes to optimize the reclamation requirements.
- A site wide material balance, especially for the pre-production and closure periods to confirm suitable availability of construction materials.
- Condemnation drilling for the TMF site to verify the absence of mineralization

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1.0 INTRODUCTION

Golder Associates Ltd. (Golder) has been retained by Marathon Gold Corporation (Marathon) to complete a pre-feasibility Study (PFS) level design of a Tailings Management Facility (TMF) for the Valentine Gold Project, in south central Newfoundland located as shown on Figure 1. This follows the Preliminary Economic Assessment (PEA) completed in 2018. Marathon has also retained Ausenco for the design of the process plant and associated site infrastructure, Moose Mountain Technical Services for mine planning, Stantec for the environmental studies and site wide water balance and JT Boyd for the mineral resource study.

This report presents the PFS level design for the TMF. The design and analyses were completed for the latest mine plan which accounts for a total resource tonnage of approximately 40.7 million tonnes (Mt) The following studies and analyses were completed to support the PFS level design of the TMF:

- TMF site selection assessment including a fatal flaw screening of sites, option development and trade-off study (supported with process design and cost estimation from Ausenco).
- Qualitative dam hazard classification and establishment of applicable design criteria.
- Hydrology review and TMF water balance considering wet and average conditions.
- Dam design including slope stability and seepage / leakage rate assessment.
- Tailings deposition planning for various development stages of the TMF.
- Water management strategy for the TMF including hydraulic design of the conveyance structures (seepage and run-off collection ditches, emergency spillway, etc.).
- Construction quantities and cost estimates for the TMF over the life of mine.

2.0 SITE AND PROJECT INFORMATION

The site is located 57 km south of Buchans, 340 km northwest of St John's and within the Central Uplands of Newfoundland (Figure 1). It is accessed by a 73 km long, well-maintained gravel road from Millertown to the northeast of the site. The site is situated amidst gentle to moderately steep, hilly terrain and the ground surface elevation ranges from approximately 320 m to 480 m above sea level (masl). A distinct northeast trending ridge occurs along the length of the property (Figure 2). The ground cover consists of a mixture of boggy ground, spruce and fir forests, and grassy clearings with many small ponds and streams. Victoria Lake is adjacent to the site and is contained by Victoria Dam which is a hydroelectric reservoir. Valentine Lake lies north of the site.

The Valentine Gold Project consists of four primary ore deposits known as Marathon, Leprechaun, Sprite and Victory. The current Valentine Gold Project will develop the Marathon and Leprechaun deposits as part of the PFS. The Marathon and Leprechaun ore bodies are about 5 km apart. No historical mining activities have occurred at either deposits and Marathon Gold intends to construct, operate and eventually close/reclaim open pit gold mines at these sites.

Figure 2 provides a general overall site plan at for the Valentine Gold Project. The project components and activities at the site will include the construction, operation and eventual decommissioning and closure of the following key elements:

- The Leprechaun and Marathon open pits
- Mineral processing infrastructure and site buildings
- Overburden and topsoil stockpiles
- Waste rock stockpiles
- Low grade ore stockpiles
- Tailings management facility
- Polishing pond
- Water treatment plant
- Access roads
- Power / transmission lines
- Mine camp
- Other ancillary infrastructure and equipment.

3.0 BACKGROUND INFORMATION

3.1 Previous Studies and Reports

The previous studies and reports related to the TMF are listed below.

- Preliminary Economic Assessment (PEA) (Lycopodium, 2018)
- Baseline hydrology and surface water quality (Stantec, 2017a)
- Water balance for site (in support of the PEA) (Stantec, 2018a)
- Geochemistry evaluation for Acid Rock Drainage/Metal Leaching (ARD/ML) (Stantec, 2018b)
- Tailings facility site selection study (Stantec, 2018c)
- Baseline hydrogeological characterization (Stantec, 2017b and GEMTEC, 2019)
- Environmental surveys and baseline studies for:
  - Fish and fish habitat (Stantec, 2012b, 2019)
  - Forest songbirds (Stantec, 2012a)
  - Waterfowl (Stantec, 2012c)

As well, the following studies have been carried out and referenced in Marathon's October 30, 2018 National Instrument 43-101 report prepared by Lycopodium Minerals Canada (Lycopodium, 2018). Golder does not have access to these reports but understand from Marathon that they will not alter the design of the TMF.

- Winter Wildlife
- Ecological Land Classification
- Vegetation – Rare Plants
- Land and Resource Use

3.2 General Site Characteristics

The general site characteristics and properties described in this section of the report have bene summarized from existing information supplemented with information obtained specifically for use in the PFS design of the TMF.

3.3 Meteorology

The Project is in a climatic region characterized by cool summers and cold winters with moderate precipitation. Long term climate data corresponding to the 1981-2010 Climate Normals from Environment Canada's Buchans station (ID: 8400698) indicates that the mean annual air temperature is 3.8°C. The minimum and maximum monthly mean temperatures during 1981-2010 were -8.4°C and 16.3°C for February and July, respectively. There is an average annual precipitation of 1236.2 mm (877 mm as rain and 359.3 cm of snow). The Buchans station is located at Latitude 48°49'00'' N Longitude 56°52'00'' W and an Elevation of 269.70 masl, approximately 56 km northeast of the Valentine site. Long term climate data corresponding to the 1981-2010 Climate Normals from Environment Canada's Stephenville Airport station(ID: 8403800) (Latitude 48°32'00" N Longitude 58°33'00'' Elevation 24.70 masl) and Gander Airport station (ID: 8401700) (Latitude 48°56'47" N Longitude 54°34'37'' Elevation 151.20 masl) indicates that the mean annual lake evaporation is 387.3 mm (Stantec, 2018a).

Table 1 summarizes long-term average monthly and annual total precipitation and potential lake evaporation.

Table 1: Precipitation, Temperature and Evaporation Climate Normals for Buchans, NL

Month	Average Precipitation (mm)	Average Rainfall (mm)	Average Snowfall (cm)	Average Temperature (°C)	Lake Evaporation[1] (mm)
Jan	122	33.7	88.3	-8.2	-
Feb	98.1	25.6	72.5	-8.4	-
Mar	95	39.5	55.5	-4.8	-
Apr	85.7	59.5	26.2	1	-
May	86.6	82.2	4.4	7	71.6
Jun	87.8	87.7	0.1	12.1	77.4
Jul	95.3	95.3	0	16.3	84.8
Aug	123	123	0	16.2	74.0
Sep	110.4	110.3	0.1	11.9	48.5
Oct	97.5	92.5	5	6	31.0
Nov	111.8	81.5	30.4	0.5	-
Dec	123.1	46.3	76.9	-4.5	-
Annual	1236.2	877	359.3	3.8	387.3

1. Lake Evaporation is based on climate stations at Stephenville Airport and Gander Airport.

Table 2 summarizes the rainfall storm frequency values from the Environment Canada rainfall Intensity-Duration-Frequency (IDF) data for the Deer Lake Station. The Deer Lake Station is located approximately 90 km northwest of the Valentine Site. IDF data was not available for the Buchans Station (Stantec, 2017).

Table 2: Rainfall Storm Frequency Values for the Deer Lake Station

Duration	Return Period Frequency (years)					
	2-yr	5-yr	10-yr	25-yr	50-yr	100-yr
	Rainfall (mm)					
5 min	3.9	5.8	7.0	8.6	9.7	10.8
10 min	5.7	8.1	9.7	11.7	13.2	14.7
15 min	7.0	9.6	11.3	13.5	15.1	16.7
30 min	9.8	12.7	14.6	17.0	18.7	20.5
1 h	13.4	16.8	19.1	21.9	24.1	26.2
2 h	18.8	24.4	28.1	32.8	36.3	39.7
6 h	29.7	36.4	40.8	46.4	50.5	54.6
12 h	36.9	44.7	49.8	56.3	61.2	65.9
24 h	42.9	51.9	57.8	65.3	70.9	76.4

3.4 Geology

3.4.1 Regional Bedrock Geology

The following description for the bedrock geology of the site was adopted from the PEA (Lycopodium, 2018). The project area is located within the Exploits subzone, which is largely composed of Victoria Lake Group volcanic and epiclastic rocks, intruded by Cambrian to Silurian granitoid and gabbroic intrusions. The volcano-sedimentary sequence strikes northeast, dips sub-vertically, and displays a regional low grade greenschist metamorphic assemblage. The Exploits subzone is bound to the north by the Red Indian Line, a major Appalachian crustal suture in Newfoundland between the Exploits and Notre Dame subzones, and to the south by a regional scale thrust fault.

The Project is centred on the large multiphase, trondhjemite, quartz monazite, and gabbro Valentine Lake Intrusive complex (VLIC), which is dated at 563 ± 2 Ma (U-Pb zircons; Evans et al., 1990) and forms a structural inlier within the Victoria Lake Group volcano-sedimentary rocks. The VLIC is situated along the contacts between the Victoria Lake Group (VLG) to the northwest and the Silurian (or younger) Rogerson Lake Conglomerate (RLC) to the southeast. The contact between the VLG and RLC is the northeast-southwest, regional crustal-scale, subvertical to steeply northwest dipping Valentine Lake thrust fault (Valentine Lake Shear Zone) which forms the south-eastern extent of the Exploits subzone. The VLIC occurs at a flexure in this regional structure, where the strike of the thrust fault steepens to north-northeast.

Additional commentary on the local bedrock geology at the site is provided in the PEA (Lycopodium, 2018).

3.4.2 Surficial Geology

A review of available surficial geology mapping for the project area indicates that the dominant subsurface material in the TMF footprint is glacial till occurring mainly as hummocky and blanket deposits with thicknesses up to 15 m or as thin discontinuous veneer (typically less than 1.5 m thick) overlying bedrock (Smith, 2011). Finite sampling from borehole drilling suggests the till is primarily granular and non-cohesive in nature, comprising silt, sand, and gravel containing cobbles and boulders. Sand and gravel deposits of glacial outwash and fluvial origin are locally present and confined to the Victoria River valley. The area of high ground along the crest of the ridge at the north limit of the TMF is characterized by bedrock outcrop either exposed above the till veneer or concealed by vegetation. Bogs are present in poorly drained areas.

3.5 Seismicity

A site-specific hazard assessment has not been carried out for the site. The 2015 National Building Code of Canada (NBCC) provides seismic hazard information relating to a maximum AEP for seismic “Site Class C” with an average shear wave velocity, Vs, of 450 m/s for probabilities of 1:2,475 or higher, and are presented in Table 3.

Table 3: Peak Horizontal Ground Acceleration Values for the Valentine Gold TMF site (NBCC, 2015)

Annual Exceedance Probability (AEP)	1:100	1:475	1:1,000	1:2,475
PGA (g)	0.006	0.019	0.028	0.046

3.6 Geochemistry

3.6.1 Waste Rock

Based on acid rock drainage (ARD) testing to date, mine rock has low percentage of PAG (Potentially Acid Generating) samples and samples with uncertain ARD potential from the Marathon and Leprechaun pits are less than 5% and 2%, respectively (Stantec, 2018b). Therefore, Stantec expects the waste rock stockpiles to be non-acid-generating. The Metal leaching (ML) potential of mine rock is currently considered to be low, based on chemistry of leachates from humidity cells and shake flask extracts (SFE). The concentration in these leachates are two to three orders of magnitude below MDMER discharge limits. Further test work is ongoing to confirm the initial test results. Where waste rock will be used for site earthworks and grading during construction and operational development, necessary test work shall be conducted to prevent PAG materials from being used in construction.

3.6.2 Low Grade Ore

The Leprechaun low grade ore (LGO) stockpile is not expected to generate ARD based on three composite samples being non-PAG. One of three LGO composites from Marathon had uncertain ARD potential, but overall the LGO stockpile at Marathon is non-PAG and not expected to generate ARD. ML potential of LGO is currently considered to be low at both deposits because the concentration of metals in leachates from humidity cells and SFE are significantly below MDMER discharge limits (Marathon 2020).

3.6.3 Tailings

To date, ARD/ML test work has shown potential for some high grade ore (HGO) to be potentially acid generating; however, based on the geology, further metallurgical testing and ARD/ML testing on source rock, and lab-scale process tailings, it is expected that the combined tailings will not generate ARD. Humidity cells show that metal concentrations in runoff from tailings beaches will be below MDMER discharge limits. Aging tests of process water indicate that the TMF pond might have exceedances of MDMER for CN total, unionized NH_3 (product of CN decomposition) and Cu (added as catalysis during CN destruction or leached from the ore) during operation. The same parameters might exceed MDMER in seepage according to chemistry of leachates from subaqueous columns (Marathon 2020). Excess water produced by the TMF will be reclaimed to the process plant to offset process water demand and limit volumes of discharge from the TMF pond. TMF excess water that is not reused in ore processing will be treated and discharged to a polishing pond prior to discharge to the environment. Effluent discharged to the environment shall meet MDMER discharge criteria.

Ongoing testing and water balance and quality modelling will support future water management plans.

3.7 Hydrology

Drainage at the Marathon and Leprechaun deposit is into Valentine Lake, Victoria Lake and Victoria River. The proposed TMF location drains into Valentine Lake and the Victoria River. The Victoria River ultimately drains into the Exploits River via Victoria River and Red Indian Lake (Lycopodium, 2018).

4.0 TMF SITE SELECTION AND TRADE OFF STUDY UPDATE

A TMF site selection study was previously completed by Stantec as part of the PEA (Stantec 2018c). An updated siting study was required as part of the current PFS as new information became available with the advancement of the project. The study also considered different tailings dewatering technologies i.e. conventional, thickened and filtered tailings. The extent of tailings dewatering has an impact on the deposition and storage requirements, environmental performance, closure and project economics. The development of tailings disposal options was therefore a combination of site selection, tailings preparation and deposition.

A total of 14 possible candidate sites were identified within an economical transport distance from the mill. The site attributes along with comparative commentary regarding project specific criteria and constraints were provided and presented to Marathon during a conference call held on November 26th, 2019. Fatal flaw criteria were developed and considered, and three locations were designated as potential sites worthy of further consideration.

All three potential sites were within the current project footprint and near other planned mine infrastructure. Preliminary TMF configurations and layouts for conventional slurry disposal, thickened tailings and filtered tailings were developed for all three sites to aid in the comparison. This allowed for the quantification of the major costs items associated with the site development. Capital and operating cost estimates for the TMF dams were estimated by Golder. The process design and delivery along with the associated costs of dewatering technologies was completed by Ausenco.

The preferred site was selected primarily to reduce project risks associated with impacting known fish and fish habitat, avoiding known caribou migration routes and the downstream receiving water course, including the Newfoundland hydro dam (Victoria Lake). Of the three tailings deposition methods considered, filtered tailings presented the lowest environmental and permitting schedule risks and allowed for progressive reclamation during operations, however, it was also the most expensive option. The team concluded that there were no compelling design drivers to warrant filtered tailings disposal for the project. The total estimated cost over the life of mine for thickened tailings deposition was lower than conventional disposal due to lower CapEx associated with smaller containment dams. The preferred disposal method was therefore thickened tailings and this was subsequently carried forward to the pre-feasibility level design.

A technical memorandum was prepared for the updated site selection and trade-off study and this is included in Appendix A.

5.0 DESIGN CRITERIA

5.1 Operating Data

Operating data is required for the sizing of the tailings facility and flow (water balance) modelling. The required operating data is given in detail in the basis of design, included as Appendix B. The key data provided by Ausenco and Marathon Gold are listed below :

- Life of mine is 12 years
- Mill throughput ramps up from 1.875 Mtpa in Year 1 to 4.0 Mtpa in Year 5.
- Total tonnage of tailings produced is 40.68 million tonnes
- Tailings disposal location: Year 1 to 9 TMF, Year 10 to 12 Leprechaun Pit
- Total tonnage of tailings to TMF is 30.13 million tonnes
- Tailings specific gravity = 2.68
- Tailings particle size: P_{80} = 75 µm, non-plastic (Year 1 and 2), P_{80} = 150 µm, non-plastic (Year 3 onwards)
- Tailings discharge solids content = 65% (by mass)

5.2 Dam Hazard Classification

The dam safety program established in Newfoundland requires that dams must be designed, operated and maintained to meet the requirements of Canadian Dam Association (CDA) Dam Safety Guidelines. The TMF dams must be classified based the Canadian Dam Association Dam Safety Guidelines (CDA, 2013, 2014). The CDA guidelines provide recommendations on the classification of dams with respect to the consequences associated with a presumed dam failure. These consequences are to be evaluated in terms of incremental consequences over and above the consequences of the given event if the dam failure had not occurred. The guidelines recommend that the incremental consequences of a dam failure should be evaluated in terms of:

- Loss of life
- Property losses
- Environmental losses
- Cultural or built heritage losses

In accordance with the dam classification methodology presented in the CDA Dam Safety Guidelines, the proposed TMF dams have been provisionally classified as a “Very High” consequence of failure during the operations based on the following:

- Population at risk – Permanent workers in the process plant and the tailings area
- Incremental Losses:
  - Loss of life: 100 or fewer.
  - Environmental and cultural values – significant loss or deterioration of critical fish or of wildlife habitat in the Victoria River and potentially further downstream.
  - Infrastructure and economics – low economic losses; area contains limited infrastructure or services.

5.3 Design Parameters

5.3.1 Inflow Design Flood

The Inflow Design Flood (IDF) is the most severe inflow flood for which a dam and its associated facilities are designed. From a safety design perspective, the IDF is the runoff resulting from the largest discrete storm event that can be safely routed through a basin without overtopping a dam.

Based on CDA recommendations for the selection of an appropriate IDF (Table 4), the IDF of the tailings facility during operations should be determined based on an annual exceedance probability (AEP) of 2/3rd between the 1 in 1,000 year storm and the probable maximum flood (PMF). For passive closure phase, the IDF is raised the PMF.

Table 4: **CDA (2014) Minimum** Inflow Design Floods for Dams and Dykes

Dam Class	Inflow Design Flood (IDF) Annual Exceedance Probability – Operating Phase	Inflow Design Flood (IDF) Annual Exceedance Probability – Closure Passive Care Phase
Low	1 / 100	1/1000
Significant	1 / 100 to 1 / 1 000	1/3 between 1 / 1 000 and PMF[1]
High	1/3 between 1 / 1 000 and PMF[1]	2/3 between 1 / 1 000 and PMF[1]
Very High	2/3 between 1 / 1 000 and PMF[1]	PMF[1]
Extreme	PMF[1]	PMF[1]

1) PMF is the Probable Maximum Flood.

5.3.2 Seismic Design

Table 5 provides annual exceedance probability (AEP) earthquakes for the various dam classes for both operations and closure as per CDA (2014).

Table 5: CDA 2014 AEP Earthquakes for Dams and Dykes - Operation and Closure Phases

Dam Class	Annual Exceedance Probability (AEP) Earthquake – Operations Phase	Annual Exceedance Probability (AEP) Earthquake – Closure Passive Care Phase
Low	1/100	1/1,000
Significant	1/100 to 1/1,000	1/2,475
High	1/2,475	1/2 between 1/2,475 and 1/10,000 AEP or MCE
Very High	1/2 between 1/2,475 and 1/10,000 AEP or MCE	1/10,000 AEP or MCE
Extreme	1/10,000 AEP or MCE	1/10,000 AEP or MCE

1) MCE is the Maximum Credible Earthquake.

The design earthquake for a “Very High” classification tailings dam during the operating phase is 1/2 between 1/2,475 and 1/10,000 AEP or MCE. For passive closure the design earthquake considered would be the 1 in 10,000 AEP or MCE. Seismic hazard values for probabilities lower than 1:2,475 years are not available in the current NBCC. Extrapolation of the hazard model to lower probability, though possible (NBCC recommends log-log scale extrapolation), represents a degree of uncertainty and may be unreliable. However, at this stage of the project, extrapolation was deemed appropriate. The PGA for the 1:10,000 year AEP was estimated by extrapolation to be approximately 0.098 *g* for seismic “Site Class C”.

5.4 Tailings Pond Sizing

The tailings pond shall be sized to contain run-off and tailings water that accumulates in the pond over the winter months and the subsequent snowmelt. The TMF must also provide storage for the Environmental Design Flood (EDF) below the invert of the emergency spillway. The 100-year, 24-hour event (75 mm of rain) was selected as the EDF. The EDF is the most severe flood (i.e., largest design runoff event) that can be stored and does not result in an unscheduled discharge of water to the environment. The EDF is on top of the 25 year return period wet hydrological conditions.

This volume is determined using the water balance presented in Section 6.

5.5 TMF Emergency Spillway

The emergency spillway must be designed to allow for safe routing of the IDF to maintain a minimum freeboard and prevent dam overtopping. Under no circumstances should a dam be allowed to overtop. The invert of the emergency spillway is raised with each raise of the containment dams. A spillway will also be provided at closure.

5.6 Dam Stability

Table 6 presents the minimum factors of safety (FOS) for slope stability of the tailings dams to be adopted during design based on the CDA (2014) guidelines.

Table 6: Factors of Safety for Dam and Dyke Slope Stability (CDA, 2014):

Loading Condition	Minimum Factor of Safety
Short-term (immediately after construction)	1.3
Long-term steady state (once the facility is operating)	1.5
Rapid drawdown (upstream slope where applicable)	1.2 to 1.3
Pseudo-static	1.0
Post-earthquake	1.2

6.0 WATER BALANCE MODELLING

6.1 Modelling Philosophy

Tailings management is primarily a water management problem. The precipitation and process flows have to pass through a disposal facility in a near constant stream over the entire life of the mine. The challenge is to allow this to safely happen over a wide range of climatic and operating conditions in a facility that is continuously growing and expanding.

6.2 Water Balance

A deterministic water balance for the tailings facilities was developed on linked Microsoft Excel spreadsheets to simulate the operational conditions. The monthly accumulation of water in a tailings facility, for a range of climate conditions, is the basis for developing the water management plan for the facility. The model includes the following:

- Flows associated with processing the ore, including loss of water retained in the deposited tailings
- Flows associated with runoff from precipitation
- Evaporation from the pond surface
- Seepage losses - at this stage of design, it is assumed that all seepage is collected and pumped back to the facility (i.e. no net seepage loss)

The water balance assumes that the excess site water for tailings-related facilities is treated and discharged to Victoria Lake. Excess water from the open pit dewatering and run-off from waste rock stockpiles are managed separately and are not included in the TMF water balance.

Two different scenarios were considered, the full production with the tailings deposited at the TMF (years 5-9) and years 10 to 12 when tailings will be deposited in the Leprechaun Pit.

The scenarios were run under mean and 25-yr wet precipitation conditions. The water balance run for the TMF under mean annual precipitation (i.e., average year) is provided in Appendix C. The precipitation and evaporation data are entered on Sheet 4 and the collecting watershed areas (Figure 3) on Sheet 7. The operating data is presented on Sheet 8 and the flows associated with processing the ore are calculated on Sheet 9. Flows associated with runoff from precipitation are calculated on Sheets 10a and 10b. Evaporation and seepage losses are calculated on Sheet 11. Miscellaneous flows are on Sheet 12. The tailings water balance for the TMF is on Sheet 13a. All the flows are summarized on a monthly basis on Sheet 15. A flow diagram of the water balance for the site is shown in Appendix C.

6.3 Water Balance Modelling Results

A summary of the water balance results for the TMF at a production rate of 4 Mtpa are shown in Table 7. It is apparent that the site has excess water and the TMF can provide all the mill's make-up requirements. The water treatment plant will only be active during the non-winter months. The TMF pond, with a maximum storage capacity of 1 Mm^3, has been sized to store the excess site water during the non-discharge period. The Polishing Pond is located downstream of the TMF and WTP. The pond is designed to provide sufficient residence time for the settlement of solids. A retention time of 5 days was assumed based on input from Ausenco.

Table 7: Water Balance Summary for Tailings Deposited at TMF at 4 Mtpa – Ultimate Configuration

Flows		Annual Precipitation Conditions	
		Mean	25-yr Wet
		Annual Volume (Mm^3/year)	
Inflows	Tailings Water	2.15	2.15
	Surface Runoff	2.06	2.49
	Total Inflows	4.21	4.64
Outflows	Water Retained in Deposited Tailings	1.34	1.34
	Evaporation	0.32	0.32
	Reclaim to Mill	1.88	1.88
	Discharge to Water Treatment Plant	0.67	1.10
	Excess Water Stored	0	0
	Total Outflows	4.21	4.64
Mill Make-up Water Deficit (reclaim water required from an external source)		**0**	**0**

In year 10 of the mine life, tailings deposition switches from the TMA to the Leprechaun open pit. Leprechaun Pit has a smaller footprint and contributing catchment compared to the TMF. As a result, not all the mill's make-up water can be sourced from the pit. Additional make-up water is required, which can be sourced from the original TMF. The return pumping system at the TMF must therefore remain active once tailings deposition switches to the Leprechaun Pit. Note that this analysis excludes groundwater inflows to the Leprechaun Pit. The management of two tailings discharge areas, with the one being active and the other inactive, increases the total footprint and therefore increases the inflows and outflows of water within the system. As a result, the treatment and discharge rates increase once tailings deposition starts at the Leprechaun Pit. The source of the water for treatment and discharge is the TMF. A summary of the water balance results for year 10 onwards is shown in Table 8.

Table 8: Water Balance Summary for Tailings deposited at Leprechaun Pit at 4 Mtpa – Ultimate Configuration

Flows		Annual Precipitation Conditions			
		Mean		25-yr Wet	
		Annual Volume (Mm^3/year)			
Inflows	Tailings Water	2.15		2.15	
	Surface Runoff	Pit:	0.5	Pit:	0.61
		TMF:	2.06	TMF:	2.49
	Total Inflows	4.71		5.25	
Outflows	Water Retained in Deposited Tailings	1.34		1.34	
	Evaporation	Pit:	0.08	Pit:	0.08
		TMF:	0.32	TMF:	0.32
	Reclaim to Mill	Pit:	1.24	Pit:	1.34
		TMF:	0.64	TMF:	0.54
	Discharge to Water Treatment Plant	TMF: 1.10		TMF: 1.63	
	Excess Water Stored	0		0	
	Total Outflows	4.71		5.25	
Mill Make-up Water Deficit (reclaim water required from an external source)		**0**		**0**	

6.3.1 Tailings Pond Sizing

The water balance results for the scenario where the TMF is the active tailings disposal location, indicate that the TMF can meet the full make-up water requirements of the mill. As the water treatment plant will only be active during the non-winter months, the TMF pond needs to be sized to store water accumulated during the non-discharge period. Under average climate conditions, a peak capacity of 0.84 Mm^3 is required. Under the 25-year wet conditions, this requirement is raised to 0.91 Mm^3. To store the run-off from the EDF an additional 0.2 Mm^3 is required. It is therefore recommended that the TMF should allow for an operational pond with a capacity of up to 1.1 Mm^3 below the spillway invert.

7.0 TAILINGS STORAGE FACILITY DESIGN

7.1 Tailings Deposition Plan

7.1.1 Design Assumptions

The following are the design assumptions used in the development of the tailings deposition plan:

- Tailings void ratio of 0.9 (volume of voids / volume of solids)
- Based on the assumed void ration the in-situ dry density of deposited tailings can be calculated as 1.41 t/m^3
- A deposited tailings slope of 3% above the tailings pond and 10% below water
- Tailings will be deposited via spigots primarily from the natural high ground on the north west side of the TMF and secondly from the perimeter embankment
- Tailings deposition will be directed such that the the tailings pond is located on the east side of the TMF.

7.1.2 Staged Construction and Deposition

The TMF is a horseshoe shaped side-hill facility. The design takes into account the beach slope as a result of thickened tailings deposition. The tailings deposition is primarily from the existing high ground on the north west side of the site and as a result the perimeter dam is higher on the west side of the facility and slopes down to the east. The TMF will be developed over five stages, Figure 5 provides a layout of the staged TMF development. Table 9 provides details on the TMF staged construction and capacity. The deposition stages are described below.

Start-up – Stage 1A: The initial TMF construction provides storage for 3.1 Mm^3 (4.4 Mt) tailings which is sufficient for the first 2 years of mine production. The maximum tailings discharge elevation at end of this stage is 389.7 masl. Tailings deposition will start in the TMF from the high ground in the north west. A pond will form at the toe of deposited tailings. The pond will always be pushed to the east end of the TMF, with water being pumped back to the mill on an as needed basis. The capacity of the tailings pond for this stage was considered to contain maximum operating water volume of 1.1 Mm^3. The maximum operating water level at start-up is el. 371.0 masl.

Year 2 – Stage 1B: Tailings deposition will continue in the TMF from the high ground in the north west and the south perimeter dam. The TMF embankments will be raised by 2 to 3 m in the downstream direction and this raise can accommodate an additional 1.8 Mm^3 (2.5 Mt) of tailings for another year of storage, 4.9 Mm^3 total (6.9 Mt). The maximum tailings discharge elevation at end of this stage is 393 masl. The capacity of the tailings pond for this stage was considered to contain maximum operating water volume of 1.1 Mm^3. The maximum operating water level after the dam raise in Year 2 is el. 373.0 masl.

Year 3 – Stage 2: The TMF embankments will be raised by 6 to 7 m in the downstream direction and this raise can accommodate an additional 5.4 Mm^3 (7.6 Mt) of tailings for another two years of storage, 10.3 Mm^3 total (14.5 Mt). The maximum tailings discharge elevation at end of this stage is 399 masl. The capacity of the tailings pond for this stage was considered to contain maximum operating water volume of 1.1 Mm^3. The maximum operating water level after the dam raise in Year 3 is el. 380.0 masl.

Year 5 – Stage 3: The TMF embankments will be raised by 6 to 7 m in the downstream direction and this raise can accommodate an additional 5.45 Mm^3 (8.7 Mt) of tailings for another two years of storage, 15.7 Mm^3 total (22.2 Mt). The maximum tailings discharge elevation at end of this stage is 410 masl. The capacity of the tailings pond for this stage was considered to contain maximum operating water volume of 1.1 Mm^3. The maximum operating water level after the dam raise in Year 5 is el. 384.0 masl.

Year 7 – Stage 4: The TMF embankments will be raised by 6 to 7 m in the downstream direction and this raise can accommodate an additional 6.15 Mm^3 (7.6 Mt) of tailings for another two years of storage, 21.9 Mm^3 total (30.9 Mt). The final dam raise will have a 10 m wide crest. The maximum tailings discharge elevation at end of this stage is 416 masl. The capacity of the tailings pond for this stage was considered to contain maximum operating water volume of 1.1 Mm^3. The maximum operating water level after the dam raise in Year 7 is el. 388.5 masl.

Year 9 - Ultimate: Tailings deposition will continue in the TMF after the final dam raising in Year 7. For the remaining two years of the TMF life, the tailings will be deposited from the high ground in the north west and from the perimeter dams on the west, south and east sides while the tailings pond is pushed towards the east.

Table 9: Staged Tailings Facility Requirements with Dam Construction

Description	Unit	Stage 1A	Stage 1B	Stage 2	Stage 3	Stage 4
Year of Construction	Year	0	2	3	5	7
Year of Operation	Year	1-2	3	4-5	6-7	8-9
TMF Tailings Storage Capacity (per stage)	Mm^3	3.1	1.8	5.4	5.5	6.2
TMF Tailings Storage Capacity (cumulative)	Mm^3	3.1	4.9	10.3	15.8	22.0
Tailings Pond Volume	Mm^3	0.9	1.0	1.0	1.0	1.0
Dam Crest Elevation	masl	385.7-373.0	388.8-375.0	394.9-382.0	402.3-386.0	408.3-390.5
Dam Crest Length	m	2,490	2,675	2,900	3,125	3,325
Maximum Dam Height Above Natural Ground	m	20	26.5	36.5	43	49

Year 10 -12: Tailings deposition at the Leprechaun Pit. Tailings deposition will be open ended from different locations around the perimeter. This will allow the tailings pond to be located on one side for decant and return of excess water to the plant.

7.2 Water Management Plan

The TMF pond will collect direct precipitation, runoff from the tailings surface, water discharged from the mill with the tailings, and water pumped back from the seepage collection sumps around the facility. During the mine's operational phase, water will be pumped from the TMF pond via a reclaim pump system for the operation of the processing plant. Water recycled from the tailings pond is approximately 155,000 m^3/month once full production of 4 Mtpa is achieved in Year 5. For the annual reclaim water volume of 1.88 Mm^3 to the mill, the recommended pumping rate is 220 m^3/hr. Once tailings deposition commences in the Leprechaun Pit, the reclaim water is sourced from both the pit and the TMF. The total annual reclaim rate remains unchanged. The recommended pumping rate from the Leprechaun Pit is 140 m^3/hr with the balance sourced from the TMF. The water balance assumes a minimum of 10% of clean make-up water is required in the process plant. This is from an external source (e.g., Victoria Lake).

Seepage collection ditches will be constructed at the downstream toe of the TMF dam. Seepage from the ditches will be directed to sumps at various topographic low points around the dams; seepage and runoff collected in the sumps will be pumped back to the TMF.

Under mean annual precipitation conditions, annual inflows to the TMF exceed the annual mill requirements. Excess water must be pumped from the TMF, treated at the Waste Water Treatment Plant, discharged to the Polishing Pond and then to the environment. The annual discharge volume under mean precipitation conditions is 0.67 Mm^3 which increases to 1.1 Mm^3 under the 25-year wet conditions. Treatment and discharge only occur for 8 months a year. The discharge rate therefore varies from 116 m^3/hr to 190 m^3/hr for mean and 25-year wet conditions. Once tailings deposition commences in the Leprechaun Pit, the tailings footprint increases, and the annual discharge volumes increase to 190 m^3/hr for mean conditions and 280 m^3/hr for the 25-year wet conditions.

Although the water balance indicates that the TMF can supply all the mill reclaim water requirements, during the winter, water in the tailings pond will freeze and the slurry water may freeze in the tailings before getting to the pond. An alternate source should therefore be provided for the monthly mill water requirements if reclaim water from the TMF is not available.

7.3 Dam Design

The overall design objective of the TMF is to protect the regional groundwater and surface water resources during both operations and long term (post-closure), and to achieve effective reclamation upon mine closure.

The primary construction material for the TMF is the waste rock from the open pits. The rockfill shall consist of a well graded blasted rockfill. The first four stages will be constructed with a crest width of 20 m to facilitate the use of mine haulage equipment in the dam construction. The final stage will have a crest width of 10 m and may require smaller earthmoving equipment for the final few metres of the dam raise. The rockfill shall be dumped by the mine fleet and then spread, moisture conditioned and compacted, by a civil contractor.

All the stages have an intermediate upstream slope of 3 Horizontal (H): 1Vertical (V). With the provision of benches between stages on the upstream side, the average slope flattens to about 3.5H: 1V. The forming of the upstream slopes to the required grades will be completed by a civil contractor. Excess material generated by the formation of the upstream slopes will be dumped in the downstream footprint or used for ramp construction. The downstream slope is 2H: 1V for all stages. On the upstream slope, a 1 m thick (measured perpendicular to the slope) coarse filter/ transition layer will be placed on the prepared waste rock slope followed by a 1 m thick fine filter layer. A 1.5 mm thick linear low density polyethylene (LLDPE) geomembrane will be installed, as the main water retaining element, on the fine filter layer. A 0.3 m thick layer of road surfacing will be placed and compacted along the dam crest to allow for light vehicle traffic during operations

The coarse filter material shall be gravel sized and act as a transition between the rockfill and fine filter. The material should be a processed, well graded, non-frost susceptible and free draining. The coarse filter will be placed in layers and compacted. It is expected that the coarse filter material will be obtained by crushing non-acid generating rockfill. The fine filter material shall sand sized and filter compatible with the tailings and the coarse filter. The material should be a processed, well graded, non-frost susceptible, free draining material with a fines content of less than 5%. The fine filter should be placed in layers and lightly compacted. It is expected that the fine filter material will be obtained by screening granular materials found in local borrow areas.

The embankment materials shall be non-acid generating and non-metal leaching under neutral pH conditions.

7.4 Frost Protection Considerations

None of the materials used for the construction of the tailings dam is susceptible to frost or subject to the effects of freeze – thaw cycles. The depth of frost penetration at the TMF is expected to be 1 to 1.5 m. The actual depth of frost penetration will depend on a number of factors including actual temperatures and depths of snow cover.

7.5 Slope Stability Assessment

Two-dimensional limit equilibrium slope stability analyses were performed using the commercially available program SLOPE/W, developed by GEOSLOPE International Ltd., employing the Morgenstern Price method of analysis. Slope stability was analysed for a representative critical section of the dam, and for the Stage 1A (starter) and Stage 4 (ultimate) dam elevations. The details of the slope stability analyses are provided in Appendix D.

The shear strength of the dam foundation and fill materials were estimated from the limited investigation data and our experience with similar soils. All materials were modelled using effective stress parameters assuming drained conditions only given the granular, free draining behaviour of the soils.

End of Construction and long-term loading conditions were considered in the analysis. Both static and pseudo-static conditions were analyzed. It was assumed that native overburden and dam fill materials are not susceptible to liquefaction at any stages of the TMF dam's life.

The calculated factors of safety for end of construction, long-term and pseudo-static analyses satisfy the minimum target factor of safety requirements of 1.3, 1.5 and 1.0, respectively. Given the uncertainty of the foundation conditions at this time, a sensitivity analysis was carried out whereby the effective stress strength parameters of the till foundation soil were varied. The results indicate that a minimum drained strength of about 31° to 32° would be required. A geotechnical investigation at the TMF is recommended at the next stage of design to validate the stability analyses.

7.6 Seepage Assessment

The potential leakage rate through the proposed geomembrane liner is limited by the size and frequency of defects in the geomembrane liner that are not detected by the Construction Quality Assurance (CQA) inspection/testing program. These defects represent the primary pathway through which leakage through the proposed geomembrane may occur.

The potential leakage rate per defect in the geomembrane liner was calculated for two different areas of the TMF:

- The side slope area where geomembrane is in contact with water (ponded area).
- The side slope area where geomembrane is in contact with tailings (tailings area).

The leakage rate per defect in the geomembrane was estimated for the average head acting at the mid-slope point of the liner using the methods presented by Giroud et. al. (1997) or Badu-Tweneboah and Giroud, J.P. (1997) for the geomembrane in contact with water or tailings, respectively. The estimated leakage rates for the Stage 4 – ultimate configuration are summarized in Table 10 below. The details of the leakage rate estimates are provided in Appendix E.

Table 10: Estimated Leakage Rates through Geomembrane Liner

Area	Liner Surface Area (m^2)	Average Head on the Mid-slope of Liner (m)	Leakage Rate for 5 defects (m^3/year/ha)	Total Leakage Through the Liner (m^3/day)
Pond Area	20,400	3.5	2,450	13.7
Tailings Area	265,350	15.6	3.4	0.3
			TOTAL	**14.0**

Foundation seepage has not been estimated at this time as it is assumed that the liner will be anchored to treated, low permeable bedrock with the requirements for bedrock grouting seepage cut-off measures not known at this time. A geotechnical investigation with hydraulic conductivity testing is recommended to supplement the next phase of design.

7.7 Flow Conveyance Structures

7.7.1 Seepage and Run-off Collection Ditches

The seepage and run-off collection ditches are located at the downstream toe of the TMF dams to collect run-off from the downstream dam slope and shallow seepage flow. The ditches direct flow to two collection sumps. The ditches are sized to collect the runoff for up to the 25-year rainfall storm event (Golder, 2020). The typical cross-section for the seepage collection ditches is trapezoidal with a base width of 2.0 m, minimum depth of 1.0 m, and 2H:1V side slopes. The ditches are lined with riprap.

7.7.2 Emergency Spillway

The emergency spillway is required to prevent dam overtopping under extreme climate conditions. Based on the dam classification, the inflow design flood is 2/3 between the 1 in 1,000 year and the Probable Maximum Flood (see Table **4**). Under normal conditions, the TMF is sized to collect runoff, and tailings water. Under the IDF scenario, it is assumed that the mill will remain operational and tailings slurry and reclaim water pumping will continue. A preliminary analysis of the spillway sizing indicates that for a spillway width of 20 m, the maximum flow depth under the PMP is 0.8 m through the spillway.

7.8 Polishing Pond

The Polishing Pond is located downstream of the TMF and has a footprint area of 4.1 hectares. The pond will be constructed as part of the initial TMF with an operational capacity of 44,000 m^3. The pond will be lined with a geomembrane, similar to the upstream slope of the TMF embankment. The pond is designed to provide sufficient residence time for the settlement of solids. A retention time of 5 days was assumed based on a nominal flow through rate of 115 to 280 m^3/hr, which is sufficient to treat runoff, precipitation and process flows for up to a 25-year wet precipitation year. To promote settling, the pond is designed with a minimum length to width ratio of 4:1. The design also allows for up to 0.5 m of solids accumulation and has a minimum freeboard of 2 m above the maximum operating level with a spillway design to pass its IDF.

8.0 CONSTRUCTION QUANTITIES AND COST ESTIMATES

Construction quantities were estimated for the development of the TMF over the life of mine. Ausenco is responsible for preparing the ultimate cost estimate for the pre-feasibility study (both CAPEX and OPEX) and has provided the bulk of the project specific unit rates. The initial and sustaining capital cost estimates are included in Appendix F. The quantities were calculated using a combination of Microsoft Excel and AutoCAD, with the base mapping for the site.

The costs estimates exclude the following:

- Tailings discharge pipelines and pumping systems;
- Tailings disposal operating equipment;
- Water recirculation pumps and pipelines from the process plant or tailings / collection ponds;
- Quantities associated with post-closure monitoring, maintenance, and treatment requirements; and
- Access and haul roads.

9.0 PERFORMANCE MONITORING

During operations, the implementation of a systematic performance monitoring program is critical to maintaining the physical integrity of the dams and ancillary structures at the TMF. Such a program should include environmental monitoring together with regular visual inspections of the entire facility and monitoring of piezometric levels within the containment dams. The program will be documented in an Operation, Maintenance, and Surveillance Manual that shall be developed during the detailed design and construction stage of the Project and it will be re-visited on a regular basis to account for any changes in the performance or operation of the TMF.

9.1 Inspection Programs

The various components within the TMF should be regularly inspected by knowledgeable personnel, familiar with their design and operating requirements. The results of the monitoring program should form the basis for determining maintenance and remediation measures that may be required from time to time. The design and as-built construction records for the various components of the TMF are vital to assessing the performance of the facility and should be properly archived at the site and accessible for review when required.

It is common practice to implement such programs on three levels; routine observations (daily), detailed inspections (quarterly and annually) and formal dam safety reviews (every five to seven years depending on the dam hazard classification). In addition, detailed inspections should be conducted following spring runoff and after any unusual events such as heavy rainstorms, windstorms and seismic events. Water levels in the TMF Pond should be recorded on a daily basis and a detailed survey of the tailings surface shall be completed annually to assist with tailings deposition planning and construction scheduling.

9.2 Dam Instrumentation

The performance of the containment dams can be inferred, in part, from instrumentation placed within the dams and their foundations. Since the dams are of relatively modest height and it is anticipated that they will be constructed on competent foundation soils/bedrock, the installation of instrumentation during the construction program is not considered necessary. A dam instrumentation plan based on conditions encountered, and observations made, during construction shall be developed and implemented as soon after construction as possible. Factors that should be considered in the development of the instrumentation plan include conditions and difficulties encountered during construction, and the results of construction monitoring of fill materials and fill placement.

At a minimum, it is anticipated that instrumentation will be required to monitor piezometric levels of underlying foundation soils at each of the dams. Settlement plates, installed to a depth that penetrates the maximum expected frost depth, shall be installed on the crest of dams along with the piezometers to monitor settlement. Inclinometers can also be installed to monitor lateral deformations within the dam.

Initially, water level measurements in the piezometers should be taken at bi-weekly intervals. Once stabilized, the monitoring frequency will likely be reduced to monthly readings, provided that abnormal conditions have not been observed during the initial monitoring period. Additional instrumentation and/or more frequent monitoring may be required if unusual conditions are indicated by the initial monitoring or by observations recorded during facility inspections.

9.3 Groundwater Monitoring

Some seepage through and under the dams at the TMF can be anticipated. It is expected that the majority of the seepage from the dams can be collected in ditches and conveyed to small sumps and, if necessary, pumped back into the TMF. The remainder would be lost to the groundwater flow regime.

A network of groundwater monitoring wells shall be installed downstream of each of the dams during the initial construction program. It is recommended that an additional monitoring well be installed outside the TMF to serve as an indicator of background groundwater quality. The location and number of additional monitoring wells to be installed in the future shall be determined based on the performance and results of the initial monitoring wells.

Groundwater is at or close to ground surface over most of the site and a layer of glacial till is present over the bedrock. Apart from flow along discrete structural features in the bedrock (e.g. faults, jointing, etc.), it is anticipated that groundwater flow is likely to occur within the overburden, the bedrock/till contact, and within the upper slightly weathered bedrock zone. It is therefore suggested that the monitoring wells be installed to monitor these potential flow pathways. Prior to well installation, the locations and design of the monitoring wells should be reviewed on the basis of information obtained during construction. As necessary, the detected seepage can be directed back into the TMF via pump-back wells.

In addition to groundwater monitoring, it is also suggested that surface water quality monitoring be done in the creeks downstream and around the TMF.

10.0 TMF CLOSURE

The major closure and reclamation activities planned for the TMF are expected to occur during the first two years of closure. The TMF dam has been designed for long-term stability. Thus, no additional re-grading of the side slopes will be required at closure. A cover will be placed over the tailings surface at closure. Progressive reclamation of the tailings surface is not anticipated at this time based on the current tailings deposition plan. The main objective of the closure cover will be to limit the migration of contaminants, and it will also prevent wind and runoff erosion of the tailings. The proposed closure cover will be 0.3 m thick and it will consist of overburden and topsoil mixture. The top area of the cover will be hydroseeded. The tailings surface will be contoured as necessary to promote drainage towards the TMF pond. It is expected that over time, wet tailings will undergo consolidation and desiccation that will improve material strength and trafficability of construction equipment. The cover design will be finalized during the detailed design phase of the TMF. Placement of cover materials may need to occur during the winter in the softer areas if sufficient frost penetration exists to support construction traffic. The downstream slopes of the TMF dam will be left as exposed rockfill to permit drainage of the downstream shell.

At closure, the tailings pond volume will be reduced to minimize the risk associated with a large pond. Water will be treated before release, as per operation stage practices, until water quality monitoring demonstrates that water collected in the pond is acceptable for direct release to the environment. At that time, the decant pump system will be decommissioned. The spillway will be a permanent structure after closure and will be upgraded to meet the closure requirements.

The seepage water collection system, including the pumps will be kept in service until water quality monitoring demonstrates that water collected in the system is acceptable for direct release to the environment. At that time, the pumping systems will be removed and the sumps will be backfilled. The backfilled sump areas will be re-graded to restore original drainage courses to the extent possible and to enhance the area for natural re-vegetation.

When no longer required, the seepage collection ditches will be re-contoured to restore the original drainage course to the extent possible and to enhance the area for natural re-vegetation.

Closure and post-closure water quality predictions will be required during the detailed design phase of the TMF.

Monitoring and maintenance of the reclaimed facilities will be carried out during operations and into closure. It is anticipated that monitoring and maintenance will be carried out during the active closure stage at frequencies similar to those required during operations. Post-closure monitoring and maintenance will be carried out at a reduced frequency depending on the results of the monitoring and the measures of success selected for closure.

The proposed closure monitoring and maintenance activities include visual inspections of reclaimed areas to identify unstable areas, maintain all facilities and equipment to be used during closure until they are no longer required, install instrumentation at selected facilities for monitoring of the reclaimed areas, and test surface and groundwater quality and measure water volumes at select locations to confirm that the closure measurements are performing as predicted and are not adversely affecting the environment as required by the Newfoundland and Labrador Mine Regulation 42/00.

A geotechnical inspection by a qualified engineer should be performed on an annual basis to look for signs of any instability (settlement, cracking, erosion, etc) to ensure that the dams and covers are performing as intended and review maintenance carried out over the previous year.

The proposed closure and post-closure monitoring will determine the long-term maintenance that would be required for the post-closure period. The TMF closure and post-closure monitoring requirements should be prepared in conjunction with the overall Project monitoring requirements.

11.0 PATH FORWARD

The following activities are recommended to support the design of the TMF as it advances to the feasibility level study:

- Geotechnical site investigations at the preferred TMF site to characterize the foundation conditions associated with the proposed infrastructure.
- Geotechnical investigations within the property boundary to identify potential borrow sources and requirements for development of the borrow areas.
- In-situ permeability tests of the overburden soils and bedrock beneath the proposed dam foundations. The results of the investigation shall be used to evaluate the proposed dam design and seepage cut-off requirements (i.e. bedrock grouting).
- A site-specific seismic hazard assessment to inform the input parameters for dam stability assessment.
- Develop a groundwater model to evaluate the impacts of the TMF on the local environment. The model should also address the impacts of in-pit disposal of tailings in the mined-out Leprechaun pit during the latter years of operation.
- Tailings testing to determine the geotechnical properties to understand the settlement, permeability and deposition characteristics.
- Optimization and further evaluation of the proposed dam alignment, deposition planning, and construction staging based on the findings of the geotechnical site investigations and other project developments.
- A dam breach and inundation study to support the dam classification.
- Further refinement of the TMF and site wide water balance.
- Optimize the location and design of the Polishing Pond.
- Advancement of the closure cover design criteria and success attributes to optimize the reclamation requirements.
- Condemnation drilling for the TMF site to verify the absence of mineralization.

12.0 CLOSURE

We trust that this report meets your project requirements at this time. If you have any questions or require further information, please do not hesitate to contact the undersigned.

Golder Associates Ltd.

REGISTERED PROFESSIONAL ENGINEER
PEG Newfoundland and Labrador
W.P.H. MERRY
SIGNATURE
June 30, 2020
DATE
NEWFOUNDLAND & LABRADOR

Philip Addis
Project Manager

Peter Merry, P.Eng. (NL)
Principal

PA/PM/hp

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https://golderassociates.sharepoint.com/sites/117078/project files/6 deliverables/4. pfs design report/19130660 valentine gold pfs for tsf june2020 - final.docx

PROVINCE OF NEWFOUNDLAND AND LABRADOR
PEG Newfoundland and Labrador
PERMIT HOLDER
This Permit Allows
GOLDER ASSOCIATES LTD.
To practice Professional Engineering in Newfoundland and Labrador.
Permit No. as issued by PEGNL D0029
which is valid for the year 2020

REFERENCES

GEMTEC, 2019. Preliminary Summary, Baseline Hydrogeological Characterization Study, Valentine Lake Project, Central Newfoundland. Draft memo submitted to Marathon Gold Corporation December 4, 2019.

National Building Code of Canada, 2015, Seismic Hazard Calculator.

Lycopodium Minerals Canada Ltd, 2018, "Technical Report, Preliminary Economic Assessment of the Valentine Lake Gold Project, Newfoundland, NL, Canada", dated October 30, 2018, consultant's reference: File Location: 17.04.

Marathon Gold (2020), e-mail correspondence from James Powell dated 25 February 2020.

Piercey, S.J., Squires, G.C. and Brace, T.D., 2014. Lithostratigraphic, Hydrothermal, and Tectonic Setting of the Boundary Volcanogenic Massive Sulfide Deposit, Newfoundland Appalachians, Canada: Formation by Subseafloor Replacement in a Cambrian Rifted Arc. Economic Geology, v. 109, p661-687.

Smith, J.S., 2011. Surficial geology of the Victoria Lake map area (NTS 12A/06). Geological Survey, Department of Natural Resources, Government of Newfoundland and Labrador, Map 2011-03, Open File 12A/06/1509.

Stantec Consulting Ltd., 2012a, "2011 Forest Songbird Surveys at the Valentine Lake Prospect – Draft Report", Dated January 6, 2012, Consultant's reference File No. 121510621.

Stantec Consulting Ltd., 2012b, "Marathon Baseline Fish and Fish Habitat Data Report – Draft Report", Dated January 10, 2012, Consultant's Reference File No. 121510621.300.

Stantec Consulting Ltd., 2012c, "2011Baseline Waterfowl and Waterfowl Habitat Study Valentine Lake Project – Draft Report", Dated January 25, 2012, Consultant's Reference File No. 121510621.300.

Stantec Consulting Ltd., 2017a, Valentine Lake Project: Baseline Hydrology and Surface Water Quality Monitoring Program", Dated December 1, 2017, Consultant's Reference File No. 121414740.

Stantec Consulting Ltd., 2017b, "Valentine Lake Project: Preliminary Baseline Hydrogeology Assessment", Dated December 15, 2017, Consultant's Reference File No. 121414740.170.

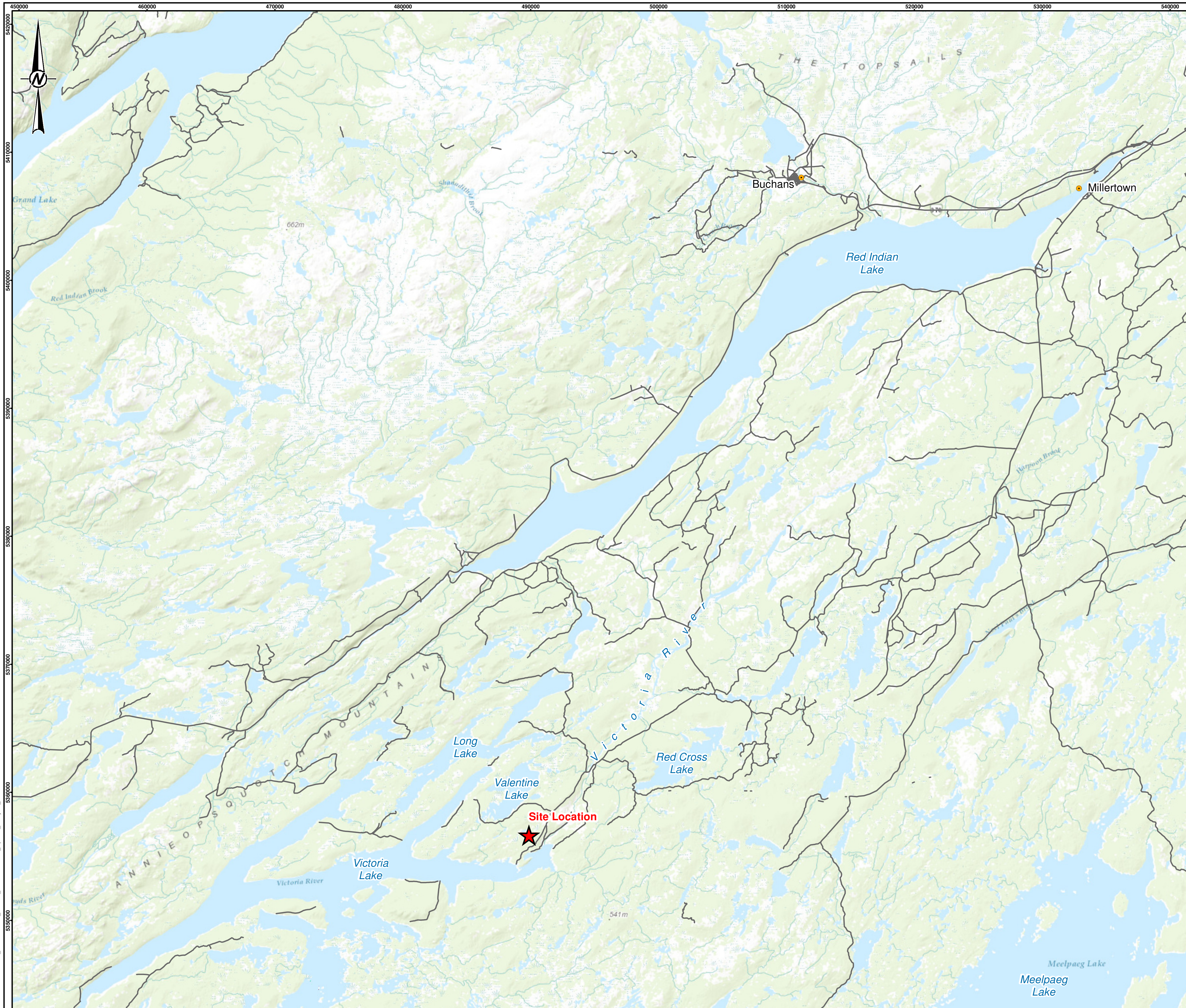
Stantec Consulting Ltd., 2018a, "Marathon Valentine Lake Gold Project – Preliminary Economic Assessment Operational Mine Site Water Balance", Dated March 19, 2018, Consultant's Reference Document No: MEM-001-400-A-19Mar18.

Stantec Consulting Ltd., 2018b, "Preliminary Results of Phase I ARD/ML Assessment – Draft Report", Dated March 7, 2018, Consultant's Reference File No. 121414740.180.

Stantec Consulting Ltd., 2018c, "PEA Level Tailings Facility Site Selection Study", Dated January 25, 2018, Consultant's Reference File No. 121621464. Document No. MEM-001-300-A-25Jan18.

Stantec Consulting Ltd., 2019, "Valentine Lake Project: 2018 Fish and Fish Habitat Data Report", Dated February 1, 2019, Consultant's Reference File No. 121415923.

FIGURES

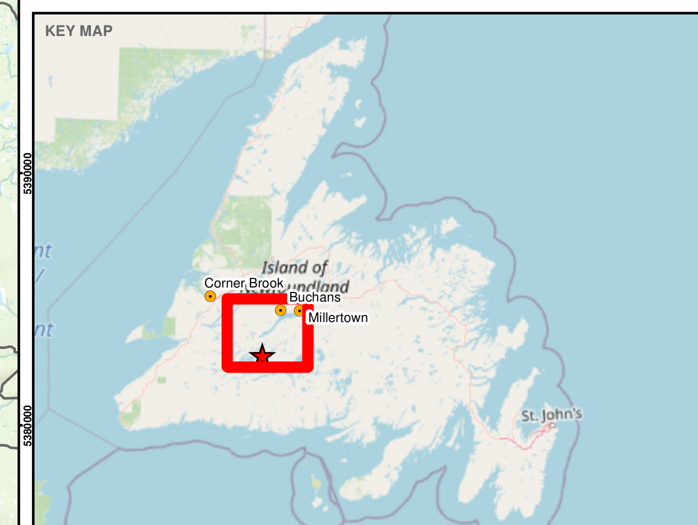


LEGEND

Site Location

Roads

Waterbody



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NOTE(S)

THIS FIGURE IS TO BE READ IN CONJUNCTION WITH THE ACCOMPANYING GOLDER ASSOCIATES LTD. REPORT NO. 19130660

REFERENCE(S)

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CLIENT

MARATHON GOLD

PROJECT

VALENTINE GOLD PROJECT

TITLE

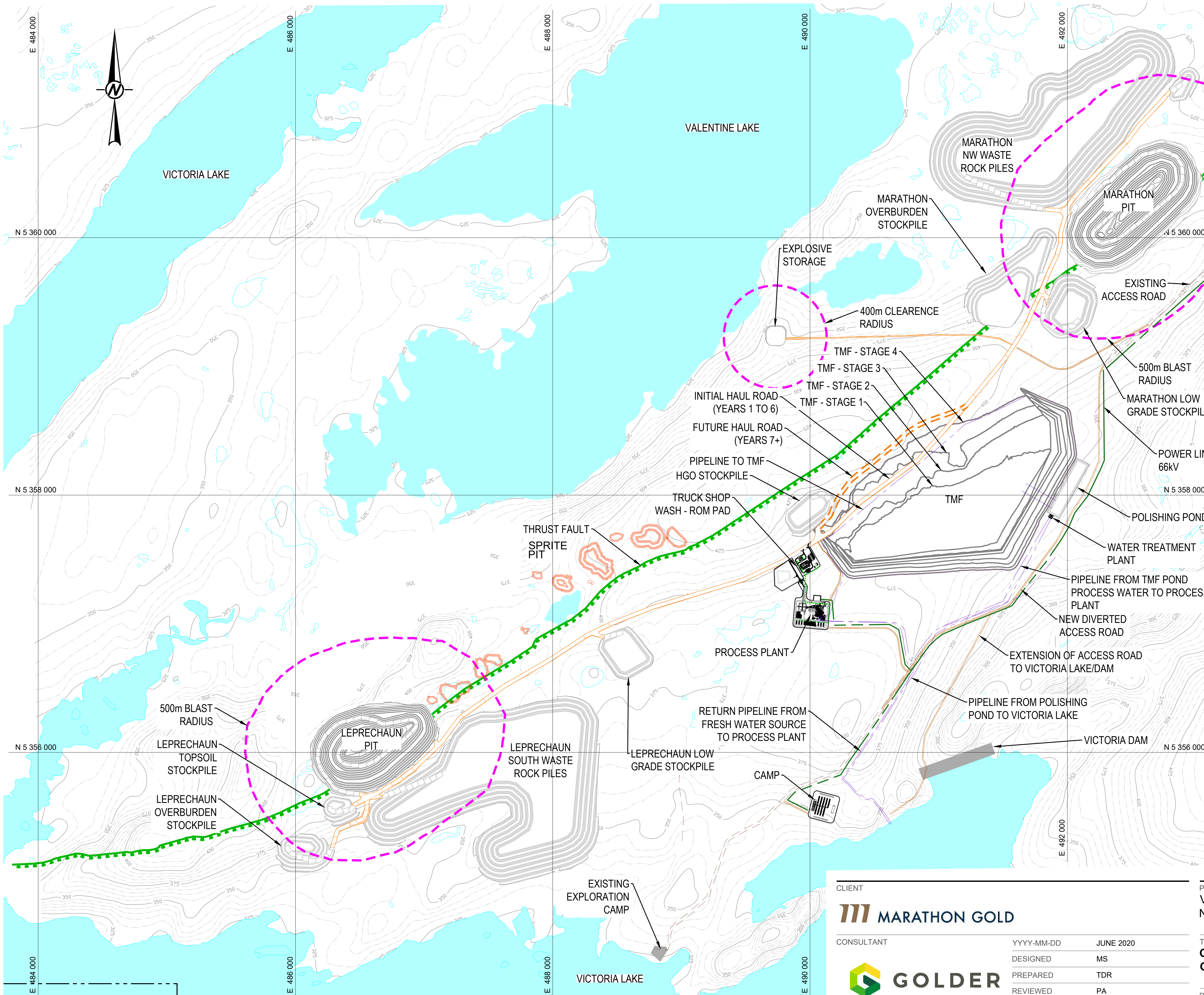
SITE LOCATION PLAN

CONSULTANT

GOLDER

YYYY-MM-DD	JUNE 2020
DESIGNED	MS
PREPARED	RRD
REVIEWED	PA
APPROVED	WPM

PROJECT NO.	CONTROL	REV.	FIGURE
19130660	0004	B	1



LEGEND

PROPERTY BOUNDARY

THRUST FAULT

NOTE(S):
1. PROJECTION: NAD83 UTM ZONE 21.

REFERENCE:
BASEPLAN PROVIDED BY AUSENCO. FILENAME 104319-0000-G-001_20200130.DWG. REVISION C, DATED 20/01-29

0 500 1,000
1:30,000 METRES

CLIENT
MARATHON GOLD

CONSULTANT
GOLDER

YYYY-MM-DD	JUNE 2020
DESIGNED	MS
PREPARED	TDR
REVIEWED	PA
APPROVED	WPM

PROJECT
VALENTINE GOLD PROJECT
NEWFOUNDLAND

TITLE
GENERAL ARRANGEMENT
OVERALL SITE PLAN

PROJECT NO.	CONTROL	REV.	FIGURE
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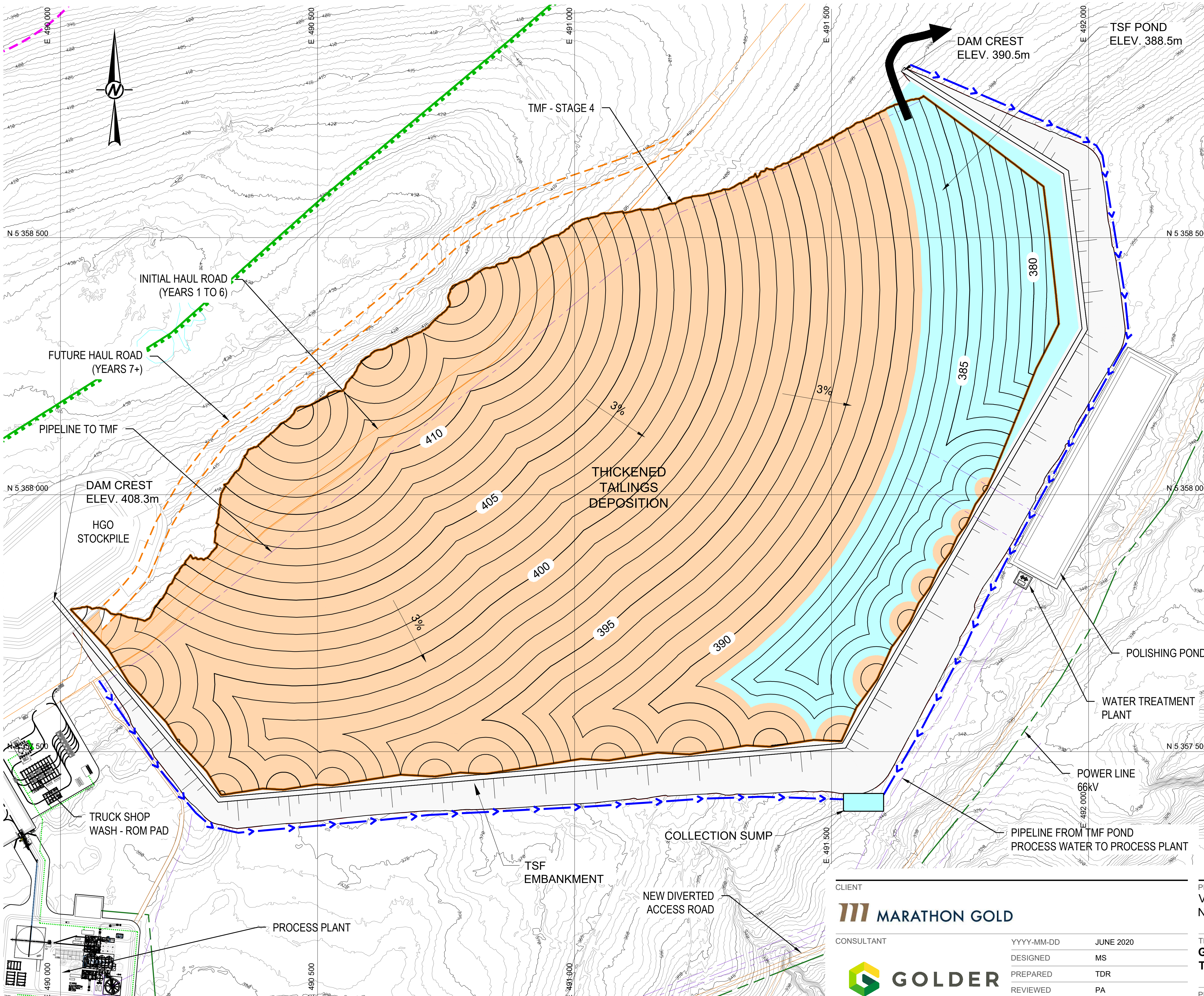

LEGEND

- THRUST FAULT
- SEEPAGE AND RUN-OFF COLLECTION DITCH
- TMF EMERGENCY SPILLWAY
- WATERCOURSE

NOTES:

1. ALL ELEVATIONS, GRID COORDINATES AND DIMENSIONS ARE IN METRES. GRID D COORDINATES ARE REFERENCE TO NAD83 UTM ZONE 21 DATUM.
2. GROUND CONTOURS INTERVALS AT 1m.

REFERENCES

BASEPLAN PROVIDED BY CLIENT. FILENAME 104319-0000-G-001_20200122.DWG. REVISION B, DATED 20/01-10 .

GROUND CONTOURS PROVIDED BY CLIENT. FILENAME MRG_01_Valentine_Mine_Survey_Data_Delivery.ZIP, DATED 11/6/2019.

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1:7,500 METRES

CLIENT
MARATHON GOLD

CONSULTANT
GOLDER

YYYY-MM-DD	JUNE 2020
DESIGNED	MS
PREPARED	TDR
REVIEWED	PA
APPROVED	WPM

PROJECT
VALENTINE GOLD PROJECT
NEWFOUNDLAND

TITLE
GENERAL ARRANGEMENT
TMF ULTIMATE CONFIGURATION

PROJECT NO.	CONTROL	REV.	FIGURE
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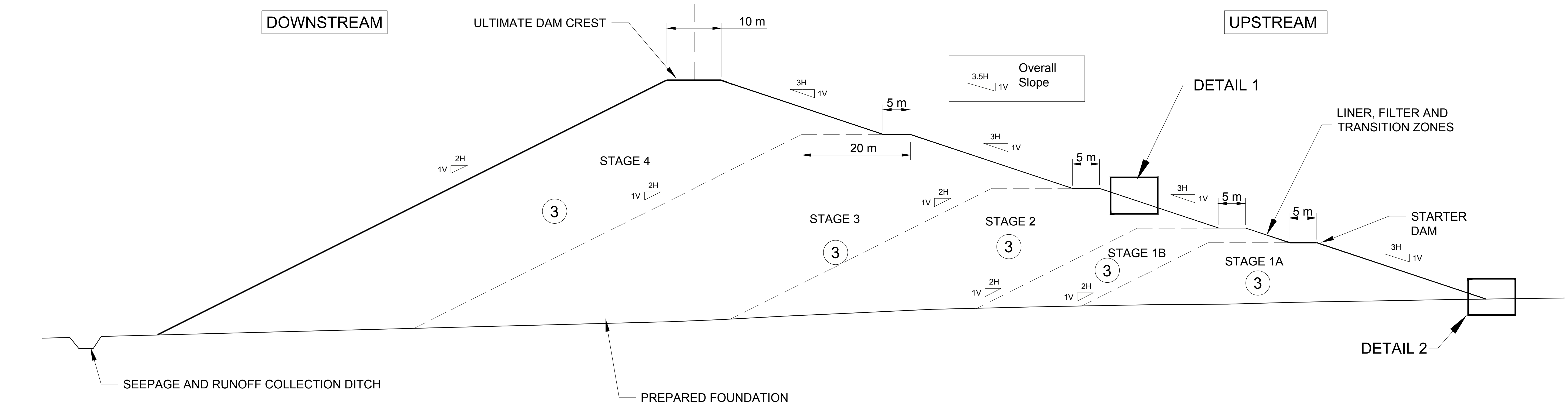

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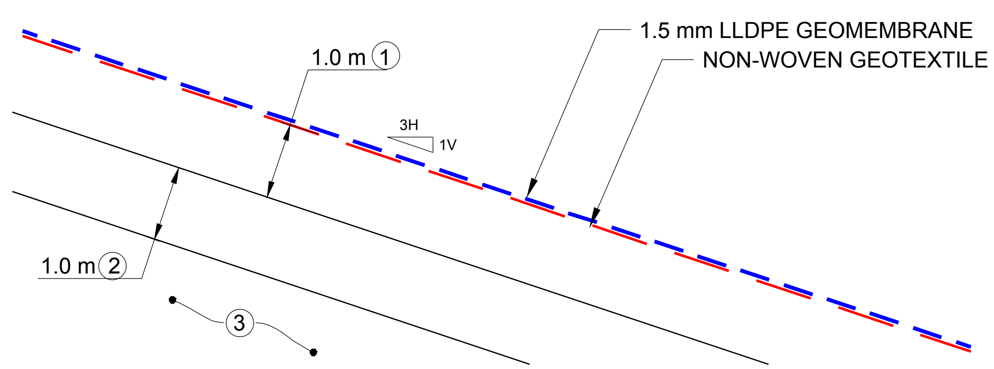

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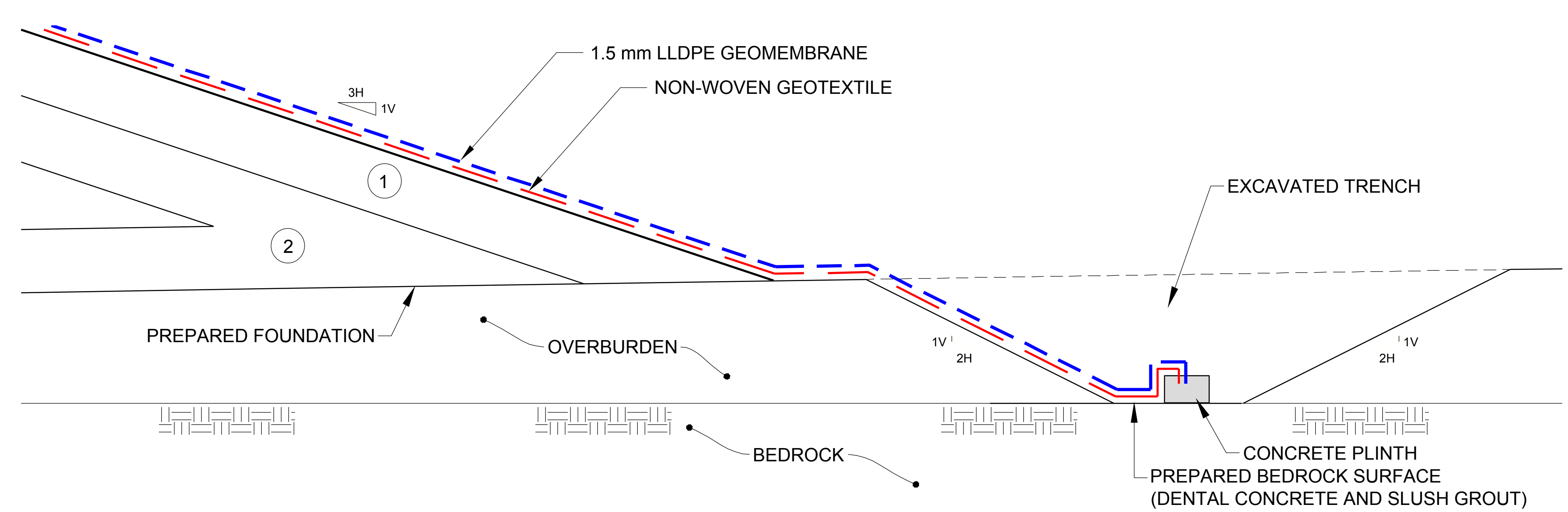

SCALE 1:100 2 - DETAIL 2: GEOMEMBRANE ANCHOR DETAIL

FILL MATERIALS		
ZONE	1	SAND BEDDING / FILTER SAND
ZONE	2	COARSE FILTER
ZONE	3	ROCKFILL

NOT FOR CONSTRUCTION

DRAFT

CLIENT
MARATHON GOLD

CONSULTANT
GOLDER

YYYY-MM-DD	2020-02-28
DESIGNED	MS
PREPARED	TDR
REVIEWED	PA
APPROVED	WPM

PROJECT
VALENTINE GOLD PROJECT
NEWFOUNDLAND

TITLE
TYPICAL DAM SECTION

PROJECT NO.	CONTROL	REV.	FIGURE
19130660	0004	A	4

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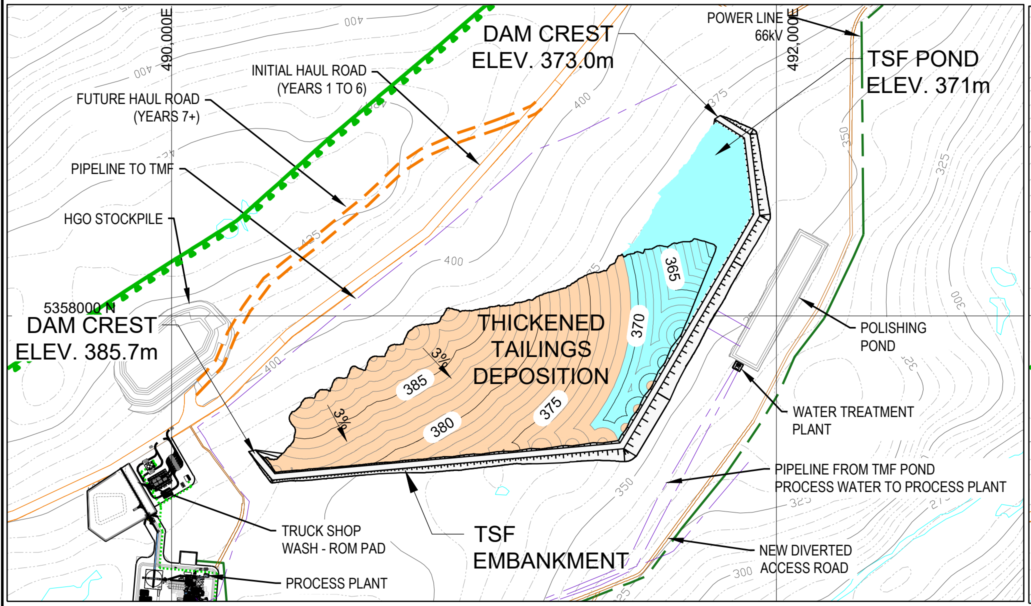
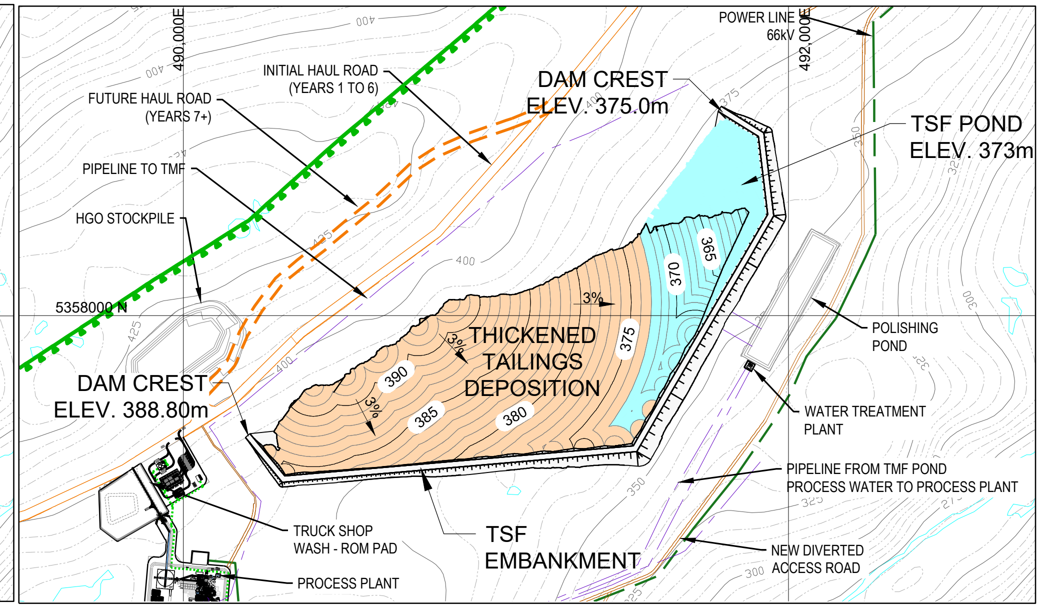
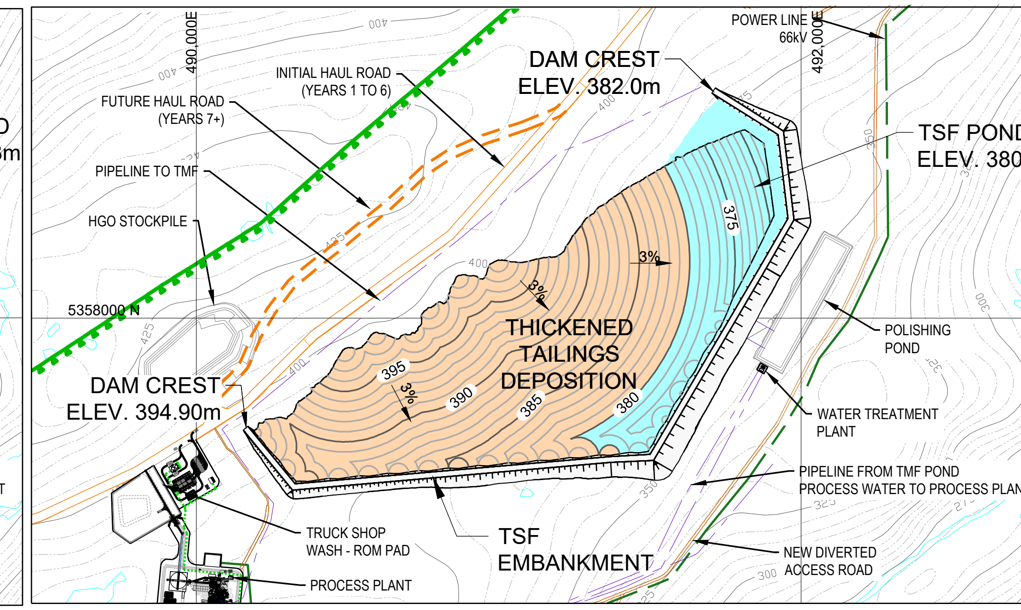


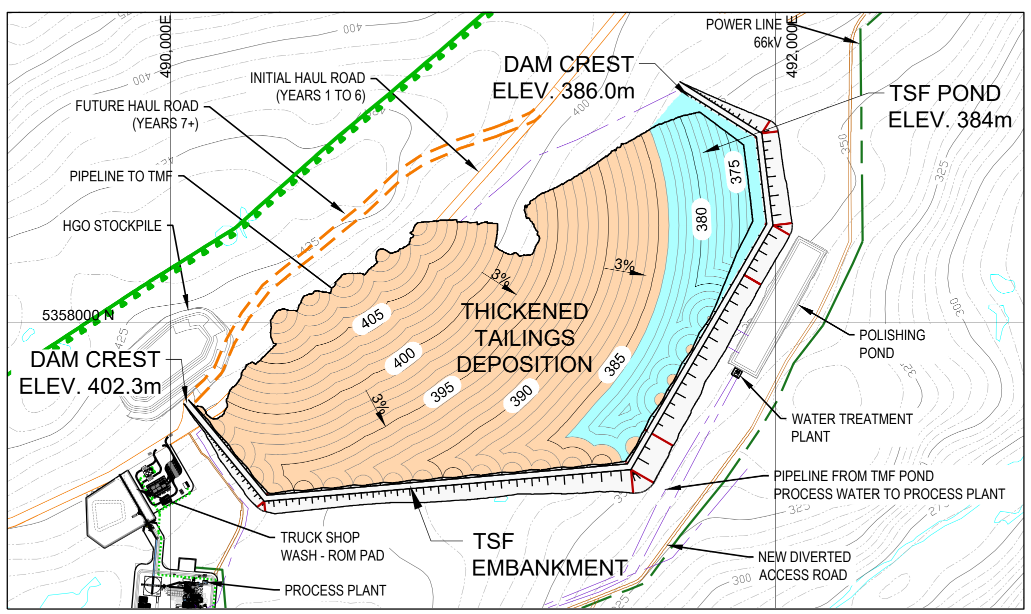
STAGE 1A PLAN - END OF YEAR 2



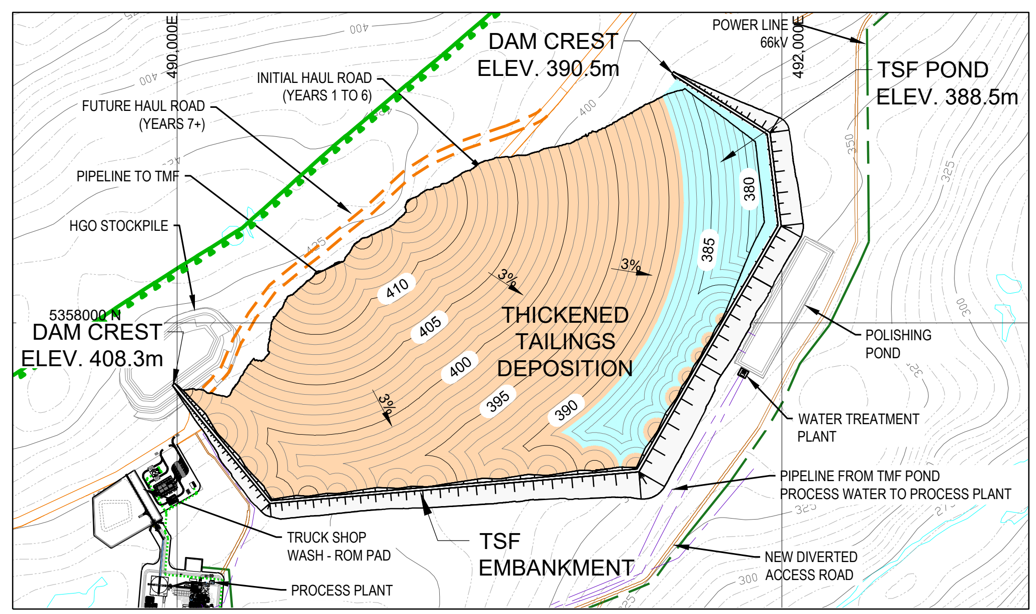
STAGE 1B PLAN - END OF YEAR 3



STAGE 2 PLAN - END OF YEAR 5



STAGE 3 PLAN - END OF YEAR 7



STAGE 4 PLAN - END OF YEAR 9

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LEGEND

- THRUST FAULT
- DEPOSITED TAILINGS
- TSF POND
- GROUND ELEVATION CONTOURS
- WATERCOURSE

NOTES:

1. ALL ELEVATIONS, GRID COORDINATES AND DIMENSIONS ARE IN METRES. GRID D COORDINATES ARE REFERENCE TO NAD83 UTM ZONE 21 DATUM.
2. ORIGINAL GROUND CONTOURS INTERVALS AT 5m.
3. STAGED DEVELOPMENT DOES NOT SHOW DAM CONSTRUCTION FOR THAT YEAR.

REFERENCE:
BASEPLAN PROVIDED BY CLIENT. FILENAME 104319-0000-G-001_20200122.DWG. REVISION B, DATED 20/01-10

CLIENT		PROJECT			
MARATHON GOLD		VALENTINE GOLD PROJECT NEWFOUNDLAND			
CONSULTANT	YYYY-MM-DD: JUNE 2020	TITLE			
GOLDER	DESIGNED: MS	**TMF STAGED DEVELOPMENT AND TAILINGS DEPOSITION PLAN - STAGES 1A TO 4**			
	PREPARED: TDR				
	REVIEWED: PA	PROJECT NO.	CONTROL	REV.	FIGURE
	APPROVED: WPM	19130660	0004	B	5

25 mm

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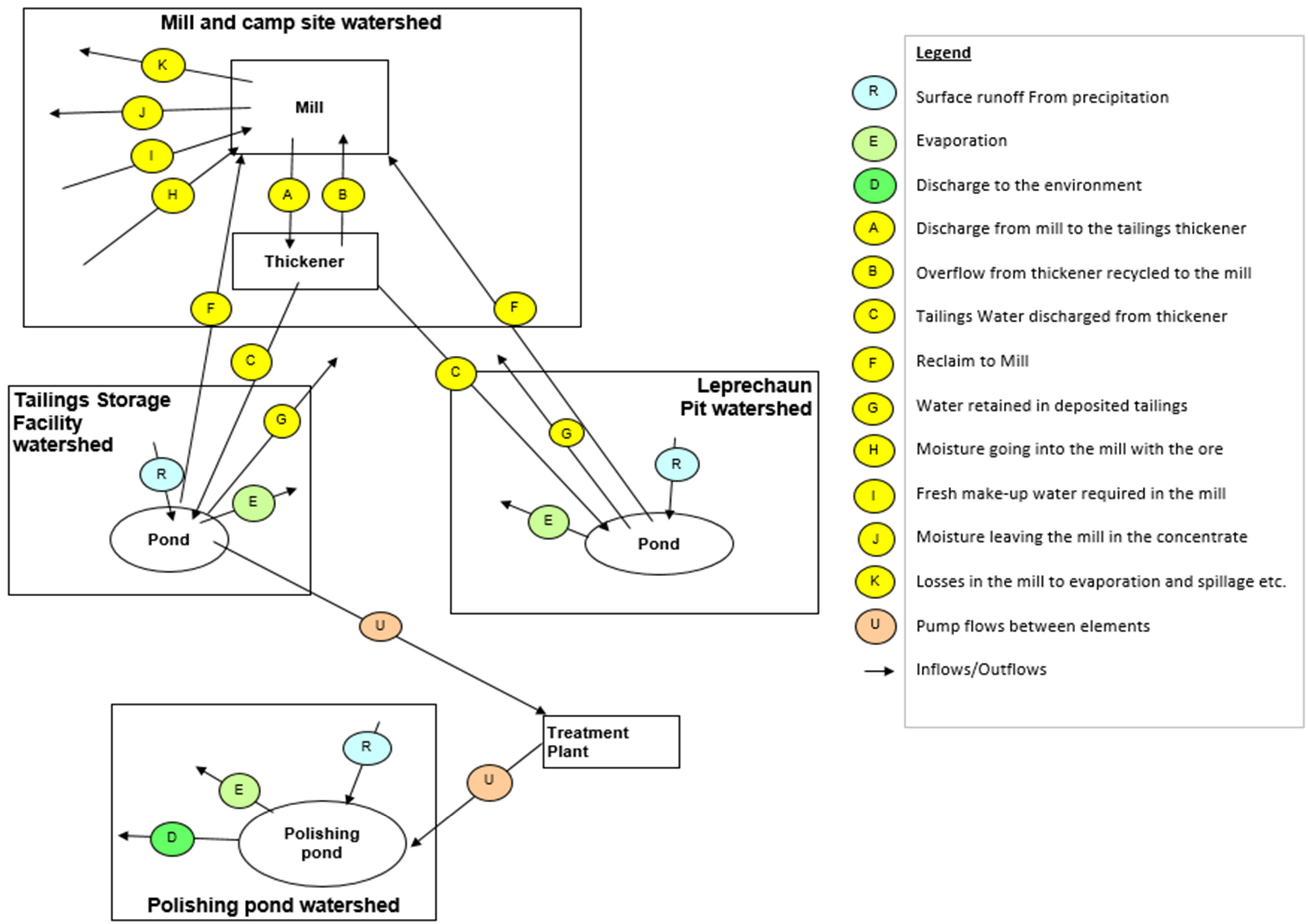

CLIENT
MARATHON GOLD CORPORATION

PROJECT
Valentine Gold Pre-feasibility Study

CONSULTANT
GOLDER

YYYY-MM-DD	2020-07-03
DESIGNED	PP
PREPARED	PP
REVIEWED	AC
APPROVED	WPM

TITLE
Water Balance Flow Diagram

TITLE	PHASE	REV	FIGURE
19130660	**4000**	**0**	**6**

APPENDIX A

TMF Site Selection and Trade-Off Study Update

GOLDER

TECHNICAL MEMORANDUM

DATE	18 February 2020	**Project No.** 19130660
TO	Robbert Borst Marathon Gold	
CC	Ausenco - Jared Dietrich	
FROM	Peter Merry	**EMAIL pmerry@golder.com**

VALENTINE GOLD PROJECT –TAILINGS STORAGE FACILTY SITE SELECTION STUDY

1.0 INTRODUCTION

Golder Associates Ltd. (Golder) has been retained by Marathon Gold Corporation (Marathon) to complete a pre-feasibility Study (PFS) level design of a tailings storage facility (TSF) for the Valentine Gold Project, in south central Newfoundland. The first stage of the PFS is to conduct a site selection study to identify the preferred location for the TSF. A TSF site selection study was previously completed by Stantec Consulting Ltd. (Stantec) as part of the Preliminary Economic Assessment (PEA) (Stantec 2018a). An updated siting study was required as part of the current PFS as new information is available with the advancement of the project. Engineering services to support the process design and mine planning are being provided by Ausenco and Moose Mountain Technical Services (Moose Mountain), respectively.

This memo presents the results of the updated study to identify a suitable site for tailings storage.

2.0 PROCEDURE

The site selection study followed the three steps listed below:

- Step 1: Selection of all viable siting options within the project area, including those previously identified during the PEA;
- Step 2: Definition of qualitative fatal flaw screening and site selection criteria to narrow down 3 siting options for trade-off;
- Step 3: Comparative evaluation/trade-off using relative ranking matrix and selection of preferred site and tailings disposal method.

The extent of tailings dewatering has an impact on the deposition and storage requirements, environmental performance, closure and project economics. These were considered in the overall assessment of tailings management alternatives. Thickened non-segregating tailings material will generally have a steeper beach compared to conventional slurry tailings and there is less supernatant water to manage. Filtered tailings are deposited as a "dry stack" product which reduces the requirement for perimeter dams but typically incurs higher capital and operating expenditure associated with the process and delivery. The development of tailings disposal options is therefore a combination of site selection, tailings preparation and deposition. The required revisions to the containment dams for the different tailings dewatering technologies being considered was accounted for in this study and further discussed in Section 6.1.

Capital and operating cost estimates for the TSF dams were estimated by Golder. The process design and delivery along with the associated costs of dewatering technologies was completed by Ausenco.

A site visit was made by Golder in October, 2019 prior to the site selection study. Golder participated in a project alignment kick-off meeting with representatives from Marathon, Moose Mountain, Stantec and Ausenco to discuss (amongst other project details) some of the TSF siting constraints and took part in a helicopter tour to observe the topography and natural features of the site to aid in site selection.

Topographic mapping used for the study was obtained from Marathon and comprised 5 m contour interval data over the broader project area and 1 m contour interval data from aerial survey in the area roughly bounded by Victoria River, Victoria Lake and Valentine Lake.

A fish habitat baseline study for the project was undertaken by Stantec in 2018 to support the assessment of potential interactions and environmental effects of the mine development on fish and fish habitat (Stantec, 2019). The study was supplemental to a survey completed in 2011 and focused on major ponds and watercourses within the watersheds potentially affected by the PEA conceptual layout of the project. Marathon provided an electronic layout file for ponds and watercourses referenced in Stantec's 2019 study and for two important unnamed tributaries (not part of the 2018 study) for use in this siting study.

3.0 TAILINGS OPERATIONAL DATA

A preliminary evaluation of the operating data is presented in Table 1. These data may be revised as the design advances; however assumptions were required as a basis for the site selection process.

The ore reserves for the Project comprise approximately 53 million metric tonnes (Mt). Currently the mine plan will consist of two open pits (Leprechaun Pit and Marathon Pit) with the potential for development of some smaller satellite pits (Sprite Pit, Victory Pit etc.). Hence the ultimate potential of the mine could be even larger and will be indicated with the results of ongoing drilling.

The waste rock to ore ratio over the life of mine is estimated to be 6.5:1. The proposed production rate is 1.5 Mt per year, increasing to 4 Mt per year in year 4 of operation and reducing to 2.6 Mt per year in the final year of operation (Year 15). Marathon is planning to deposit tailings in the TSF for the initial 10 years of production after which tailings will be deposited in the expired Leprechaun Pit (i.e. in-pit disposal). This amounts to a total tailings storage requirement of approximately 35 Mt in the TSF.

Marathon has indicated that Acid Rock Drainage (ARD) potential testing to date suggests that the tailings from Leprechaun and Marathon pits are non-potentially acid generating (non-PAG) which will simplify the management of tailings. Further testing is required to determine treatment requirements of the tailings supernatant water. Provision has been made for a separate polishing pond downstream of the tailings basin itself to provide flexibility in water management.

A tailings specific gravity of 2.68 has been estimated. Assuming a deposited void ratio and dry density for conventional slurry, thickened and filtered tailings technologies, the following resultant storage volumes required for 35 Mt of ore have been estimated:

- Conventional Slurry: 26.1 Mm^3

- Thickened Tailings: 24.8 Mm^3
- Filtered Tailings: 23.5 Mm^3

4.0 SITE SELECTION CRITERIA AND CONSTRAINTS

In general, the site selection study evaluated different constraints according to the following important criteria:

- **Basin capacity**. The selected site should have the potential to accommodate the required tailings tonnage and should be expandable.
- **Ore bodies (potential zones of mineralization).** A potential ore body or the extension of an existing ore body can be a major constraint. Ore is defined by the prevailing economic conditions which are subject to change. The selection of a tailings or mine rock disposal site should never be allowed to jeopardize the future mining of potential ore. Condemnation drilling is typically required.
- **Geological and hydrogeological considerations.** Stable, relatively watertight formations simplify dam construction. Mounded groundwater in the hillsides around a facility inhibits seepage and promotes a high groundwater level in a tailings mass thus inhibiting acid generation. Faults and structured rock are potential seepage pathways which have to be identified and potentially grouted.
- **Distance and elevation from the mill.** The shorter the distance from the mill the lower the costs will be for access roads, pipelines and pumping. In addition, disturbance and environmental risk are reduced as a consequence of shorter pipeline length, reduced interference with wildlife as well as less destruction or alteration to terrestrial and aquatic habitat.
- **Topographic relief.** Good relief enhances containment, minimizes dam construction, reduces aesthetic and environmental impacts and generally provides an inherently safer facility.
- **Storage capacity / containment dam volume ratio.** The larger the ratio, the lower will be the cost of dam construction per ton of ore milled. Small containment dams minimize environmental impacts and the volume of borrow (construction) materials required. In addition, small dams are inherently safer than large dams.
- **Watershed considerations.** The less water there is to manage and treat, the less expensive a waste facility will be to construct and manage. A small watershed or a location high in a watershed will minimize run-off, diversion and spillway costs and the quantity of runoff that will come in contact with the tailings or waste rock. On the other hand, an adequate supply of water, from the surrounding watershed, may be required at closure to ensure that acid generating materials remain flooded, if applicable. In this case, watershed diversion during the operating period should be considered.
- **Buffering zone.** A good buffering zone, between a disposal facility and the receiving water course, is desirable to provide space for effluent treatment and polishing ponds. In some cases, it may be desirable to locate a disposal facility in the same watershed as the mining and milling operations to avoid impacting more than one watershed.
- **Construction materials.** The availability of naturally occurring construction materials for containment structures close to a site minimizes haulage costs, access road construction and adverse environmental impacts. Use of waste rock from mine development can be utilized for dam construction if it is chemically stable, available at the required construction stage, and cost advantageous.

- **Existing land use.** The current and historical use of a proposed disposal area and the receiving watershed are important. Recreation, parks, native land claims, human habitation, archaeological considerations, mining, logging, farming, hunting and fishing are all important considerations.
- **Stakeholder Engagement.** Stakeholders can include local communities and provincial governments responsible for nearby infrastructure. There is nearly always a conflict with people and local communities. A disposal facility with small number of people living close by has a better chance of acceptance than a facility with a lot of people. Visual impact is frequently an issue with people living close to the site. Newfoundland & Labrador owns and operates a hydro dam on Victoria Lake.
- **Sustainability.** The restoration and after use of a waste disposal facility is an important consideration at the site selection stage.
- **Environmental constraints.** It is important to minimize the impact on the flora and fauna (including migration routes) within the area and off-site from contaminated surface water, groundwater and airborne dust. Strict regulations are in force in most jurisdictions.
- **Permitting.** The timeframes associated with permitting can be a major project constraint and must be understood at an early stage of study.
- **Property ownership.** Areas around mines often have complicated ownership and mineral rights, which may include patented land, mining claims, land use permits, easements, crown (government) land and aboriginal land claims. Patented land is the most difficult to deal with because it implies ownership of both the surface and mineral rights. Easements include power lines and transportation corridors. Crown (government) land is normally the easiest to deal with. The property ownership must be determined at the start of the project.
- **Closure.** The ease of decommissioning, long term liability, costs, monitoring and environmental impact are important. Habitat recovery and the overall potential for the establishment of a sustainable, reclaimable secure landscape have to be considered at the site selection stage.
- **Costs.** Capital, operating and decommissioning costs can vary tremendously from site to site. To allow for a comparison of options, quantities of the major cost items will be determined.

5.0 SELECTION OF POSSIBLE TSF SITES AND FATAL FLAW QUALITATIVE SCREENING

All the possible sites within an economical transport distance from the mill were identified. As a general philosophy, a possible site is any site large enough to hold tailings regardless of its suitability with respect to other criteria. It is important to identify all the possible sites at the outset to ensure that reconsideration and delays are avoided later in the design process.

A total of 14 possible sites for tailings disposal were identified as shown on Figure 1. Sites are numbered 1 through 14. Sites 9, 1 and 6 correspond to siting Options 1, 2 and 3 from the previous site selection study, respectively.

The possible sites are listed in Table 2. Site attributes along with comparative commentary regarding project specific criteria and constraints are provided in Table 2. The sites were presented to Marathon during a conference call held on November 26th, 2019. The following fatal flaw criteria were developed.

- Operational and Technical
  - Storage capacity limitations
  - Land ownership and land use restrictions (e.g. property boundary)
  - Potential Mineralization (e.g. areas north of thrust-fault are most likely sterilize ore bodies)
  - Limitations on tailings transport and access routes (e.g. complex bridge structures, bunded emergency discharge areas for power outages)
  - State of industry limitations (has technology been achieved at throughputs considered)
  - Water recovery requirements
  - Surface water diversion requirements
  - Interaction with external stakeholders' assets (e.g. Newfoundland & Labrador (NL) Hydro Victoria reservoir and dam)
- Environmental, Social and Permitting
  - Critical drainage basins and watershed impacts (surface water impacts)
  - Groundwater impacts
  - Air, noise and dust impacts
  - Population proximity (i.e. cabins, Newfoundland hydro staff, mine workers)
  - Environmental / culturally sensitive area proximity (in particular fish habitat, dams may overprint habitat, but tailings may not)
  - Permitting timeframe (Schedule 2 listing under federal Metal Mining Effluent Regulations, cellularization the TSF to delay permitting could be considered)
  - Animal migration routes (e.g. Caribou migration route roughly from northeast end of Valentine Lake through Marathon Pit and traversing Victoria River due southeast)

Marathon also provided feedback on all sites presented with respect to the viability of possible TSF sites. A summary discussion highlighting the key comparative criteria and potential fatal flaws for each of the possible sites is provided below.

Site 1 and Site 13 lie within the existing project footprint the same sub-watershed to Victoria Lake as some of the proposed site infrastructure (construction camp and Leprechaun waste dump). Site 1 has a moderate degree of natural topographic containment provided by the ridge on the north perimeter and hill at the southeast corner. Site 13 however has less efficient topographic containment. The receiving water course for these sites would be Victoria Lake which likely has good buffering potential due to its size. The sites are close to the proposed mill. Site 1 overlays a known fish habitat (Pond "L2") which would likely require Schedule 2 listing under the federal Metal Mining Effluent Regulation (MMER). Site 1 cannot be efficiently cellularized to delay Schedule 2 due to the presence of several other small unnamed ponds likely to contain fish. This was considered a fatal flaw for Site 1.

Site 13 may avoid any known fish habitats although unnamed tributaries to Victoria Lake to the east need to be avoided as they have high likelihood of fish given their connection to ponds. Site 13 was selected as a viable site for further consideration as a potential site.

Site 2 and **Site 14** lie within the existing project footprint and the same sub-watershed to Victoria River as the proposed mill site (although if selected, the mill would require relocation). The sites have moderate natural topographic containment provided by the ridge on the north perimeter. Site 14 is close to the caribou migration route and may act as a barrier. In the event of a dam failure, these sites are expected to have less of an impact on the NL hydro dam as run-out is likely to occur downstream of dam in the river, although backwater effects may cause inundation of the downstream toe. Site 2 overprints a known fish habitat (Tributary No. "14") and would likely require Schedule 2 listing under MMER). Site 2 was therefore considered to be fatally flawed. Site 14 avoids any known fish habitat and was selected as a viable site for further consideration.

Site 3, **Site 12 and Site 6** are located north of the thrust fault within the Valentine Lake sub-watershed. Site 12 is located within Valentine Lake which will likely require Schedule 2 listing, complex construction methods, is aesthetically unpleasant and may be considered by the regulators as the least favourable of the sites given that the tailings are non-acid generating and don't require a water cover. Site 3 is far from the mill, has limited expansion potential, poor topographic containment and likely requires Schedule 2 listing given overprinting of several small ponds likely to contain fish. Site 6 has a small and confined footprint with relatively poor storage and expandability. Site 6 also overprints known fish habitat (Pond "VALP3") likely requiring Schedule 2 listing. Sites 3, 12 and 6 were therefore all eliminated as potential sites for further consideration.

Site 4 and **Site 5** are located in the Victoria River valley downstream of the NL hydro dam, with Site 4 utilizing the NL hydro dam for containment. These sites would be very efficient in utilizing the natural containment of the valley but they require permanent diversions of the river, complicated interactions with the NL hydro dam to ensure operational safety and likely overprint fish habitat in the river. Site 5 also conflicts with the caribou migration route. These sites were not considered further due to the fatal flaws listed.

Site 7 and **Site 8** are located outside the current project footprint, far from the mill and east of the Victoria River valley. The sites provide moderate to high natural topographic containment utilizing the high relief in the area but would require permanent complex bridge structures over the river valley to access. Furthermore, pipeline crossings over the river would be required and would be generally viewed as unfavourable. The greatest fatal flaw for these sites is the direct conflict with the caribou migration route, and for this reason the sites were not considered further.

Site 9 is located just northeast of the Marathon pit and very near to the thrust fault which would require condemnation drilling to confirm no potential for ore sterilization. Site 9 overprints several fish habitats and cannot be cellularized to delay the likely Schedule 2 listing. For this reason, Site 9 was not considered worthy of further study.

Site 10 and **Site 11**, like Sites 7 and 8, are located east of the Victoria River Valley and outside the current project footprint. Except for conflict with the caribou migration route, Sites 10 and 11 suffer from the same fatal flaws as Sites 7 and 8 and are located even further from the mill which would incur greater costs. Site 10 lies primarily outside of the property boundary and, although not considered a fatal flaw by Marathon, application for surface rights would be required to develop this site. Site 10 likely impacts fish habitat. Site 11 may avoid fish habitat, but it is too far away compared to other sites (such as Site 13 and Site 14). Site 10 and 11 were therefore eliminated from further study.

6.0 COMPARISON OF POTENTIAL SITES

Of the fourteen possible TSF sites that were identified, three were designated as potential sites worthy of further consideration. These sites are Sites 1, 13 and 14. It is noted that Site 1 was fatally flawed (conflict with fish habitat) but was included for further study as a base case for comparison.

6.1 TSF Configurations

Preliminary TSF configurations and layouts for conventional slurry disposal, thickened tailings and filtered tailings were developed to aid in the comparison of the potential sites. This allowed for the quantification of the major costs items associated with the site development.

Conventional and thickened tailings deposition will require low-permeable perimeter dams for containment. A conceptual typical dam cross section was developed and is presented on Figure 2. The dams are proposed as downstream raised rockfill embankments. Observations made during the site visit and review of limited investigation data and surficial geology mapping suggests that there is no significant source of clays at the site for use as low permeable construction material in the dams (site dominated by granular tills), therefore a linear low density polyethylene (LLDPE) liner on the upstream face of the dams has been proposed. The upstream and downstream slopes of the dams are proposed at overall grades of 3.5H:1V and 2H:1V, respectively. The final dam crests are 10 m wide. Filter and transition zones below the liner are included to protect from piping in the event of liner failure. Seepage cut-off measures (such as foundation bedrock grouting and concrete plinth for anchoring the liner) have not been considered at this time.

Filtered tailings are proposed to be “stacked” by mechanical placement and compaction to an overall slope grade of 5H:1V, as presented on a proposed typical section on Figure 3. Provision for a granular toe drain to promote drainage of the “stack” and erosion protection on the slopes is included.

“Struck-level” stage storage relationships for each of the potential site basins and are shown on Figure 4. For each of the potential sites, preliminary deposition modelling for each tailings dewatering option (conventional, thickened and filtered) was carried out using Civil 3D. Target storage volumes for each option are provided in Table 1. Dams were modelled based on the typical section geometry and included a 2 m nominal freeboard. The TSF layouts for conventional slurry, thickened and filtered “stack” deposition options are shown on Figures 5 through 7. More accurate deposition modelling will be carried out during the PFS design stage.

6.2 TSF Construction Quantities and Cost Estimates

Preliminary earthwork construction quantities, capital expenditure (CapEx) and operating expenditure (OpEx) for the TSFs were estimated by Golder. The estimates undertaken for this study are provisional and for comparative purposes only. The earthworks quantities were estimated based on the conceptual typical sections and TSF configurations modelled in Civil 3D.

Golder’s CapEx costs considered dam construction (direct costs), site development and servicing (indirect costs) and closure costs. CapEx costs were also estimated on a yearly basis assuming 4 stages of dam construction over the life of the mine and spreading costs over each stage; 30% at Year 0, 25% at Years 2 and 4, 20% at Year 6.

Golder's OpEx costs for the TSF considered engineering studies, construction quality control and quality assurance, maintenance contractors, mine employees and equipment and lodging. OpEx costs for the TSF were estimated on an annual basis over the life of mine.

Unit rates rates previously established for the project from the 2018 PEA study (Stantec 2018b) or from Golder's experience with similar projects were utilized for the exercise. Golder's TSF quantity and cost estimates are collated in Appendix A. Our cost estimate is summarized in Table 3.

Ausenco completed cost estimates for processing and delivery for each of the tailings disposal options at the potential TSF sites. Ausenco's costs were built up from the latest mine plan and thickening/filtration test work. Golder provided the estimated costs for the TSF for inclusion in Ausenco's overall cumulative net present cost estimate analysis, which was broken down on a yearly basis and assumed a 5% yearly discount rate. The Ausenco cost analysis was provided to Golder and is included in Appendix B.

A summary of the estimated cumulative total costs plotted on a yearly basis are shown on Figure 8. All costs are presented in CAD. For each potential site, the thickened tailings deposition option had the lowest life of mine (LOM) net present cost and lowest CapEx cost.

6.3 Comparative Assessment

The 3 potential sites and variants for tailings deposition methods are compared in Table 4. A meeting was held on December 16th, 2019 with key members from Marathon, Ausenco, Golder and Moose Mountain to discuss the pros and cons of the potential sites and tailings deposition methods and to select a preferred alternative to advance the PFS design.

Site 1 is attractive as it has a relatively high storage efficiency compared to Sites 13 and 14 (tailings storage/dam fill ratio of 4.7 to 9.5 for conventional and thickened options, respectively) however it was ruled out for consideration given the fish habitat and permitting fatal flaws previously outlined in Section 5.0.

Site 13 has a short haul distance to Leprechaun pit for rockfill placement in the dams and is a slightly more efficient storage basin than Site 14. Although there are likely to be fish present in the unnamed tributaries, dams could be aligned such that no tailings overprint the watercourses. There are however some smaller ponds present which would require further fish survey studies to confirm no Schedule 2 listing would be necessary. Site 13 is within a sub-watershed of Victoria Lake and the extent of a dam breach may lead into the lake and impact the NL hydro dam. Filtered tailings deposition would greatly reduce the consequence of failure here and would allow for progressive closure during operations, however this option would be the most expensive to develop (total cost of $160 million).

Site 14 is a slightly less efficient basin but presents a significant advantage over Site 13 as it completely avoids known fish habitat and limits any schedule risk for permitting. A potential dam break would likely not directly impact the NL hydro dam, rather would inundate the Victoria River valley downstream. Some backwater flooding and inundation of the downstream toe could be experienced. Thickened, non-segregated tailings deposition here would further reduce the consequence of failure (lower relative dam heights than conventional tailings deposition). Although there is more operational complexity and risk (i.e. reliance on deposition from high ground and minimum beach slopes of 3% to achieve efficiency) it is the least expensive development option (total cost of $89 million). Development of Site 14 would require relocation of the plant site to the southwest.

Subsequent to the December 16th meeting, an evaluation matrix was prepared by Golder to reflect the comparative discussions and concluding results of the informal ranking workshop conducted by attendees of the meeting. The matrix is provided in Table 5.

The matrix presents relative rankings for each of the quantifiable criteria under the three main categories for comparison listed in Table 4; ‘Operational and Technical’, ‘Environmental and Social’, and ‘Financial’. Weightings for categories, and criteria within each category, were selected based on our understanding of Marathon’s sentiments. In general, given the sensitivity of environmental impacts to the projects’ development (e.g. avoidance of abundant fish habitat, dam failure consequences on the NL hydro dam and Victoria Lake and river), the ‘Environmental and Social’ category was most heavily weighted, followed by ‘Financial’ and lastly ‘Operational and Technical’.

In general, the matrix was completed by assigning relative point scores to each of the criteria within each category per TSF site and disposal option. Total point scores for each the categories were tallied and multiplied by the respective weighting to determine an overall total score, which is a percentage of the maximum total score achievable. Finally, a relative rank was assigned for each TSF site and disposal option.

7.0 CONCLUSIONS

Three TSF sites (Site 1, Site 13 and Site 14) were considered worthy for consideration as potential TSF sites after screening of a total of fourteen possible sites identified as part of the study. All three potential sites lie within the current project footprint and near other planned mine infrastructure. Although Site 1 imprints known fish habitat, it was considered as a base case in the comparative analysis of potential sites as it was selected as the preferred site for the PEA.

TSF Site 13 has a slight advantage in storage efficiency over Site 14 however is less attractive due to the presence of probable fish bearing tributaries on its northern and eastern flanks and some small ponds within the footprint. Moreover, a potential dam failure would result in tailings inundating Victoria Lake which could cause overtopping failure of the NL Hydro dam under extreme flood events. Given that the estimated costs for development of Site 13 and Site 14 are comparable (difference within 10% for each respective disposal method), TSF Site 14 is preferred by Marathon as it reduces project risk.

Of the three tailings deposition methods considered for Site 14, filtered tailings would present the lowest environmental and permitting schedule risks and allow for progressive reclamation during operations, however, it is the most expensive option and there are no compelling design drivers at this time to warrant filtered tailings disposal for the project. The total estimated cost over the LOM for thickened tailings deposition is lower (~40% less) than conventional disposal due to lower CapEx. The preferred disposal method for Site 14 is thickened tailings and should be carried forward for pre-feasibility level design.

The results of the relative ranking matrix evaluation supports the notion that TSF Site 14 should be carried forward as the preferred site with thickened tailings disposal ranking highest as the best technology for development.

8.0 CLOSURE

We trust that the information contained in this technical memorandum meets your requirements and expectations. Should you have any questions or concerns, please do not hesitate to contact the undersigned.

GOLDER ASSOCIATES LTD.

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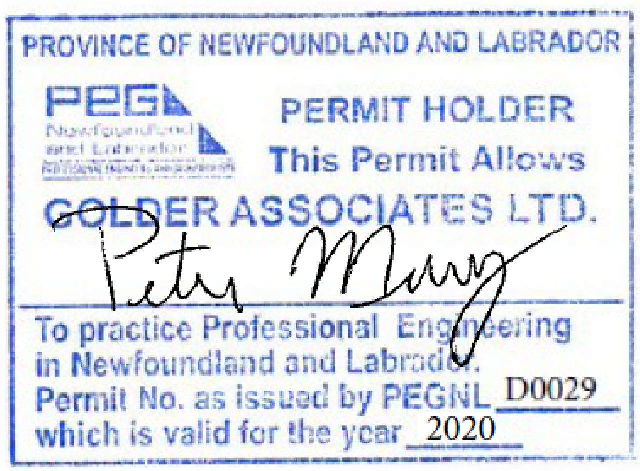

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https://golderassociates.sharepoint.com/sites/117078/project files/6 deliverables/2. trade-off study memo/final/19130660 valentine gold-tsf site selection study_final_18feb2020.docx

REFERENCES

Stantec, 2018a. Valentine Lake Gold Project – Preliminary Economic Assessment (PEA) – PEA Level Tailings Facility Site Selection Study. Memo prepared for Marathon Gold. January 25, 2018.

Stantec, 2018b. Valentine Lake PEA – TSF Stage 1 Configuration and Quantities. Memo prepared for Marathon Gold. April 2, 2018.

Stantec 2019. Valentine Lake Project: 2018 Fish and Fish Habitat Data Report. Report prepared for Marathon Gold. February 1, 2019.

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Table 1: Tailings Operational Data utilized in Site Selection Study

PARAMETER	VALUE	SOURCE
PRODUCTION		
Life of Mine	15 years	Ausenco
Ore Reserves [1.]	53.095 Mt	Ausenco
Tailings Disposal Location	TSF (years 1 to 10)	Marathon Gold
	Leprechaun Open Pit (years 11 to 15)	
Total Tailings to TSF	35.0 Mt	Marathon Gold
TAILINGS CHARACTERISTICS		
Design Slurry Density (solids concentration) (by mass)	Conventional:	Ausenco
	Thickened: 65%	
	Filtered: 86%	
Average Specific Gravity	2.68	Ausenco
Deposited Void Ratio (e) for Tailings	Conventional: 1.0	Golder
	Thickened: 0.9	
	Filtered: 0.8	
Average Deposited In-Situ dry Density	Conventional: 1.34 t/m^3	Assumed based on specific gravity and void ratio (e)
	Thickened: 1.41 t/m^3	
	Filtered: 1.49 t/m^3	
Deposited Beach Slope	Conventional: 1%	Golder
	Thickened: 3%	
Required Storage Volume of Tailings in TSF	Conventional: 26.1 Mm3	Assumed based on dry density
	Thickened: 24.8 Mm3	
	Filtered: 23.5 Mm3	

Note: 1. Includes ore from all sources (Marathon Pit, Leprechaun Pit, Victory Pit, Sprite Pits)

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TABLE 2
POSSIBLE TSF SITE COMPARATIVE SCREENING

CRITERIA	POSSIBLE TSF SITE OPTION NUMBER													
	1	2	3	4	5	6	7	8	9	10	11	12	13	14
Site Footprint Area	180 ha	201 ha	208 ha	222 ha	308 ha	155 ha	314 ha	351 ha	381 ha	252 ha	369 ha	349 ha	218 ha	265 ha
Surface Water Impact (watershed and diversion)	Low - Located in same sub-watershed as proposed mine infrastructure (Leprechaun Pit WRSF etc.) and no major diversions required.	Low - Located in same sub-watershed as proposed mine infrastructure (Process Plant etc.) and no major diversion required.	Low - Located in same sub-watershed as proposed mine infrastructure (Leprechaun Pit) and no major diversions required.	High - Runoff diversion ditches required along valley perimeter. Victoria Lake spillway discharge to bypass TSF.	High - Upstream section of Victoria River diversion required and Victoria Lake spillway discharge to bypass TSF	Low - Located in same sub-watershed as proposed mine infrastructure (Marathon WRSF etc) and minimal runoff diversion ditching required	Low to Moderate - Located in different adjacent sub-watershed to mine infrastructure but no diversions required	Low to Moderate - Located in different adjacent sub-watershed to mine infrastructure but no diversions required	Moderate - diversion of tributary "6" between Pond "M5" and "M6" required	Low to Moderate - Located in different adjacent sub-watershed to mine infrastructure but no diversions required.	Low to Moderate - Located in different adjacent sub-watershed to mine infrastructure but no diversions required.	Low - Occupies existing portion of lake and no diversions required	Low - Located in same sub-watershed as proposed mine infrastructure (leprechaun pit, leprechaun S WRSF etc) and minimal runoff diversion ditching required	Low - Located in same sub-watershed as proposed mine infrastructure (leprechaun pit, leprechaun S WRSF etc) and minimal runoff diversion ditching required
Fish Habitat Impact (with reference to 2018 Stantec study).	Yes - Pond "L2"	Yes - Tributary "14"	Unkown but likely due to small ponds present in footprint	Unkown but likely in Victoria River and contributory watercourses	Unkown but likely in Victoria River and contributory watercourses	Yes - Pond "VALP3"	Unkown but likely due to small ponds at west limit. Watercourses within footprint	Unkown but likely due to several small ponds in footprint and contributory watercourses	Yes - Ponds "M6", "M7", "M8". Tributaries "7" "8" and "9"	Unkown but likely due to presence of large ponds in centre of basin	Unkown but unlikely, no ponds present and limited watercourses from topographic high area	Yes - Valentine Lake	Basin avoids known fish habitat, perimeter dam overprints known fish habitat (local unnamed tributary to Victoria Lake at SE extent of TMF)	Basin avoids known fish habitat
Conflict with Caribou Migration Route	No	No	No	No	Yes	No	Yes	Yes	Yes	No	No	No	No	No - East perimeter dam could act as barrier to caribou
Permitting	Provincial and Federal (Schedule 2 under MMER)	Provincial and Federal (Schedule 2 under MMER)	Provincial and Federal (Schedule 2 under MMER)	Provincial and Federal (Schedule 2 under MMER)	Provincial and Federal (Schedule 2 under MMER)	Provincial and Federal (Schedule 2 under MMER)	Provincial and Federal (Schedule 2 under MMER)	Provincial and Federal (Schedule 2 under MMER)	Provincial and Federal (Schedule 2 under MMER)	Provincial and Federal (Schedule 2 under MMER)	Provincial	Provincial and Federal (Schedule 2 under MMER)	Provincial	Provincial
Social Impact / Visual Effects	Low - adjacent to other mine facilities	Low - adjacent to other mine facilities	Moderate - visible to any recreational users of Valentine Lake	Low - confined to valley	Low - confined to valley	Low - adjacent to proposed Marathon waste rock dump	Moderate - more visible due to high ground area	Moderate - more visible due to high ground area	Low - adjacent to other mine facilities	Low - isolated low visibilty area	Moderate - more visible due to high ground area	High - Poor aesthetics associated with "in-lake" disposal	Low - adjacent to other mine facilities	Low - adjacent to other mine facilities
Relative Storage Capacity (based on footprint area only)	Low	Moderate	Moderate	Moderate	High	Low	High	High	High	Moderate	High	High	Moderate	Moderate
Elevation Difference to Mill (~El. 360 m) from Centre of Basin	20 m	20 m	-20 m	-85 m	-90 m	-30 m	40 m	60 m	-50 m	-25 m	-35 m	-40 m	0 m	Overprints current mill location (mill requires relocating)
Distance from the Mill (straight line to centre of basin)	2.3 km	0.9 km	5.7 km	1.0 km	1.9 km	2.1 km	3.3 km	4.4 km	5.0 km	8.0 km	8.5 km	4.3 km	3.7 km	Overprints current mill location (mill requires relocating)
Potential for Expansion	Yes - Good potential east and west	Yes - Good potential west	Yes - poor potential southwest	Yes - good potential further down river valley	Yes - good potential further up or down river valley	No - limited by Valentine Lake and Marathon Pit	Yes - good potential west into river valley or east	Yes - good potential utilizing surrounding topography	Yes - moderate potential east into river valley and west	Yes - good potential utilizing surrounding topography	Yes - good potential utilizing surrounding topography	Yes - good potential utilizing surrounding topography	Yes - good potential east	Yes - Good potential west
Topographic Containment	Moderate	Moderate	Poor	Good	Good	Moderate	Moderate	Good	Moderate	Good	Moderate	Moderate (depending on bathymetry)	Moderate	Moderate
Potential Mineralization (North of Slip Fault)	No	No	Yes	No	No	Yes	No	No	Yes	No	No	Yes	No	No
Property Boundary	Within	Within	Within	Within	Within	Within	Within	Partially outside	Within	Primarily outside	Within	Within	Within	Within
Water Management	Seepage collection in perimeter ditching / berming, polishing pond and direct discharge to environment or pump back into TMA.											In lake - requiring robust seepage barrier	Seepage collection in perimeter ditching / berming, polishing pond and direct discharge to environment or pump back into TMA.	
Receiving Watercourse and Buffering Potential	Victoria Lake, good buffering potential. Space available for polishing pond.	Victoria Lake and/or Victoria River, Good buffering potential for Lake, moderate for River. Space available for polishing pond.	Valentine Lake, moderate buffering potential, limited space for polishing pond	Victoria River, poor buffering potential. Space available for polishing pond.	Victoria River, poor buffering potential. Space available for polishing pond.	Valentine Lake, moderate buffering potential, limited space for polishing pond	Victoria River, poor buffering potential, space available for polishing pond	Victoria Lake or tributary from Red Cross Lake to Victoria River, good buffering potential, space available for polishing pond	Victoria River, poor buffering potential. Limited space available for polishing pond.	Tributary from Red Cross Lake to Victoria River, good buffering potential, space available for polishing pond	Victoria River, moderate buffering potential. Limited space available for polishing pond.	Valentine Lake and creek from Valentine Lake to Victoria Lake, moderate buffering potential, limited space for polishing pond.	Victoria Lake, good buffering potential, limited space for polishing pond.	Victoria River (downstream of Victoria Dam), moderate buffering potential, space available for polishing pond.
Construction complexity	Standard. Feasible to construct with direct haulage from Marathon or Leprechaun Pits	Standard. Feasible to construct with direct haulage from Marathon or Leprechaun Pits	Standard. May be feasable to construct with direct haulage from Leprechaun Pit	Moderate due to spillway diversion from Victoria Lake. Direct haulage from Marathon Pit possible	Moderate due to spillway diversion from Victoria Lake. Direct haulage from Marathon Pit possible	Moderate (low lying area, dewatering likely required). Feasible to construct with direct haulage from Marathon Pit	Requires river crossing for construction equipment. Direct haulage from open pits not likely feasible.	Requires river crossing for construction equipment. Direct haulage from open pits not likely	Standard. Direct haulage from Marathon Pit feasible	Requires river crossing. Direct haulage from open pits not likely feasible.	Requires river crossing. Direct haulage from open pits not likely feasible.	Complex (in-lake construction). Likely not feasable to construct with direct haulage from Leprechaun Pit	Standard. Feasible to construct with direct haulage from Marathon or Leprechaun Pits	Standard. Feasible to construct with direct haulage from Marathon or Leprechaun Pits
Dam Inundation Risk	Mine Camp, Victoria Lake recreational (assume Victoria Dam does not fail)	Mine Camp, Plant Operators and Victoria Lake Recreational (assume Victoria Dam does not fail)	Recreational Valentine Lake	Downstream of Victoria River, Victoria Lake dam operators	Downstream of Victoria River, Victoria Lake dam operators	Recreational Valentine Lake	Downstream of Victoria River	Victoria Lake recreational and downstream of Victoria River	Downstream of Victoria River	Downstream of Victoria River	Downstream of Victoria River, assume Victory Pit is not impacted	Recreational Valentine Lake	Mine Camp and Victoria Lake Recreational (assume Victoria Dam does not fail)	Mine Camp, Plant Operators (depending on relocated plant site) and downstream of Victoria Dam (assume Victoria Dam does not fail)

Table 3: TSF Cost Estimate Summary (Golder)

TSF SITE AND DISPOSAL METHOD		Direct CapEx ($M)	Indirect CapEx ($M)	OpEx ($M)	TOTAL COST ($M)
Site 1	Conventional	45.2	6.0	13.6	**64.8**
	Thickened	28.4	3.5	13.6	**45.5**
	Filtered	11.5	1.1	11.0	**23.5**
Site 13	Conventional	72.3	9.9	13.6	**95.8**
	Thickened	46.9	6.1	13.6	**66.5**
	Filtered	14.6	1.3	11.0	**27.0**
Site 14	Conventional	82.9	11.7	13.6	**108.2**
	Thickened	57.9	7.9	13.6	**79.4**
	Filtered	14.6	1.4	11.0	**27.0**

Note: 1. Costs estimated are in CAD.

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TABLE 4
COMPARISON OF POTENTIAL TSF SITES AND DISPOSAL OPTIONS

Operational and Technical

TSF Sites: Site Option No.	TSF Sites: Alternative	Tailings Dewatering Consistency: Conventional Slurry	Tailings Dewatering Consistency: Thickened Tailings	Tailings Dewatering Consistency: Filtered Tailings	Potential for Expansion	Maximum Dam / Filtered Stack Height (m)	TSF Footprint Area (ha)	Topographic Containment	Bulk Dam Volume (M-m^3)[1]	Storage Capacity / Dam Volume Ratio	Elevation Difference to Mill (~El. 360 m) from Centre of Basin	Distance from the Mill (straight line to centre of basin)	Tailings Deposition / Operational Complexity	Construction Complexity	Construction Material Haul Length	Water Management and Discharge Requirements	Potential for Sterilization of Ore Bodies
1	A	X			Yes - Good potential east and west	34	180	Moderate to Good	5.5	4.7	20 m	2.3 km	Standard - spigot from dam perimeters	Standard	Close to Leprechaun Pit	Seepage collection in perimeter ditching / berming, polishing pond and direct discharge to environment or pump back into TMA or Mill.	No - south of thrust fault line
	B		X			25	160		2.6	9.5			Complex, may need central tower and spigots to keep pond forming against dam. Tailings beach slope determines storage capacity			Seepage collection in perimeter ditching / berming, polishing pond and direct discharge to environment or pump back into TMA or Mill.	
	C			X		34	118		0.5	-			Standard - Mechanical placement, additional requirments for winter	Moderate - need to dewater several surface water ponds		No tailings pond. Sedimentation pond for surface run-off	
13	A	X			Yes - good potential east	55	210	Moderate	11.6	2.3	0 m	3.7 km	Standard - spigot from dam perimeters	Standard	Close to Leprechaun Pit but far from Marathon Pit	Seepage collection in perimeter ditching / berming, polishing pond and direct discharge to environment or pump back into TMA or Mill.	No - south of thrust fault line
	B		X			44	190		6.4	3.9			Complex, may need central tower and spigots to keep pond forming against dam. Tailings beach slope determines storage capacity			Seepage collection in perimeter ditching / berming, polishing pond and direct discharge to environment or pump back into TMA or Mill.	
	C			X		54	159		0.6	-			Standard - Mechanical placement, additional requirments for winter			No tailings pond. Sedimentation pond for surface run-off	
14	A	X			Yes - Good potential west	62	180	Moderate	14.6	1.8	Overprints current mill location (mill requires relocating)	Overprints current mill location (mill requires relocating)	Standard - spigot from dam perimeters	Standard	Close to Marathon Pit but far from Leprechaun Pit	Seepage collection in perimeter ditching / berming, polishing pond and direct discharge to environment or pump back into TMA or Mill.	No - south of thrust fault line
	B		X			51	160		8.7	2.9			Complex, may need central tower and spigots to keep pond forming against dam. Tailings beach slope determines storage capacity			Seepage collection in perimeter ditching / berming, polishing pond and direct discharge to environment or pump back into TMA or Mill.	
	C			X		52	142		0.5	-			Standard - Mechanical placement, additional requirments for winter			No tailings pond. Sedimentation pond for surface run-off	

Financial

Site Option No.	Alternative	Total Initial CAPEX ($M)[4]	Total Sustaining CAPEX ($M)[4]	Total OPEX ($M)[4]	Total Net Present Cost (discounted 5% annually) ($M)[4]
1	A	19.5	41.8	48.2	87
	B	17.1	28.0	46.5	72
	C	43.0	25.8	119.1	154
13	A	29.2	64.0	48.2	115
	B	23.5	43.2	46.5	91
	C	44.2	28.7	123.3	160
14	A	32.3	71.2	48.2	125
	B	22.9	41.6	46.5	89
	C	44.4	27.9	112.8	151

Environmental and Social

Site Option No.	Alternative	Existing Land Use and Property Boundary	Closure Requirements[3]	Surface Area (ha)	Progressive Closure	Surface Water Impact (Watershed and Diversions)	Receiving Watercourse and Buffering Potential	Hydrogeological / Groundwater Impacts	Potential Social Impacts / Visual Effects	Dam Break Inundation Risk	Fish Habitat Impact (with reference to 2018 Stantec study)	Permitting Requirements
1	A	Within property boundary and proposed mine footprint	Regrading and vegetated soil cover	155	Must wait until end of operations for closure commencement	Low - Located in same sub-watershed as proposed mine infrastructure (leprechaun pit, leprechaun S WRSF etc) and minimal runoff diversion ditching required	Victoria Lake, good buffering potential. Space available for polishing pond.	Foundation conditions not well understood - may require grouting of dam foundation and/or seepage cut-off trench. Seepage collection ditches required to pump seepage water back to pond	Low - adjacent to other mine facilities	Mine Camp, Victoria Lake (assume Victoria Dam does not fail)	Yes - Pond "L2", and several other small ponds which likely have fish present.	Provincial and Federal (Schedule 2 under MMER)
	B			147								
	C			118	Can progressively close slopes of stack during operation			Minimal impact as tailings are "dry", surface water managed with ditches and sump		Same as above but with limited run-out extent due to removal of supernatant pond		
13	A	Within property boundary and proposed mine footprint	Regrading and vegetated soil cover	176	Must wait until end of operations for closure commencement	Low - Located in same sub-watershed as proposed mine infrastructure (leprechaun pit, leprechaun S WRSF etc) and minimal runoff diversion ditching required	Victoria Lake, good buffering potential. Space available for polishing pond.	Located close to Victoria Lake. Foundation conditions not well understood - may require grouting of dam foundation and/or seepage cut-off trench. Seepage collection ditches required to pump seepage water back to pond	Low - adjacent to other mine facilities	Mine Camp and Victoria Lake (assume Victoria Dam does not fail)	Basin avoids known fish habitat, perimeter dam overprints known fish habitat (local tributary to Victoria Lake at SE extent of TMF), some very small ponds present (need to confirm no fish present).	Provincial
	B			174								
	C			160	Can progressively close slopes of stack during operation			Minimal impact as tailings are "dry", surface water managed with ditches and sump		Same as above but with limited run-out extent due to removal of supernatant pond	Footprint avoids known fish habitat however some very small ponds present (need to confirm no fish present).	
14	A	Within property boundary and proposed mine footprint	Regrading and vegetated soil cover	144	Must wait until end of operations for closure commencement	Low - Located in small sub-watershed and minimal runoff diversion ditching required	Victoria River, moderate buffering potential, space available for polishing pond.	Foundation conditions not well understood - may require grouting of dam foundation and/or seepage cut-off trench. Seepage collection ditches required to pump seepage water back to pond	Low - adjacent to other mine facilities	Mine Camp, Plant Operators (depending on relocated plant site) and downstream of Victoria Dam	Basin avoids known fish habitat.	Provincial
	B			153								
	C			143	Can progressively close slopes of stack during operation			Minimal impact as tailings are "dry", surface water managed with ditches and sump		Same as above but with limited run-out extent due to removal of supernatant pond		

Notes:
1. Bulk volume for the filtered tailings option is for the toe drain installation.
2. Target storage volumes are estimated to be 26.1 Mm^3 and 24.8 Mm^3 for conventional and thickened tailings, respectively, and is based on assumptions of void ratios.
3. It is understood that the tailings and waste rock are NAG. Closure requirements may change as a result of geochemical stability of mine waste.
4. Total initial and sustaining CAPEX, OPEX, and Total Net Present Cost (discounted at 5% annually) were provided by Ausenco.

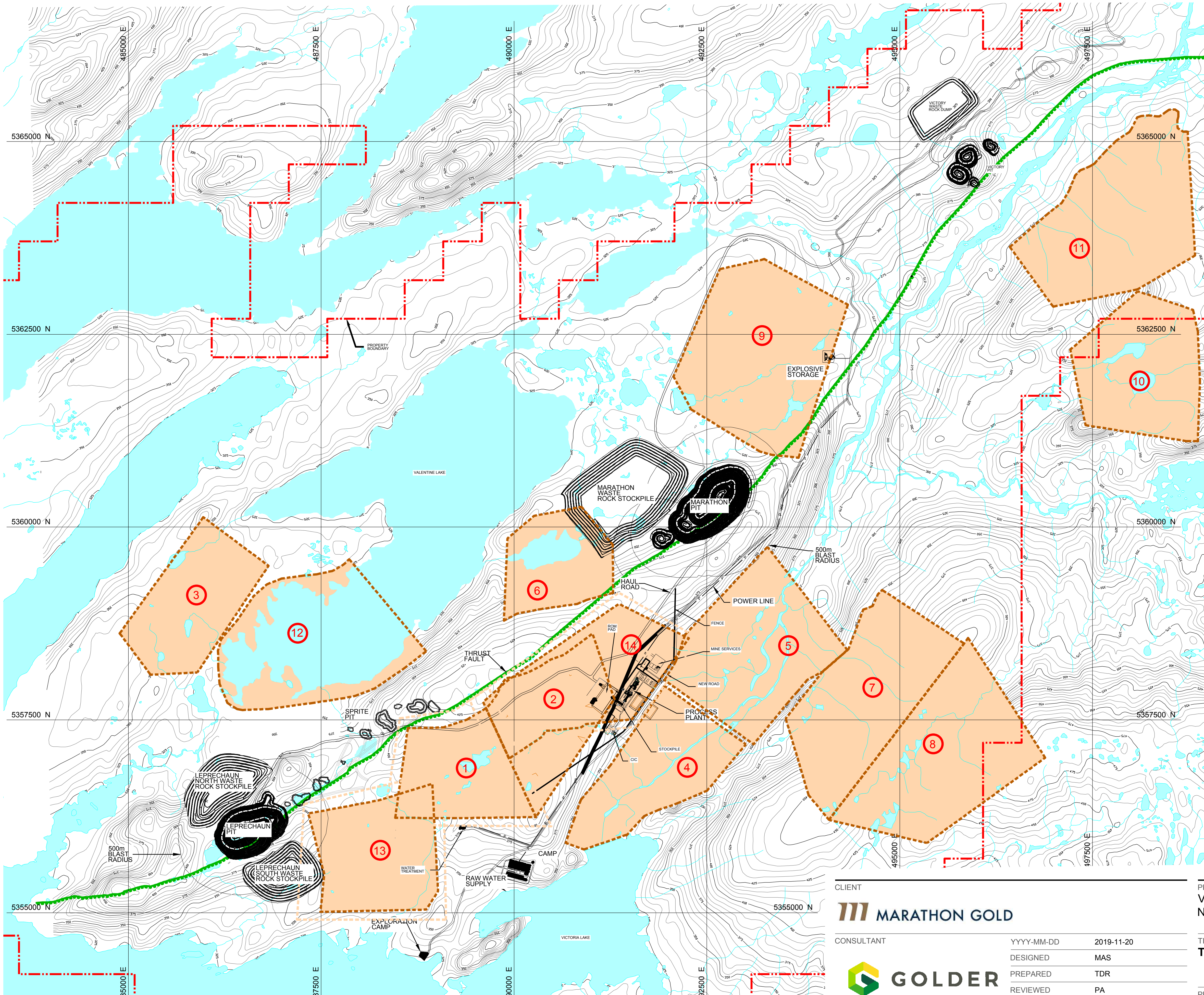
TABLE 5
EVALUATION MATRIX FOR POTENTIAL TSF SITES

		Criteria	Maximum Score	Option Number: Site 1 – 1A Conventional	1B Thickened	1C Filtered	Site 13 – 13A Conventional	13B Thickened	13C Filtered	Site 14 – 14A Conventional	14B Thickened	14C Filtered
ENVIRONMENT	1	Fish habitat impact	20	1	1	1	11	11	11	20	20	20
	2	Dam break inundation risk	10	4	5	8	1	4	7	7	7	10
	3	Permitting requirements / Stakeholder engagement	10	1	1	1	5	5	5	10	10	10
	4	Receiving watercourse and buffering potential	5	4	4	5	4	4	5	1	1	2
	5	Surface water impact (watershed and diversions)	5	2	3	5	1	2	3	3	3	4
	6	Progressive closure	5	1	2	5	1	2	5	1	2	4
		Total Points	55	13	16	25	23	28	36	42	43	50
		Score		0.24	0.29	0.45	0.42	0.51	0.65	0.76	0.78	0.91
FINANCIAL	1	Initial Capex (Yr 0)	10	9	10	1	5	8	1	5	8	1
	2	Sustaining Capex (Yr 1-13 + Closure)	10	6	10	10	3	6	10	1	6	10
	3	Opex	10	10	10	3	10	10	3	10	10	1
		Total Points	30	25	30	14	18	24	14	16	24	12
		Score		0.83	1.00	0.47	0.60	0.80	0.47	0.53	0.80	0.40
Operational and Technical	1	Potential for expansion	5	5	5	5	2	2	2	1	1	1
	2	Maximum dam / filtered stack height	5	5	5	5	2	2	2	1	1	1
	3	Bulk dam volume	5	3	4	5	1	2	5	1	2	5
	4	Distance to mill	5	3	3	3	1	1	1	5	5	5
	5	Storage capacity / dam volume ratio	5	3	4	5	1	2	5	1	2	5
	6	Tailings deposition/operational complexity	10	10	7	1	10	7	1	10	7	1
	7	Dam Construction complexity	5	4	4	5	2	2	5	1	1	5
	8											
	9											
	10											
		Total Points	40	33.00	32.00	29.00	19.00	18.00	21.00	20.00	19.00	23.00
		Score		0.83	0.80	0.73	0.48	0.45	0.53	0.50	0.48	0.58
Remark												

Notes: 1. Scoring system for criteria is on a relative scale. Best-performing option must be scored maximum points, worst-performing must be scored lowest points. High scores are better.

SUMMARY

Criteria	Maximum Score	Option Number: 1A Conventional	1B Thickened	1C Filtered	13A Conventional	13B Thickened	13C Filtered	14A Conventional	14B Thickened	14C Filtered
Environment	1.50	0.35	0.44	0.68	0.63	0.76	0.98	1.15	1.17	1.36
Financial	1.00	0.83	1.00	0.47	0.60	0.80	0.47	0.53	0.80	0.40
Operation and Technical	0.50	0.41	0.40	0.36	0.24	0.23	0.26	0.25	0.24	0.29
Total Points	3.00	1.60	1.84	1.51	1.46	1.79	1.71	1.93	2.21	2.05
Overall Score		**53.35%**	**61.21%**	**50.37%**	**48.83%**	**59.62%**	**57.03%**	**64.29%**	**73.67%**	**68.37%**
Ranking		**7**	**4**	**8**	**9**	**5**	**6**	**3**	**1**	**2**



LEGEND

6 TMF FOOTPRINT (APPROX.)

PROPERTY BOUNDARY

THRUST FAULT

NOTE(S):
1. PROJECTION: NAD83 UTM ZONE 21.

REFERENCE:
BASEPLAN PROVIDED BY CLIENT. FILENAME 5091-101-GEDGA-0003.DWG. REV.E DATED 08MAR19.

PRELIMINARY FOR DISCUSSION

0 1,000 2,000
METRES

CLIENT			PROJECT
MARATHON GOLD			VALENTINE GOLD PROJECT NEWFOUNDLAND
CONSULTANT	YYYY-MM-DD	2019-11-20	TITLE
GOLDER	DESIGNED	MAS	TMF SITING OPTIONS
	PREPARED	TDR	
	REVIEWED	PA	
	APPROVED	WPM	

PROJECT NO.	CONTROL	REV.	FIGURE
19130660	0001	A	1

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

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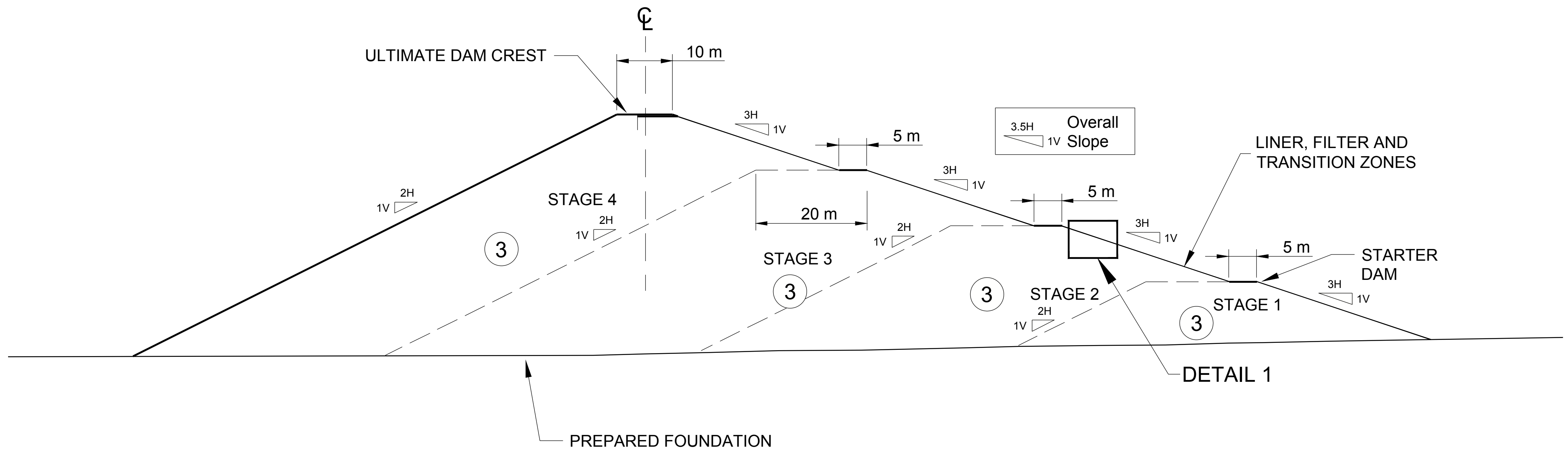
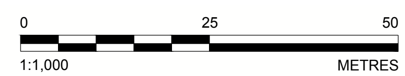
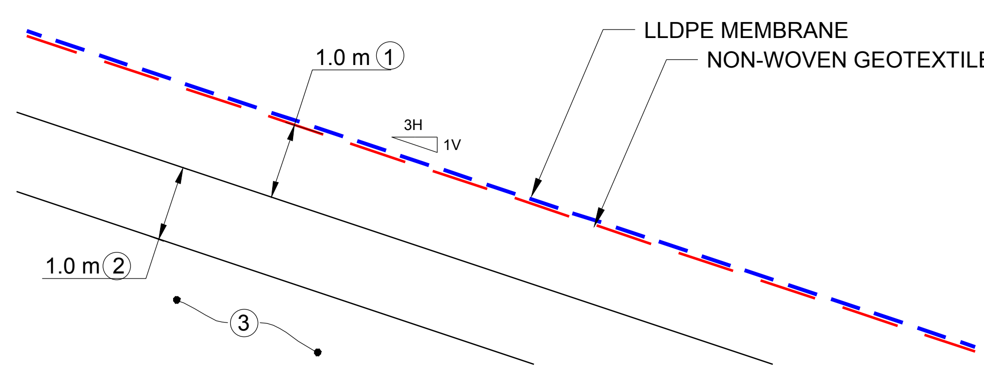


TYPICAL CROSS-SECTION





SCALE 1:100 1 - DETAIL 1: TYPICAL SLOPE DETAIL

FILL MATERIALS		
ZONE	1	SAND BEDDING / FILTER SAND
ZONE	2	TRANSITION
ZONE	3	ROCKFILL

PRELIMINARY FOR DISCUSSION

CLIENT
MARATHON GOLD

CONSULTANT
GOLDER

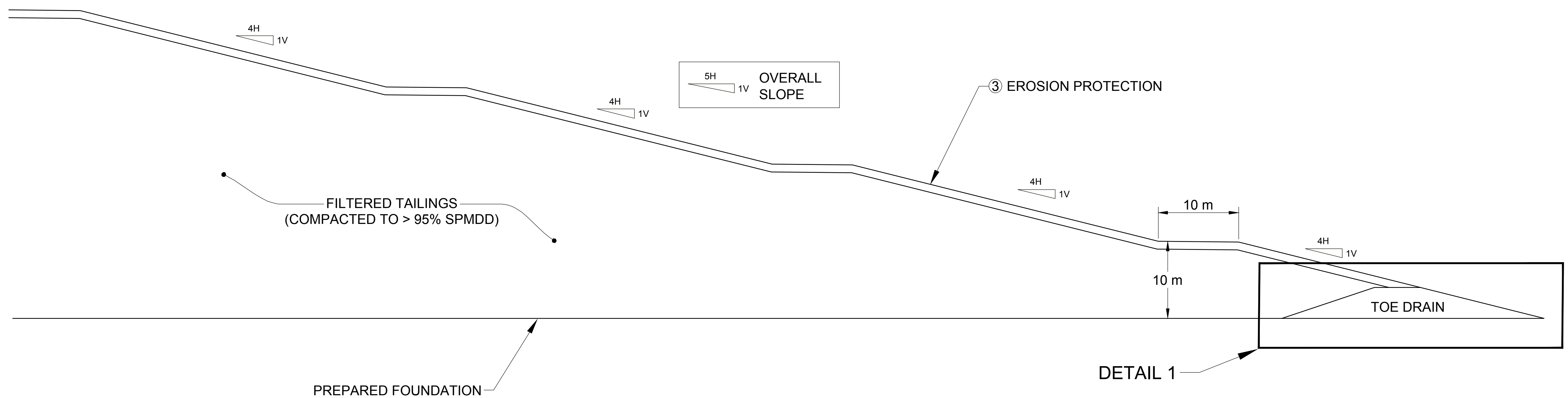
YYYY-MM-DD	2019-12-05
DESIGNED	MS
PREPARED	TDR
REVIEWED	PA
APPROVED	WPM

PROJECT
VALENTINE GOLD PROJECT
NEWFOUNDLAND

TITLE
TYPICAL DAM SECTION FOR CONVENTIONAL AND THICKENED TAILINGS OPTIONS

PROJECT NO.	CONTROL	REV.	FIGURE
19130660	0001	A	2

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



TYPICAL SECTION

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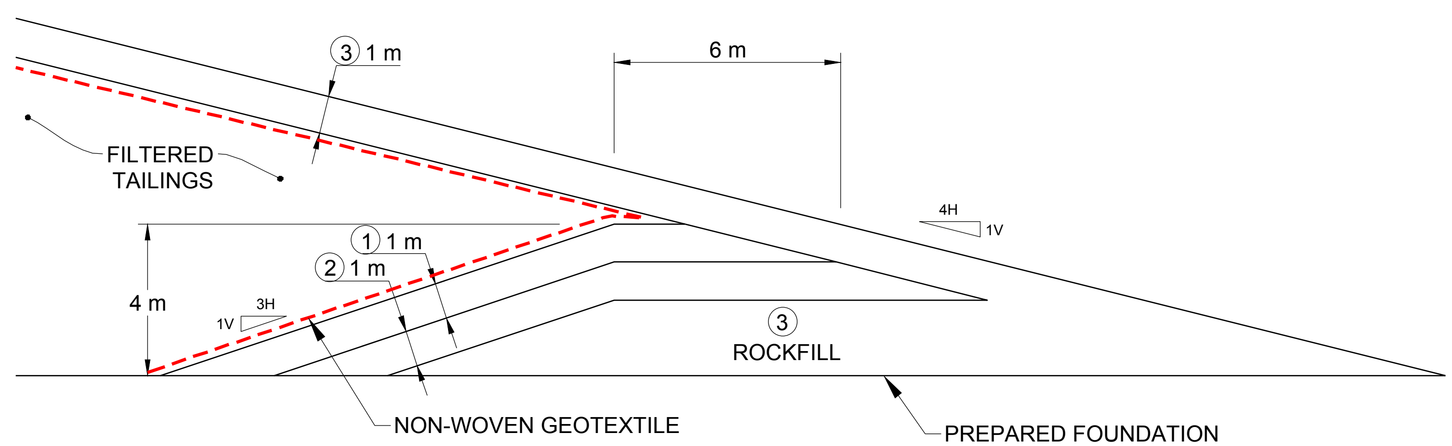
1:600 METRES



DETAIL 1 : TOE DRAIN AND SLOPE PROTECTION

0 5 10
1:200 METRES

FILL MATERIALS		
ZONE	1	SAND BEDDING / FILTER SAND
ZONE	2	TRANSITION
ZONE	3	ROCKFILL

PRELIMINARY FOR DISCUSSION

CLIENT
MARATHON GOLD

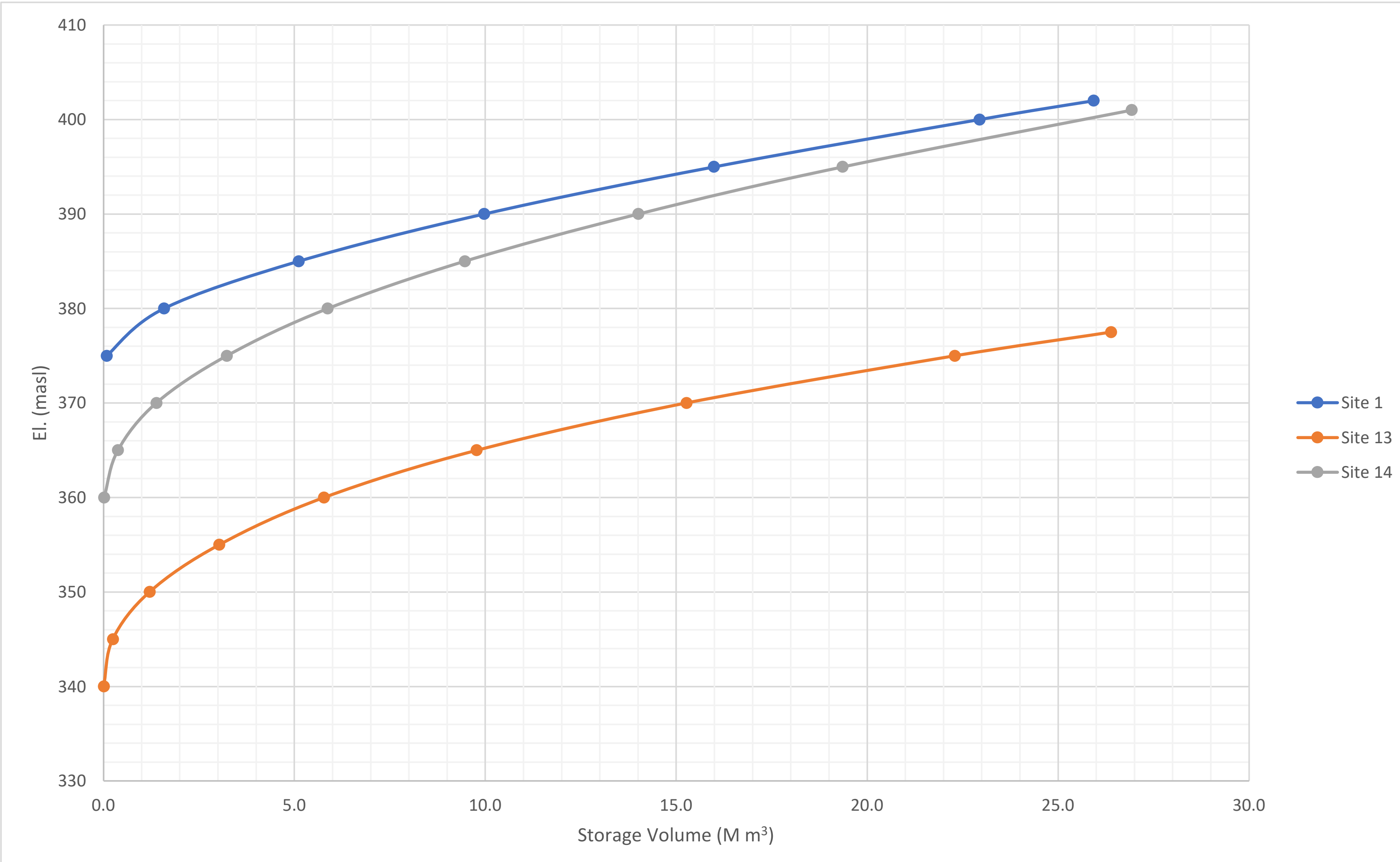
CONSULTANT
GOLDER

YYYY-MM-DD	2019-11-12
DESIGNED	PA
PREPARED	TDR
REVIEWED	PA
APPROVED	WPM

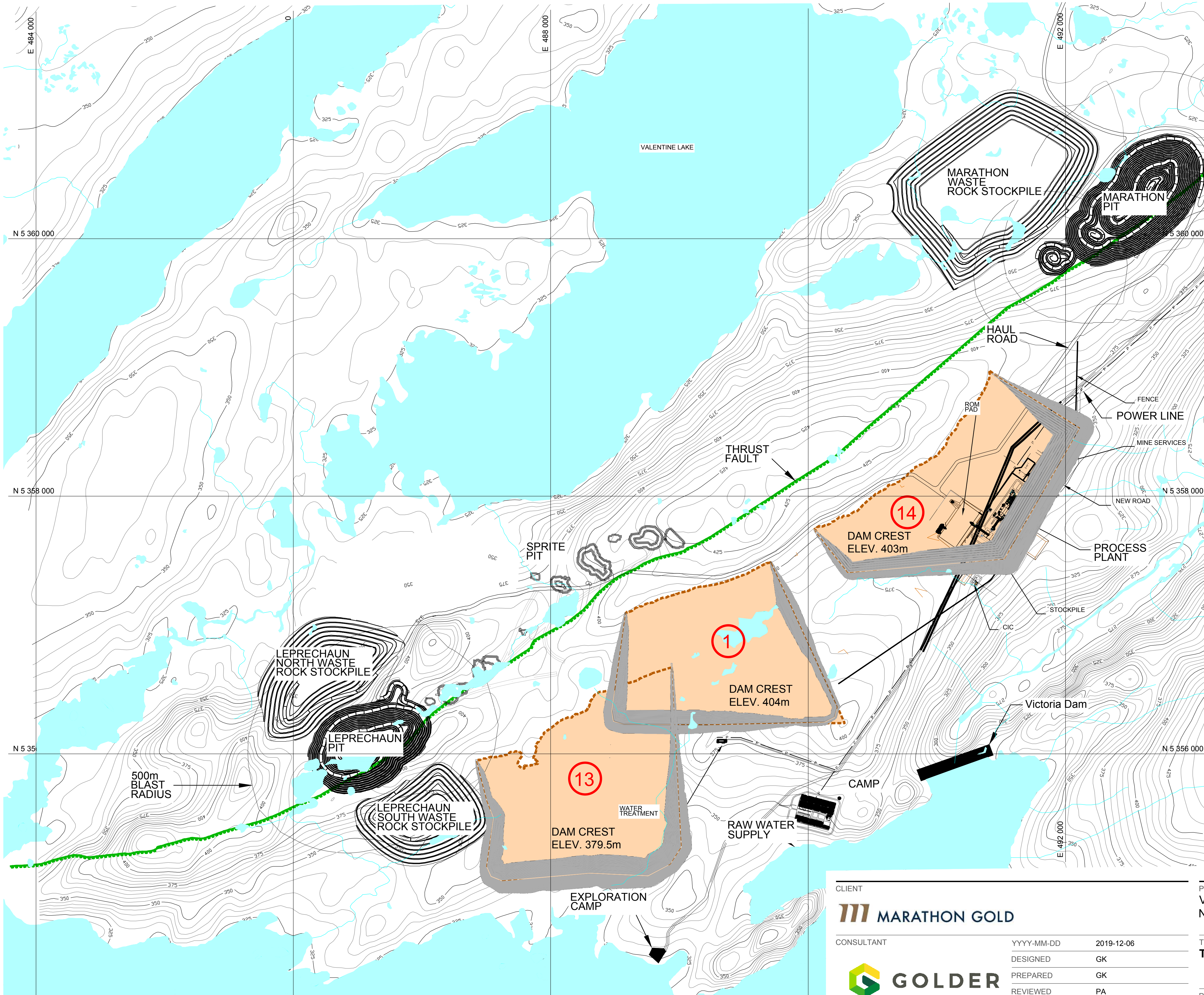
PROJECT
VALENTINE GOLD PROJECT
NEWFOUNDLAND

TITLE
TYPICAL FILTERED TAILINGS STACK SECTION

PROJECT NO.	CONTROL	REV.	FIGURE
19130660	0001	A	3



Client: **MARATHON GOLD**			**Project:** **VALENTINE GOLD PROJECT**
GOLDER	Date:	Jan-20	**Title:** **STRUCK LEVEL STAGE STORAGE RELATIONSHIPS FOR POTENTIAL TSF SITES**
	Prep'd:	MAS	
	Rev'd:	PA	Project No. 19130660 5000 Revision 0 Figure No. 4



LEGEND

- 6 TMF FOOTPRINT (APPROX.)
- PROPERTY BOUNDARY
- THRUST FAULT

NOTE(S):
1. PROJECTION: NAD83 UTM ZONE 21.

REFERENCE:
BASEPLAN PROVIDED BY CLIENT. FILENAME 5091-101-GEDGA-0003.DWG. REV.E DATED 08MAR19.

PRELIMINARY FOR DISCUSSION

0 500 1,000
METRES

CLIENT
MARATHON GOLD

CONSULTANT
GOLDER

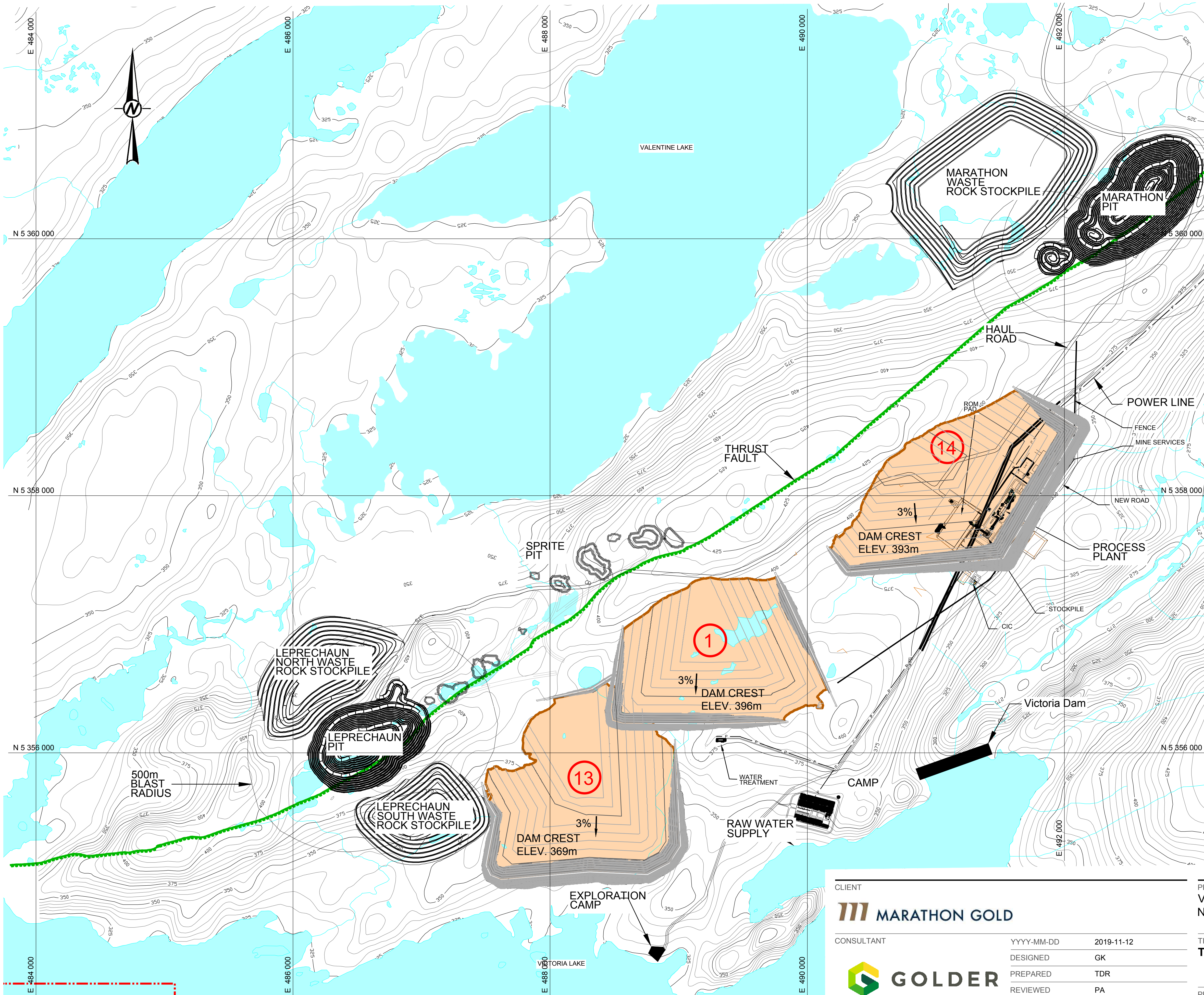
YYYY-MM-DD	2019-12-06
DESIGNED	GK
PREPARED	GK
REVIEWED	PA
APPROVED	WPM

PROJECT
VALENTINE GOLD PROJECT
NEWFOUNDLAND

TITLE
TSF CONVENTIONAL TAILINGS DISPOSAL LAYOUTS

PROJECT NO.	CONTROL	REV.	FIGURE
19130660	0001	A	5

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



LEGEND

6 TMF FOOTPRINT (APPROX.)

PROPERTY BOUNDARY

THRUST FAULT

NOTE(S):
1. PROJECTION: NAD83 UTM ZONE 21.

REFERENCE:
BASEPLAN PROVIDED BY CLIENT. FILENAME 5091-101-GEDGA-0003.DWG. REV.E DATED 08MAR19.

PRELIMINARY FOR DISCUSSION

0 500 1,000
METRES

CLIENT		PROJECT	
MARATHON GOLD		VALENTINE GOLD PROJECT NEWFOUNDLAND	
CONSULTANT	YYYY-MM-DD 2019-11-12	TITLE	
GOLDER	DESIGNED GK	TSF THICKENED TAILINGS DEPOSITION LAYOUTS	
	PREPARED TDR		
	REVIEWED PA	PROJECT NO. 19130660	CONTROL 0001
	APPROVED WPM	REV. A	FIGURE 6

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

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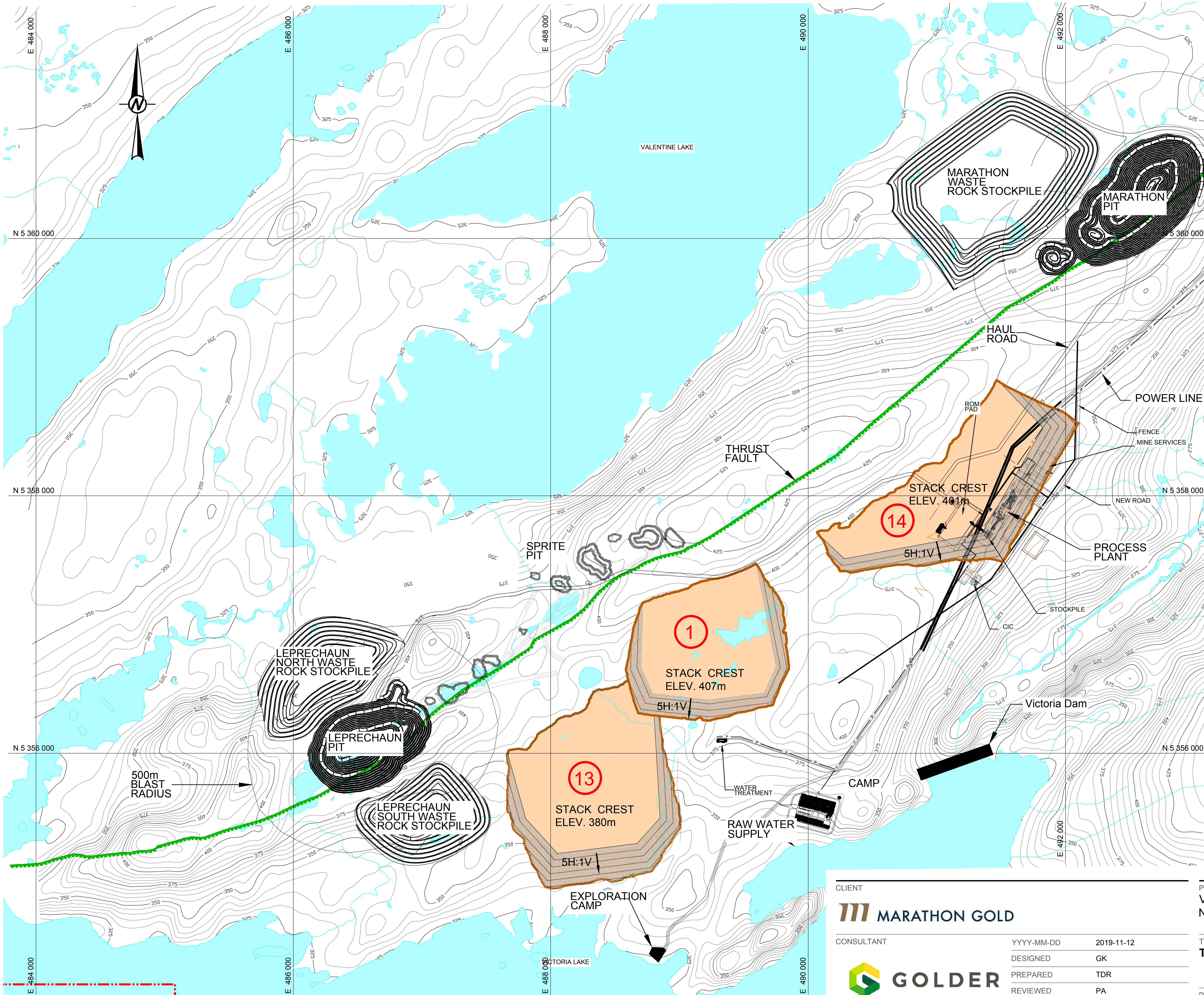


LEGEND

6 TMF FOOTPRINT (APPROX.)

PROPERTY BOUNDARY

THRUST FAULT

NOTE(S):
1. PROJECTION: NAD83 UTM ZONE 21.

REFERENCE:
BASEPLAN PROVIDED BY CLIENT. FILENAME 5091-101-GEDGA-0003.DWG. REV.E DATED 08MAR19.

PRELIMINARY FOR DISCUSSION

0 500 1,000
METRES

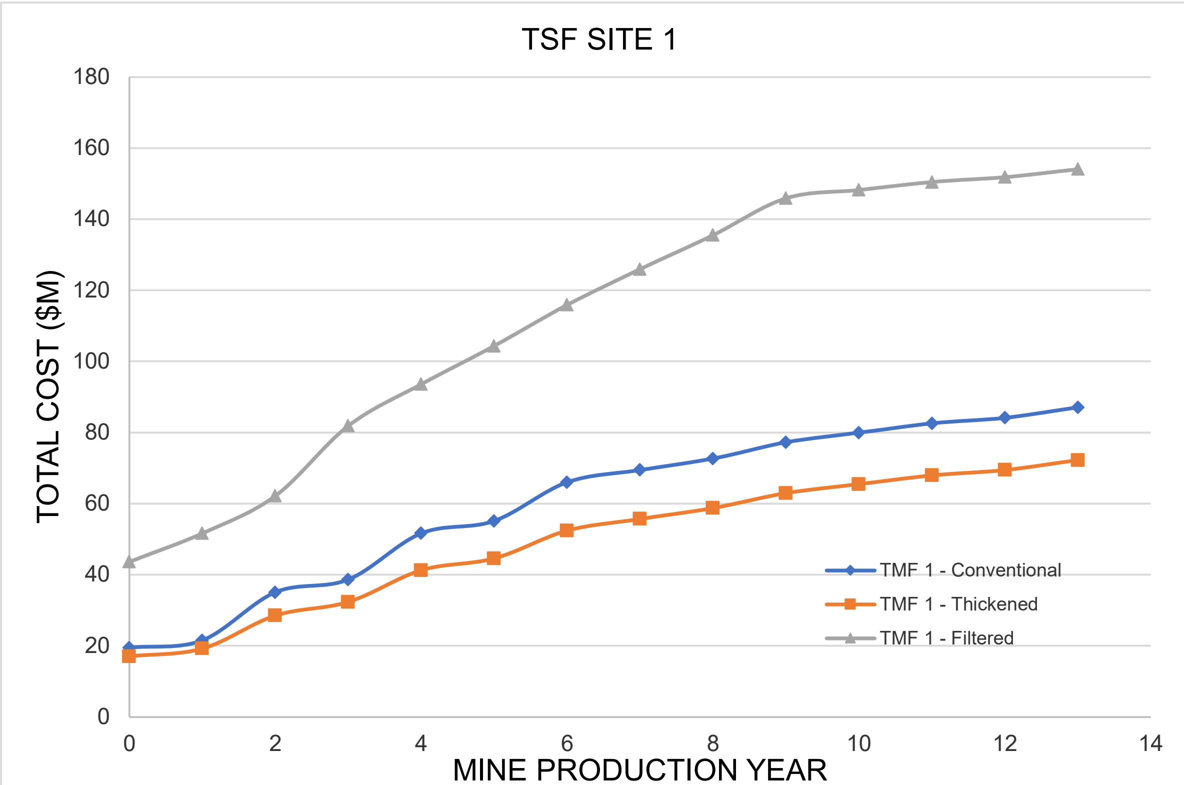
CLIENT	MARATHON GOLD	
CONSULTANT	YYYY-MM-DD	2019-11-12
GOLDER	DESIGNED	GK
	PREPARED	TDR
	REVIEWED	PA
	APPROVED	WPM

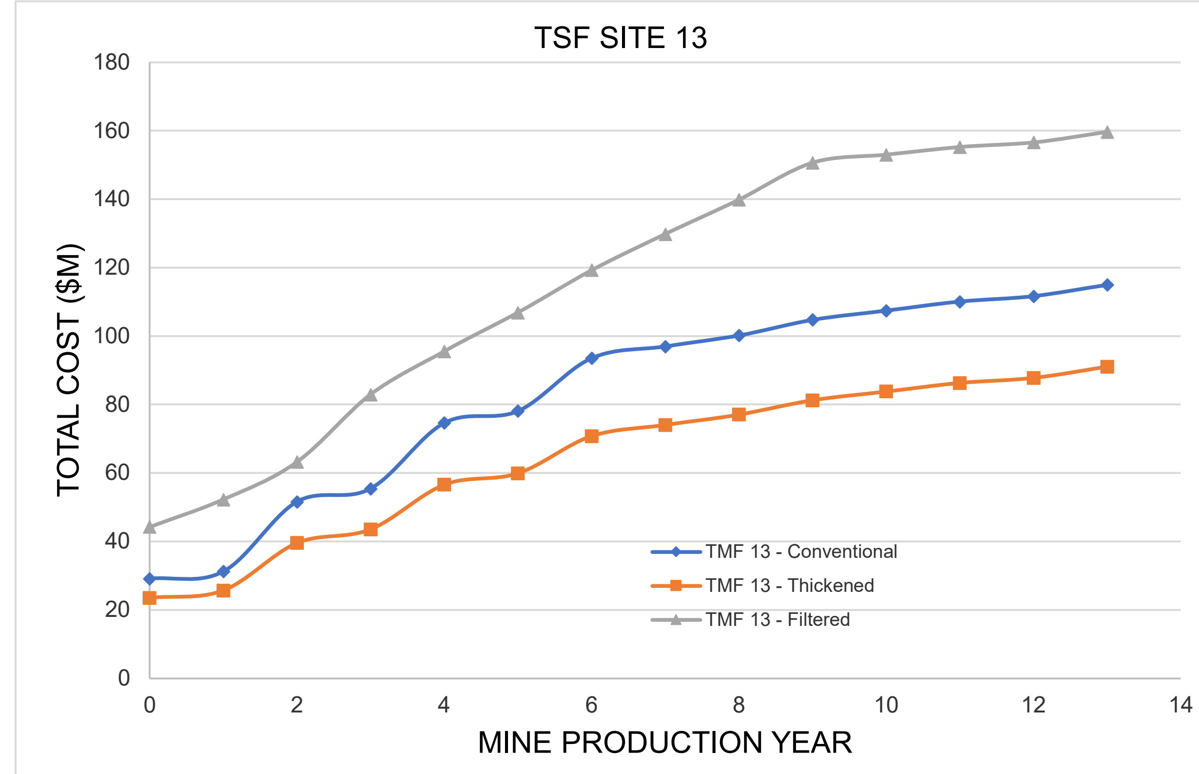
PROJECT
VALENTINE GOLD PROJECT
NEWFOUNDLAND

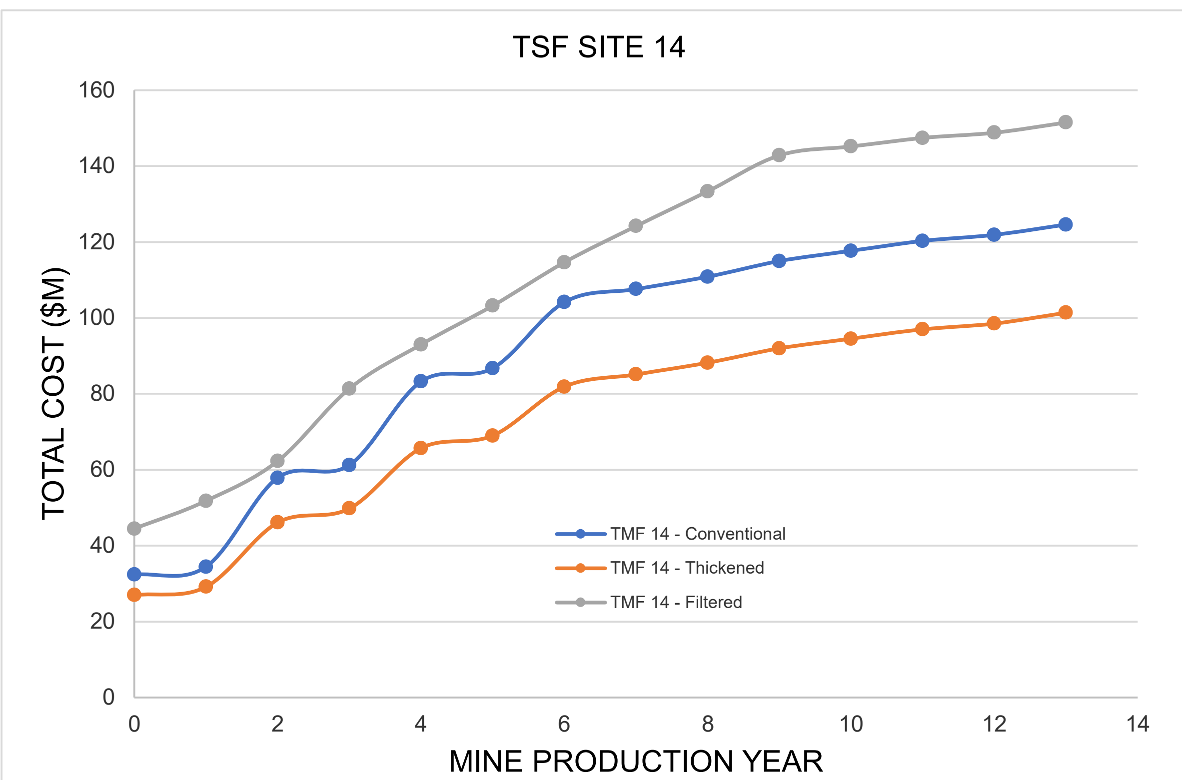
TITLE
TSF FILTERED TAILINGS "STACK" DISPOSAL LAYOUT

PROJECT NO.	CONTROL	REV.	FIGURE
19130660	0001	A	7

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







TSF SITES AND DEPOSITION OPTIONS		TOTAL CUMULATIVE COSTS ($M)
SITE 1	CONVENTIONAL	87
	THICKENED	72
	FILTERED	154
SITE 13	CONVENTIONAL	115
	THICKENED	91
	FILTERED	160
SITE 14	CONVENTIONAL	125
	THICKENED	101
	FILTERED	151

NOTE:

1. COST ESTIMATE DATA PROVIDED BY AUSENCO AND REPRESENT TOTAL CAPEX AND OPEX COSTS IN Q4 2019 CAD DISCOUNTED AT 5% ANNUALLY.

CLIENT
MARATHON GOLD

CONSULTANT
GOLDER

YYYY-MM-DD	2020-02-14
DESIGNED	MAS
PREPARED	MAS
REVIEWED	PA
APPROVED	WPM

PROJECT
VALENTINE GOLD PROJECT
NEWFOUNDLAND

TITLE
TOTAL DISCOUNTED COST ESTIMATES FOR TSF SITE AND DEPOSITION OPTIONS

TITLE	PHASE	REV	FIGURE
19130660	1000	A	8

APPENDIX A: TSF COMPARATIVE QUANTITY AND COST ESTIMATES (GOLDER)

Site 1: Conventional Slurry - Dam Crest El. 404 m

Max. Dam Height: 34 m
TSF Footprint: 180 hectares

Item	Description	Quantity	Unit	Unit Rate	Cost	Total Cost	Rate Reference
1. TSF CAPITAL COSTS							
Dam Construction						**$ 39,738,550**	
1	Dam footprint grubbing/clearing/stripping (0.5 m depth)	225,000	m³	$7.49	$1,685,250		Stantec (2)
2	Waste Rock: dump, place, compact	5,695,000	m³	$3.00	$17,085,000		
3	Coarse Filter (1m thick)	260,000	m³	$31.99	$8,317,400		
4	Fine Filter (1m thick)	260,000	m³	$30.19	$7,849,400		
5	Non-woven Geotextile (400 g/m²) supply and install	291,000	m²	$4.50	$1,309,500		Golder
6	Geomembrane (1.5 mm LLDPE White, textured) supply and install	291,000	m²	$12.00	$3,492,000		
Indirect Costs						**$ 5,960,783**	
7	Mob/Demob, Site development and servicing, Move tailings pipelines for dam raises	1	lump sum.	$5,960,782.50	$5,960,783		Golder
Closure Cost						**$ 5,497,850**	
8	Soil Cover: load, haul, place, 0.3 m thick	465,000	m³	$7.49	$3,482,850		Stantec (2)
9	Hydroseed cover area	1,550,000	m²	$1.30	$2,015,000		Golder
TOTAL COSTS						**$ 51,197,183**	

Site 13: Conventional Slurry - Dam Crest El. 379.5 m

Max. Dam Height: 55 m
TSF Footprint: 210 hectares

Item	Description	Quantity	Unit	Unit Rate	Cost	Total Cost	Rate Reference
1. TSF CAPITAL COSTS							
Dam Construction						**$ 66,069,930**	
1	Dam footprint grubbing/clearing/stripping (0.5 m depth)	307,000	m³	$7.49	$2,299,430		Stantec (2)
2	Waste Rock: dump, place, compact	11,907,000	m³	$3.00	$35,721,000		
3	Coarse Filter (1m thick)	350,000	m³	$31.99	$11,196,500		
4	Fine Filter (1m thick)	350,000	m³	$30.19	$10,566,500		
5	Non-woven Geotextile (400 g/m²) supply and install	381,000	m²	$4.50	$1,714,500		Golder
6	Geomembrane (1.5 mm LLDPE White, textured) supply and install	381,000	m²	$12.00	$4,572,000		
Indirect Costs						**$ 9,910,490**	
7	Mob/Demob, Site development and servicing, Move tailings pipelines for dam raises	1	lump sum.	$9,910,489.50	$9,910,490		Golder
Closure Cost						**$ 6,242,720**	
8	Soil Cover: load, haul, place, 0.3 m thick	528,000	m³	$7.49	$3,954,720		Stantec (2)
9	Hydroseed cover area	1,760,000	m²	$1.30	$2,288,000		Golder
TOTAL COSTS						**$ 82,223,140**	

Site 14: Conventional Slurry - Dam Crest El. 403 m

Max. Dam Height: 62 m
TSF Footprint: 180 hectares

Item	Description	Quantity	Unit	Unit Rate	Cost	Total Cost	Rate Reference
1. TSF CAPITAL COSTS							
Dam Construction						**$ 77,831,900**	
1	Dam footprint grubbing/clearing (0.5 m depth)	350,000	m³	$7.49	$2,621,500		Stantec (2)
2	Waste Rock: dump, place, compact	14,950,000	m³	$3.00	$44,850,000		
3	Coarse Filter (1m thick)	380,000	m³	$31.99	$12,156,200		
4	Fine Filter (1m thick)	380,000	m³	$30.19	$11,472,200		
5	Non-woven Geotextile (400 g/m²) supply and install	408,000	m²	$4.50	$1,836,000		Golder
6	Geomembrane (1.5 mm LLDPE White, textured) supply and install	408,000	m²	$12.00	$4,896,000		
Indirect Costs						**$ 11,674,785**	
7	Mob/Demob, Site development and servicing, Move tailings pipelines for dam raises	1	lump sum.	$11,674,785.00	$11,674,785		Golder
Closure Cost						**$ 5,107,680**	
8	Soil Cover: load, haul, place, 0.3 m thick	432,000	m³	$7.49	$3,235,680		Stantec (2)
9	Hydroseed cover area	1,440,000	m²	$1.30	$1,872,000		Golder
TOTAL COSTS						**$ 94,614,365**	

Notes:
1. Estimates are provisional and for comparative purposes only.
2. Stantec rates are from the memo submitted to Marathon Gold Corp, "Valentine Lake PEA - TSF Stage 1 Configuration and Quantities. Doc No. MEM-002-300-D-2April18"
3. Golder rates are from similar projects in Canada
4. Dam volumes are based on a 10 m wide crest, 1(V): 3.5(H) upstream slope and 1(V): 2(H) downtream slope
5. Clearing and grubbing of the tailings basin has not been included in the estimate.
6. Basin lining has been excluded. Areas estimated for the geotextile and geomembrane are based on lining the upstream slope of the dam only.
7. It is assumed that no foundation bedrock grouting (or other seepage cut-off measure) is required.

Site 1: Thickened Tailings - Dam Crest El. 396 m

Max. Dam Height: 25 m
TSF Footprint: 160 hectares

Item	Description	Quantity	Unit	Unit Rate	Cost	Total Cost	Rate Reference
1. TSF CAPITAL COSTS							
Dam Construction						**$ 23,199,090**	
1	Dam footprint grubbing/clearing/stripping (0.5 m depth)	137,000	m³	$7.49	$1,026,130		Stantec (2)
2	Waste Rock: dump, place, compact	2,737,000	m³	$3.00	$8,211,000		
3	Coarse Filter (1m thick)	172,000	m³	$31.99	$5,502,280		
4	Fine Filter (1m thick)	172,000	m³	$30.19	$5,192,680		
5	Non-woven Geotextile (400 g/m²) supply and install	198,000	m²	$4.50	$891,000		Golder
6	Geomembrane (1.5 mm LLDPE White, textured) supply and install	198,000	m²	$12.00	$2,376,000		
Indirect Costs						**$ 3,479,864**	
7	Mob/Demob, Site development and servicing, Move tailings pipelines for dam raises	1	lump sum.	$3,479,863.50	$3,479,864		Golder
Closure Cost						**$ 5,214,090**	
8	Soil Cover: load, haul, place, 0.3 m thick	441,000	m³	$7.49	$3,303,090		Stantec (2)
9	Hydroseed cover area	1,470,000	m²	$1.30	$1,911,000		Golder
TOTAL COSTS						**$ 31,893,044**	

Site 13: Thickened Tailings - Dam Crest El. 369 m

Max. Dam Height: 44 m
TSF Footprint: 190 hectares

Item	Description	Quantity	Unit	Unit Rate	Cost	Total Cost	Rate Reference
1. TSF CAPITAL COSTS							
Dam Construction						**$ 40,687,860**	
1	Dam footprint grubbing/clearing/stripping (0.5 m depth)	206,000	m³	$7.49	$1,542,940		Stantec (2)
2	Waste rock: dump, place, compact	6,506,000	m³	$3.00	$19,518,000		
3	Coarse Filter (1m thick)	244,000	m³	$31.99	$7,805,560		
4	Fine Filter (1m thick)	244,000	m³	$30.19	$7,366,360		
5	Non-woven Geotextile (400 g/m²) supply and install	270,000	m²	$4.50	$1,215,000		Golder
6	Geomembrane (1.5 mm LLDPE White, textured) supply and install	270,000	m²	$12.00	$3,240,000		
Indirect Costs						**$ 6,103,179**	
7	Mob/Demob, Site development and servicing, Move tailings pipelines for dam raises	1	lump sum.	$6,103,179.00	$6,103,179		Golder
Closure Cost						**$ 6,164,686**	
8	Soil Cover: load, haul, place, 0.3 m thick	521,400	m³	$7.49	$3,905,286		Stantec (2)
9	Hydroseed cover area	1,738,000	m²	$1.30	$2,259,400		Golder
TOTAL COSTS						**$ 52,955,725**	

Site 14: Thickened Tailings - Dam Crest El. 393 m

Max. Dam Height: 51 m
TSF Footprint: 160 hectares

Item	Description	Quantity	Unit	Unit Rate	Cost	Total Cost	Rate Reference
1. TSF CAPITAL COSTS							
Dam Construction						**$ 52,493,230**	
1	Dam footprint grubbing/clearing (0.5 m depth)	255,000	m³	$7.49	$1,909,950		Stantec (2)
2	Waste Rock: dump, place, compact	8,955,000	m³	$3.00	$26,865,000		
3	Coarse Filter (1m thick)	296,000	m³	$31.99	$9,469,040		
4	Fine Filter (1m thick)	296,000	m³	$30.19	$8,936,240		
5	Non-woven Geotextile (400 g/m²) supply and install	322,000	m²	$4.50	$1,449,000		Golder
6	Geomembrane (1.5 mm LLDPE White, textured) supply and install	322,000	m²	$12.00	$3,864,000		
Indirect Costs						**$ 7,873,985**	
7	Mob/Demob, Site development and servicing, Move tailings pipelines for dam raises	1	lump sum.	$7,873,984.50	$7,873,985		Golder
Closure Cost						**$ 5,426,910**	
8	Soil Cover: load, haul, place, 0.3 m thick	459,000	m³	$7.49	$3,437,910		Stantec (2)
9	Hydroseed cover area	1,530,000	m²	$1.30	$1,989,000		Golder
TOTAL COSTS						**$ 65,794,125**	

Notes:
1. Estimates are provisional and for comparative purposes only.
2. Stantec rates are from the memo submitted to Marathon Gold Corp, "Valentine Lake PEA - TSF Stage 1 Configuration and Quantities. Doc No. MEM-002-300-D-2April18"
3. Golder rates are from similar projects in Canada
4. Dam volumes are based on a 10 m wide crest, 1(V): 3.5(H) upstream slope and 1(V): 2(H) downtream slope
5. Clearing and grubbing of the tailings basin has not been included in the estimate.
6. Basin lining has been excluded. Areas estimated for the geotextile and geomembrane are based on lining the upstream slope of the dam only.
7. It is assumed that no foundation bedrock grouting (or other seepage cut-off measure) is required.
8. Indirect costs assumed to be 15% of the direct costs.

Site 1: Filtered Tailings - Crest El. 407 m

Max. TSF Height: 34 m
TSF Footprint: 118 hectares

Item	Description	Quantity	Unit	Unit Rate	Cost	Total Cost	Rate Reference
1. TSF CAPITAL COSTS							
Dam Construction						**$ 7,280,630**	
1	Toe berm footprint grubbing/clearing/stripping (0.5 m depth)	29,000	m^3	$7.49	$217,210		Stantec (2,3)
2	Waste rock for toe berm and erosion protection : Load, haul, dump, spread, compact	446,000	m^3	$10.40	$4,638,400		
3	Coarse Filter (1m thick)	39,000	m^3	$31.99	$1,247,610		
4	Fine Filter (1m thick)	39,000	m^3	$30.19	$1,177,410		
5	Non-woven Geotextile (400 g/m^2) supply and install	0	m^2	$4.50	$0		Golder
6	Geomembrane (1.5 mm LLDPE White, textured) supply and install	0	m^2	$12.00	$0		
Indirect Costs						**$ 1,092,095**	
7	Mob/Demob, Site development and servicing, Move tailings pipelines for dam raises	1	lump sum.	$1,092,094.50	$1,092,095		Golder
Closure Cost						**$ 4,192,554**	
8	Soil Cover: load, haul, place, 0.3 m thick	354,600	m^3	$7.49	$2,655,954		Stantec (2)
9	Hydroseed cover area	1,182,000	m^2	$1.30	$1,536,600		Golder
TOTAL COSTS						**$ 12,565,279**	

Site 13: Filtered Tailings - Crest El. 380 m

Max. TSF Height: 54 m
TSF Footprint: 159 hectares

Item	Description	Quantity	Unit	Unit Rate	Cost	Total Cost	Rate Reference
1. TSF CAPITAL COSTS							
Dam Construction						**$ 8,973,270**	
1	Toe berm footprint grubbing/clearing/stripping (0.5 m depth)	33,000	m^3	$7.49	$247,170		Stantec (2,3)
2	Waste rock for toe berm and erosion protection : Load, haul, dump, spread, compact	570,000	m^3	$10.40	$5,928,000		
3	Coarse Filter (1m thick)	45,000	m^3	$31.99	$1,439,550		
4	Fine Filter (1m thick)	45,000	m^3	$30.19	$1,358,550		
5	Non-woven Geotextile (400 g/m^2) supply and install	0	m^2	$4.50	$0		Golder
6	Geomembrane (1.5 mm LLDPE White, textured) supply and install	0	m^2	$12.00	$0		
Indirect Costs						**$ 1,345,991**	
7	Mob/Demob, Site development and servicing, Move tailings pipelines for dam raises	1	lump sum.	$1,345,990.50	$1,345,991		Golder
Closure Cost						**$ 5,675,200**	
8	Soil Cover: load, haul, place, 0.3 m thick	480,000	m^3	$7.49	$3,595,200		Stantec (2)
9	Hydroseed cover area	1,600,000	m^2	$1.30	$2,080,000		Golder
TOTAL COSTS						**$ 15,994,461**	

Site 14: Filtered Tailings - Crest El. 401 m

Max. TSF Height: 53 m
TSF Footprint: 142 hectares

Item	Description	Quantity	Unit	Unit Rate	Cost	Total Cost	Rate Reference
1. TSF CAPITAL COSTS							
Dam Construction						**$ 9,517,200**	
1	Toe berm footprint grubbing/clearing (0.5 m depth)	32,000	m^3	$7.49	$239,680		Stantec (2,3)
2	Waste rock for toe berm and erosion protection : Load, haul, dump, spread, compact	629,000	m^3	$10.40	$6,541,600		
3	Coarse Filter (1m thick)	44,000	m^3	$31.99	$1,407,560		
4	Fine Filter (1m thick)	44,000	m^3	$30.19	$1,328,360		
5	Non-woven Geotextile (400 g/m^2) supply and install	0	m^2	$4.50	$0		Golder
6	Geomembrane (1.5 mm LLDPE White, textured) supply and install	0	m^2	$12.00	$0		
Indirect Costs						**$ 1,427,580**	
7	Mob/Demob, Site development and servicing, Move tailings pipelines for dam raises	1	lump sum.	$1,427,580.00	$1,427,580		Golder
Closure Cost						**$ 5,079,304**	
8	Soil Cover: load, haul, place, 0.3 m thick	429,600	m^3	$7.49	$3,217,704		Stantec (2)
9	Hydroseed cover area	1,432,000	m^2	$1.30	$1,861,600		Golder
TOTAL COSTS						**$ 16,024,084**	

Notes:
1. Estimates are provisional and for comparative purposes only.
2. Stantec rates are from the memo submitted to Marathon Gold Corp, "Valentine Lake PEA - TSF Stage 1 Configuration and Quantities. Doc No. MEM-002-300-D-2April18"
3. It is assumed that the waste rock for the toe berm and erosion protection will not be placed as part of the mining operation. Therefore a rate of $10.40 is used
4. Golder rates are from similar projects in Canada
5. Clearing and grubbing of the tailings basin has not been included in the estimate.
6. It is assumed that no foundation bedrock grouting (or other seepage cut-off measure) is required.
7. Indirect costs assumed to be 15% of the direct costs.

Yearly Operating and Engineering Costs - Conventional and Thickened

		Yr 0	Yr 1	Yr 2	Yr 3	Yr 4	Yr 5	Yr 6	Yr 7	Yr 8	Yr 9	Yr 10	Yr 11	Yr 12	Yr 13	Yr 14	Yr 15	TOTAL
Contractors/Consultants																		
Full Time Construction CQA		$700,000	-	$700,000	-	$700,000	-	$700,000	-	-	-	-	-	-	-	-	-	$2,800,000
Engineering - Raise Design		$250,000	-	$250,000	-	$250,000	-	$250,000	-	-	-	-	-	-	-	-	-	$1,000,000
InPit Disposal Studies + Design		-	-	-	-	-	-	-	$450,000	-	-	-	-	-	-	-	-	$450,000
InPit Disposal CQA		-	-	-	-	-	-	-	-	$350,000	-	-	-	-	-	-	-	$350,000
Dam Safety Review		-	$100,000	-	-	-	-	$100,000	-	-	-	-	$100,000	-	-	-	-	$300,000
Dam Safety Inspection		-	$30,000	$30,000	$30,000	$30,000	$30,000	$30,000	$30,000	$30,000	$30,000	$30,000	$30,000	$30,000	$30,000	$30,000	$30,000	$390,000
OMS Manual udpate		-	$5,000	$5,000	$5,000	$5,000	$5,000	$5,000	$5,000	$30,000	$5,000	$5,000	$5,000	$5,000	$5,000	$5,000	$5,000	$90,000
Vegetation control		-	-	$10,000	-	$10,000	-	$10,000	-	$10,000	-	$10,000	-	$10,000	-	$10,000	-	$60,000
TMF Operations																		$0
Tailings Engineer / Responsible		-	$200,000	$200,000	$200,000	$200,000	$200,000	$200,000	$200,000	$200,000	$200,000	$200,000	$200,000	$200,000	$200,000	$200,000	$200,000	$2,600,000
TMF Technician		-	$60,000	$60,000	$60,000	$60,000	$60,000	$60,000	$60,000	$60,000	$60,000	$60,000	$60,000	$60,000	$60,000	$60,000	$60,000	$780,000
Light Vehicle (purchase, maintenance, fuel, insurance)		-	$110,000	$50,000	$50,000	$50,000	$110,000	$50,000	$50,000	$50,000	$50,000	$50,000	$50,000	$50,000	$50,000	$50,000	$50,000	$770,000
Water Quality Monitoring		-	$140,000	$140,000	$140,000	$140,000	$140,000	$140,000	$140,000	$140,000	$140,000	$140,000	$140,000	$140,000	$140,000	$140,000	$140,000	$1,820,000
Camp/Lodging		-	$80,000	$80,000	$80,000	$80,000	$80,000	$80,000	$80,000	$80,000	$80,000	$80,000	$80,000	$80,000	$80,000	$80,000	$80,000	$1,040,000
	CAD	$950,000	$725,000	$1,525,000	$565,000	$1,525,000	$625,000	$1,625,000	$1,015,000	$950,000	$565,000	$575,000	$665,000	$575,000	$565,000	$575,000	$565,000	$12,450,000
(if exchange rate is 1 cost is in	CAD	$950,000	$725,000	$1,525,000	$565,000	$1,525,000	$625,000	$1,625,000	$1,015,000	$950,000	$565,000	$575,000	$665,000	$575,000	$565,000	$575,000	$565,000	$12,450,000

1USD = 1 CAD

Yearly Operating and Engineering Costs - Filtered Tailings

		Yr 0	Yr 1	Yr 2	Yr 3	Yr 4	Yr 5	Yr 6	Yr 7	Yr 8	Yr 9	Yr 10	Yr 11	Yr 12	Yr 13	Yr 14	Yr 15	TOTAL
Contractors/Consultants																		
Full/Part-time Construction CQA		$700,000	-	$60,000	-	$60,000	-	$60,000	-	-	-	-	-	-	-	-	-	$880,000
Engineering		$250,000	-	$50,000	-	$50,000	-	$50,000	-	-	-	-	-	-	-	-	-	$400,000
InPit Disposal Studies + Design		-	-	-	-	-	-	-	$450,000	-	-	-	-	-	-	-	-	$450,000
InPit Disposal CQA		-	-	-	-	-	-	-	-	$350,000		-	-	-	-	-	-	$350,000
Dam Safety Review		-	$100,000	-	-	-	-	-	-	$100,000	-	-	-	-	-	-	-	$200,000
Dam Safety Inspection		-	$30,000	$30,000	$30,000	$30,000	$30,000	$30,000	$30,000	$30,000	$30,000	$30,000	$30,000	$30,000	$30,000	$30,000	$30,000	$390,000
OMS Manual udpate		-	$5,000	$5,000	$5,000	$5,000	$5,000	$5,000	$5,000	$30,000	$5,000	$5,000	$5,000	$5,000	$5,000	$5,000	$5,000	$90,000
Vegetation control		-	-	$10,000	-	$10,000	-	$10,000	-	$10,000	-	$10,000	-	$10,000	-	$10,000	-	$60,000
TMF Operations																		
Tailings Engineer / Responsible		-	$200,000	$200,000	$200,000	$200,000	$200,000	$200,000	$200,000	$200,000	$200,000	$200,000	$200,000	$200,000	$200,000	$200,000	$200,000	$2,600,000
TMF Technician		-	$60,000	$60,000	$60,000	$60,000	$60,000	$60,000	$60,000	$60,000	$60,000	$60,000	$60,000	$60,000	$60,000	$60,000	$60,000	$780,000
Light Vehicle (purchase, maintenance, fuel, insurance)		-	$110,000	$50,000	$50,000	$50,000	$110,000	$50,000	$50,000	$50,000	$50,000	$50,000	$50,000	$50,000	$50,000	$50,000	$50,000	$770,000
Water Quality Monitoring		-	$140,000	$140,000	$140,000	$140,000	$140,000	$140,000	$140,000	$140,000	$140,000	$140,000	$140,000	$140,000	$140,000	$140,000	$140,000	$1,820,000
Camp/Lodging		-	$80,000	$80,000	$80,000	$80,000	$80,000	$80,000	$80,000	$80,000	$80,000	$80,000	$80,000	$80,000	$80,000	$80,000	$80,000	$1,040,000
	CAD	$950,000	$725,000	$685,000	$565,000	$685,000	$625,000	$685,000	$1,015,000	$1,050,000	$565,000	$575,000	$565,000	$575,000	$565,000	$575,000	$565,000	$9,830,000
(if exchange rate is 1 cost is in	CAD	$950,000	$725,000	$685,000	$565,000	$685,000	$625,000	$685,000	$1,015,000	$1,050,000	$565,000	$575,000	$565,000	$575,000	$565,000	$575,000	$565,000	$9,830,000

1USD = 1 CAD

TMF - Site 1 Cost Estimate - Yearly Breakdown

Conventional Tailings	Unit	Year																Closure	Total
		0	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15		
Direct Capital Cost	$M [CAD]	11.9	-	9.9	-	9.9	-	7.9	-	-	-	-	-	-	-	-	-	5.5	45.2
Indirect Capitial Cost	$M [CAD]	1.8	-	1.5	-	1.5	-	1.2	-	-	-	-	-	-	-	-	-	-	6.0
Operating and Engineering Costs	$M [CAD]	1.0	0.7	1.5	0.6	1.5	0.6	1.6	1.0	1.0	0.6	0.6	0.7	0.6	0.6	0.6	0.6	-	13.6
Total	**$M [CAD]**	**14.7**	**0.7**	**12.9**	**0.6**	**12.9**	**0.6**	**10.8**	**1.0**	**1.0**	**0.6**	**0.6**	**0.7**	**0.6**	**0.6**	**0.6**	**0.6**	**5.5**	**64.8**

Thickened Tailings	Unit	Year																Closure	Total
		0	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15		
Direct Capital Cost	$M [CAD]	7.0	-	5.8	-	5.8	-	4.6	-	-	-	-	-	-	-	-	-	5.2	28.4
Indirect Capitial Cost	$M [CAD]	1.0	-	0.9	-	0.9	-	0.7	-	-	-	-	-	-	-	-	-	-	3.5
Operating and Engineering Costs	$M [CAD]	1.0	0.7	1.5	0.6	1.5	0.6	1.6	1.0	1.0	0.6	0.6	0.7	0.6	0.6	0.6	0.6	-	13.6
Total	**$M [CAD]**	**9.0**	**0.7**	**8.2**	**0.6**	**8.2**	**0.6**	**7.0**	**1.0**	**1.0**	**0.6**	**0.6**	**0.7**	**0.6**	**0.6**	**0.6**	**0.6**	**5.2**	**45.5**

Filtered Tailings	Unit	Year																Closure	Total
		0	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15		
Direct Capital Cost	$M [CAD]	2.2	-	1.8	-	1.8	-	1.5	-	-	-	-	-	-	-	-	-	4.2	11.5
Indirect Capitial Cost	$M [CAD]	0.3	-	0.3	-	0.3	-	0.2	-	-	-	-	-	-	-	-	-	-	1.1
Operating and Engineering Costs	$M [CAD]	1.0	0.7	0.7	0.6	0.7	0.6	0.7	1.0	1.1	0.6	0.6	0.6	0.6	0.6	0.6	0.6	-	11.0
Total	**$M [CAD]**	**3.5**	**0.7**	**2.8**	**0.6**	**2.8**	**0.6**	**2.4**	**1.0**	**1.1**	**0.6**	**0.6**	**0.6**	**0.6**	**0.6**	**0.6**	**0.6**	**4.2**	**23.5**

Notes:

1. Total capital costs distributed over 4 stages of dam raising based on the assumed breakdown below.

Stage	Year	Capital Cost Percentage
1	0	30%
2	2	25%
3	4	25%
4	6	20%

TMF - Site 13 Cost Estimate - Yearly Breakdown

Conventional Tailings	Unit	Year																Closure	Total
		0	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15		
Direct Capital Cost	$M [CAD]	19.8	-	16.5	-	16.5	-	13.2	-	-	-	-	-	-	-	-	-	6.2	72.3
Indirect Capitial Cost	$M [CAD]	3.0	-	2.5	-	2.5	-	2.0	-	-	-	-	-	-	-	-	-	-	9.9
Operating and Engineering Costs	$M [CAD]	1.0	0.7	1.5	0.6	1.5	0.6	1.6	1.0	1.0	0.6	0.6	0.7	0.6	0.6	0.6	0.6	-	13.6
Total	**$M [CAD]**	**23.7**	**0.7**	**20.5**	**0.6**	**20.5**	**0.6**	**16.8**	**1.0**	**1.0**	**0.6**	**0.6**	**0.7**	**0.6**	**0.6**	**0.6**	**0.6**	**6.2**	**95.8**

Thickened Tailings	Unit	Year																Closure	Total
		0	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15		
Direct Capital Cost	$M [CAD]	12.2	-	10.2	-	10.2	-	8.1	-	-	-	-	-	-	-	-	-	6.2	46.9
Indirect Capitial Cost	$M [CAD]	1.8	-	1.5	-	1.5	-	1.2	-	-	-	-	-	-	-	-	-	-	6.1
Operating and Engineering Costs	$M [CAD]	1.0	0.7	1.5	0.6	1.5	0.6	1.6	1.0	1.0	0.6	0.6	0.7	0.6	0.6	0.6	0.6	-	13.6
Total	**$M [CAD]**	**15.0**	**0.7**	**13.2**	**0.6**	**13.2**	**0.6**	**11.0**	**1.0**	**1.0**	**0.6**	**0.6**	**0.7**	**0.6**	**0.6**	**0.6**	**0.6**	**6.2**	**66.5**

Filtered Tailings	Unit	Year																Closure	Total
		0	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15		
Direct Capital Cost	$M [CAD]	2.7	-	2.2	-	2.2	-	1.8	-	-	-	-	-	-	-	-	-	5.7	14.6
Indirect Capitial Cost	$M [CAD]	0.4	-	0.3	-	0.3	-	0.3	-	-	-	-	-	-	-	-	-	-	1.3
Operating and Engineering Costs	$M [CAD]	1.0	0.7	0.7	0.6	0.7	0.6	0.7	1.0	1.1	0.6	0.6	0.6	0.6	0.6	0.6	0.6	-	11.0
Total	**$M [CAD]**	**4.0**	**0.7**	**3.3**	**0.6**	**3.3**	**0.6**	**2.7**	**1.0**	**1.1**	**0.6**	**0.6**	**0.6**	**0.6**	**0.6**	**0.6**	**0.6**	**5.7**	**27.0**

Notes:

1. Total capital costs distributed over 4 stages of dam raising based on the assumed breakdown below.

Stage	Year	Capital Cost Percentage
1	0	30%
2	2	25%
3	4	25%
4	6	20%

TMF - Site 14 Cost Estimate - Yearly Breakdown

Conventional Tailings	Unit	Year 0	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	Closure	Total
Direct Capital Cost	$M [CAD]	23.3	-	19.5	-	19.5	-	15.6	-	-	-	-	-	-	-	-	-	5.1	82.9
Indirect Capitial Cost	$M [CAD]	3.5	-	2.9	-	2.9	-	2.3	-	-	-	-	-	-	-	-	-	-	11.7
Operating and Engineering Costs	$M [CAD]	1.0	0.7	1.5	0.6	1.5	0.6	1.6	1.0	1.0	0.6	0.6	0.7	0.6	0.6	0.6	0.6	-	13.6
Total	**$M [CAD]**	**27.8**	**0.7**	**23.9**	**0.6**	**23.9**	**0.6**	**19.5**	**1.0**	**1.0**	**0.6**	**0.6**	**0.7**	**0.6**	**0.6**	**0.6**	**0.6**	**5.1**	**108.2**

Thickened Tailings	Unit	Year 0	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	Closure	Total
Direct Capital Cost	$M [CAD]	15.7	-	13.1	-	13.1	-	10.5	-	-	-	-	-	-	-	-	-	5.4	57.9
Indirect Capital Cost	$M [CAD]	2.4	-	2.0	-	2.0	-	1.6	-	-	-	-	-	-	-	-	-	-	7.9
Operating and Engineering Costs	$M [CAD]	1.0	0.7	1.5	0.6	1.5	0.6	1.6	1.0	1.0	0.6	0.6	0.7	0.6	0.6	0.6	0.6	-	13.6
Total	**$M [CAD]**	**19.1**	**0.7**	**16.6**	**0.6**	**16.6**	**0.6**	**13.7**	**1.0**	**1.0**	**0.6**	**0.6**	**0.7**	**0.6**	**0.6**	**0.6**	**0.6**	**5.4**	**79.4**

Filtered Tailings	Unit	Year 0	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	Closure	Total
Direct Capital Cost	$M [CAD]	2.9	-	2.4	-	2.4	-	1.9	-	-	-	-	-	-	-	-	-	5.1	14.6
Indirect Capitial Cost	$M [CAD]	0.4	-	0.4	-	0.4	-	0.3	-	-	-	-	-	-	-	-	-	-	1.4
Operating and Engineering Costs	$M [CAD]	1.0	0.7	0.7	0.6	0.7	0.6	0.7	1.0	1.1	0.6	0.6	0.6	0.6	0.6	0.6	0.6	-	11.0
Total	**$M [CAD]**	**4.2**	**0.7**	**3.4**	**0.6**	**3.4**	**0.6**	**2.9**	**1.0**	**1.1**	**0.6**	**0.6**	**0.6**	**0.6**	**0.6**	**0.6**	**0.6**	**5.1**	**27.0**

Notes:
1. Total capital costs distributed over 4 stages of dam raising based on the assumed breakdown below.

Stage	Year	Capital Cost Percentage
1	0	30%
2	2	25%
3	4	25%
4	6	20%

APPENDIX B: OVERALL COST ESTIMATES (AUSENCO)

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Financials	Value	
Discount Rate [%]	5%	Assumption
Inflation [%]	0%	Assumption

TMF OPTION 1 ANALYSIS

Conventional Tailings	Unit	0	1	2	3	4	5	6	7	8	9	10	11	12	13
Mine Feed Plar	Mt/y	0.0	1.5	2.0	2.0	3.0	4.0	4.0	4.0	4.0	4.0	4.0	4.0	2.4	0.0
Processing Area Capital Costs	$M [CAD]	4.8	0.0	0.0	1.6	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
Tailings Area Capital Costs	$M [CAD]	14.7	0.0	11.4	0.0	11.4	0.0	9.1	0.0	0.0	2.7	0.0	0.0	0.0	5.5
Processing Area Operating Costs	$/t mill feed	0.0	0.1	0.1	0.1	0.1	0.1	0.1	0.1	0.1	0.1	0.1	0.1	0.1	0.0
Tailings Area Operating Costs	$/t mill feed	0.0	1.4	1.6	1.2	1.4	1.0	1.3	1.1	1.1	1.0	1.0	1.0	1.1	0.0
Tailings Area Operating Costs	$M/y [CAD]	0.0	2.0	3.3	2.3	4.2	4.1	5.1	4.5	4.5	4.1	4.1	4.2	2.7	0.0
Total Operating Costs	$M/y [CAD]	0.0	2.2	3.5	2.5	4.4	4.4	5.4	4.8	4.8	4.4	4.4	4.5	2.8	0.0
Annual Cash Flow (Negative)	$M/y [CAD]	19.5	2.2	14.9	4.1	15.8	4.4	14.6	4.8	4.8	7.1	4.4	4.5	2.8	5.5
Discounted Cash Flow (Negative	$M/y [CAD]	19.5	2.1	13.5	3.6	13.0	3.5	10.9	3.4	3.2	4.6	2.7	2.6	1.6	2.9
Cumulative Net Present Costs	**$M [CAD]**	**19**	**22**	**35**	**39**	**52**	**55**	**66**	**69**	**73**	**77**	**80**	**83**	**84**	**87**

Thickened Tailings	Unit	0	1	2	3	4	5	6	7	8	9	10	11	12	13
Mine Feed Plar	Mt/y	0.0	1.5	2.0	2.0	3.0	4.0	4.0	4.0	4.0	4.0	4.0	4.0	2.4	0.0
Processing Area Capital Costs	$M [CAD]	8.1	0.0	0.0	1.8	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
Tailings Area Capital Costs	$M [CAD]	9.0	0.0	6.7	0.0	6.7	0.0	5.3	0.0	0.0	2.3	0.0	0.0	0.0	5.2
Processing Area Operating Costs	$/t mill feed	0.0	0.2	0.2	0.2	0.1	0.1	0.1	0.1	0.1	0.1	0.1	0.1	0.1	0.0
Tailings Area Operating Costs	$/t mill feed	0.0	1.3	1.6	1.1	1.3	0.9	1.2	1.0	1.0	0.9	0.9	1.0	1.0	0.0
Tailings Area Operating Costs	$M/y [CAD]	0.0	1.9	3.1	2.1	3.9	3.8	4.8	4.2	4.1	3.7	3.7	3.8	2.5	0.0
Total Operating Costs	$M/y [CAD]	0.0	2.3	3.6	2.6	4.2	4.2	5.2	4.6	4.5	4.1	4.2	4.2	2.7	0.0
Annual Cash Flow (Negative)	$M/y [CAD]	17.1	2.3	10.3	4.4	10.9	4.2	10.5	4.6	4.5	6.5	4.2	4.2	2.7	5.2
Discounted Cash Flow (Negative	$M/y [CAD]	17.1	2.2	9.3	3.8	8.9	3.3	7.9	3.3	3.1	4.2	2.5	2.5	1.5	2.8
Cumulative Net Present Costs	**$M [CAD]**	**17**	**19**	**29**	**32**	**41**	**45**	**52**	**56**	**59**	**63**	**65**	**68**	**69**	**72**

Filtered Tailings	Unit	0	1	2	3	4	5	6	7	8	9	10	11	12	13
Mine Feed Plar	Mt/y	0.0	1.5	2.0	2.0	3.0	4.0	4.0	4.0	4.0	4.0	4.0	4.0	2.4	0.0
Processing Area Capital Costs	$M [CAD]	40.2	0.0	0.0	13.4	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
Tailings Area Capital Costs	$M [CAD]	3.5	0.0	2.1	0.0	2.1	0.0	1.7	0.0	0.0	2.3	0.0	0.0	0.0	4.2
Processing Area Operating Costs	$/t mill feed	0.0	1.6	1.6	1.6	1.0	1.0	1.0	1.0	1.0	1.0	0.1	0.1	0.1	0.0
Tailings Area Operating Costs	$/t mill feed	0.0	4.0	3.1	3.1	3.0	2.4	2.5	2.5	2.5	2.4	0.9	0.8	0.9	0.0
Tailings Area Operating Costs	$M/y [CAD]	0.0	6.0	6.3	6.2	9.1	9.8	9.8	10.1	10.2	9.7	3.4	3.4	2.3	0.6
Total Operating Costs	$M/y [CAD]	0.0	8.4	9.5	9.4	12.1	13.7	13.8	14.1	14.2	13.7	3.8	3.8	2.5	0.0
Annual Cash Flow (Negative)	$M/y [CAD]	43.7	8.4	11.6	22.8	14.2	13.7	15.5	14.1	14.2	16.0	3.8	3.8	2.5	4.2
Discounted Cash Flow (Negative	$M/y [CAD]	43.7	8.0	10.5	19.7	11.7	10.8	11.6	10.0	9.6	10.3	2.3	2.2	1.4	2.2
Cumulative Net Present Costs	**$M [CAD]**	**44**	**52**	**62**	**82**	**94**	**104**	**116**	**126**	**136**	**146**	**148**	**150**	**152**	**154**

Financials	Value	
Discount Rate [%]	5%	Assumption
Inflation [%]	0%	Assumption

TMF OPTION 13 ANALYSIS

Conventional Tailings	Unit	0	1	2	3	4	5	6	7	8	9	10	11	12	13
Mine Feed Plan	Mt/y	0.0	1.5	2.0	2.0	3.0	4.0	4.0	4.0	4.0	4.0	4.0	4.0	2.4	0.0
Processing Area Capital Costs	$M [CAD]	5.4	0.0	0.0	1.9	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
Tailings Area Capital Costs	$M [CAD]	23.7	0.0	19.0	0.0	19.0	0.0	15.2	0.0	0.0	2.7	0.0	0.0	0.0	6.2
Processing Area Operating Costs	$/t mill feed	0.0	0.1	0.1	0.1	0.1	0.1	0.1	0.1	0.1	0.1	0.1	0.1	0.1	0.0
Tailings Area Operating Costs	$/t mill feed	0.0	1.4	1.6	1.2	1.4	1.0	1.3	1.1	1.1	1.0	1.0	1.0	1.1	0.0
Tailings Area Operating Costs	$M/y [CAD]	0.0	2.0	3.3	2.3	4.2	4.1	5.1	4.5	4.5	4.1	4.1	4.2	2.7	0.0
Total Operating Costs	$M/y [CAD]	0.0	2.2	3.5	2.5	4.4	4.4	5.4	4.8	4.8	4.4	4.4	4.5	2.8	0.0
Annual Cash Flow (Negative)	$M/y [CAD]	29.2	2.2	22.5	4.4	23.4	4.4	20.6	4.8	4.8	7.1	4.4	4.5	2.8	6.2
Discounted Cash Flow (Negative	$M/y [CAD]	29.2	2.1	20.4	3.8	19.2	3.5	15.4	3.4	3.2	4.6	2.7	2.6	1.6	3.3
Cumulative Net Present Costs	**$M [CAD]**	**29**	**31**	**52**	**55**	**75**	**78**	**94**	**97**	**100**	**105**	**107**	**110**	**112**	**115**

Thickened Tailings	Unit	0	1	2	3	4	5	6	7	8	9	10	11	12	13
Mine Feed Plan	Mt/y	0.0	1.5	2.0	2.0	3.0	4.0	4.0	4.0	4.0	4.0	4.0	4.0	2.4	0.0
Processing Area Capital Costs	$M [CAD]	8.5	0.0	0.0	2.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
Tailings Area Capital Costs	$M [CAD]	15.0	0.0	11.7	0.0	11.7	0.0	9.4	0.0	0.0	2.3	0.0	0.0	0.0	6.2
Processing Area Operating Costs	$/t mill feed	0.0	0.2	0.2	0.2	0.1	0.1	0.1	0.1	0.1	0.1	0.1	0.1	0.1	0.0
Tailings Area Operating Costs	$/t mill feed	0.0	1.3	1.6	1.1	1.3	0.9	1.2	1.0	1.0	0.9	0.9	1.0	1.0	0.0
Tailings Area Operating Costs	$M/y [CAD]	0.0	1.9	3.1	2.1	3.9	3.8	4.8	4.2	4.1	3.7	3.7	3.8	2.5	0.0
Total Operating Costs	$M/y [CAD]	0.0	2.3	3.6	2.6	4.2	4.2	5.2	4.6	4.5	4.1	4.2	4.2	2.7	0.0
Annual Cash Flow (Negative)	$M/y [CAD]	23.5	2.3	15.3	4.6	15.9	4.2	14.6	4.6	4.5	6.5	4.2	4.2	2.7	6.2
Discounted Cash Flow (Negative	$M/y [CAD]	23.5	2.2	13.9	4.0	13.1	3.3	10.9	3.3	3.1	4.2	2.5	2.5	1.5	3.3
Cumulative Net Present Costs	**$M [CAD]**	**24**	**26**	**40**	**44**	**57**	**60**	**71**	**74**	**77**	**81**	**84**	**86**	**88**	**91**

Filtered Tailings	Unit	0	1	2	3	4	5	6	7	8	9	10	11	12	13
Mine Feed Plan	Mt/y	0.0	1.5	2.0	2.0	3.0	4.0	4.0	4.0	4.0	4.0	4.0	4.0	2.4	0.0
Processing Area Capital Costs	$M [CAD]	40.2	0.0	0.0	13.4	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
Tailings Area Capital Costs	$M [CAD]	4.0	0.0	2.6	0.0	2.6	0.0	2.1	0.0	0.0	2.3	0.0	0.0	0.0	5.7
Processing Area Operating Costs	$/t mill feed	0.0	1.6	1.6	1.6	1.0	1.0	1.0	1.0	1.0	1.0	0.1	0.1	0.1	0.0
Tailings Area Operating Costs	$/t mill feed	0.0	4.0	3.1	3.1	3.3	2.6	2.6	2.7	2.7	2.6	0.9	0.8	0.9	0.0
Tailings Area Operating Costs	$M/y [CAD]	0.0	6.0	6.3	6.2	9.8	10.4	10.5	10.8	10.9	10.4	3.4	3.4	2.3	0.6
Total Operating Costs	$M/y [CAD]	0.0	8.4	9.5	9.4	12.8	14.4	14.5	14.8	14.9	14.4	3.8	3.8	2.5	0.0
Annual Cash Flow (Negative)	$M/y [CAD]	44.2	8.4	12.1	22.8	15.4	14.4	16.6	14.8	14.9	16.7	3.8	3.8	2.5	5.7
Discounted Cash Flow (Negative	$M/y [CAD]	44.2	8.0	11.0	19.7	12.7	11.3	12.4	10.5	10.1	10.8	2.3	2.2	1.4	3.0
Cumulative Net Present Costs	**$M [CAD]**	**44**	**52**	**63**	**83**	**96**	**107**	**119**	**130**	**140**	**151**	**153**	**155**	**157**	**160**

Financials	Value	
Discount Rate [%]	5%	Assumption
Inflation [%]	0%	Assumption

TMF OPTION 14 ANALYSIS

Conventional Tailings	Unit	0	1	2	3	4	5	6	7	8	9	10	11	12	13
Mine Feed Plan	Mt/y	0.0	1.5	2.0	2.0	3.0	4.0	4.0	4.0	4.0	4.0	4.0	4.0	2.4	0.0
Processing Area Capital Costs	$M [CAD]	4.5	0.0	0.0	1.4	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
Tailings Area Capital Costs	$M [CAD]	27.8	0.0	22.4	0.0	22.4	0.0	17.9	0.0	0.0	2.0	0.0	0.0	0.0	5.1
Processing Area Operating Costs	$/t mill feed	0.0	0.1	0.1	0.1	0.1	0.1	0.1	0.1	0.1	0.1	0.1	0.1	0.1	0.0
Tailings Area Operating Costs	$/t mill feed	0.0	1.4	1.6	1.2	1.4	1.0	1.3	1.1	1.1	1.0	1.0	1.0	1.1	0.0
Tailings Area Operating Costs	$M/y [CAD]	0.0	2.0	3.3	2.3	4.2	4.1	5.1	4.5	4.5	4.1	4.1	4.2	2.7	0.0
Total Operating Costs	$M/y [CAD]	0.0	2.2	3.5	2.5	4.4	4.4	5.4	4.8	4.8	4.4	4.4	4.5	2.8	0.0
Annual Cash Flow (Negative)	$M/y [CAD]	32.3	2.2	25.8	3.9	26.8	4.4	23.4	4.8	4.8	6.4	4.4	4.5	2.8	5.1
Discounted Cash Flow (Negative	$M/y [CAD]	32.3	2.1	23.4	3.4	22.0	3.5	17.4	3.4	3.2	4.1	2.7	2.6	1.6	2.7
Cumulative Net Present Costs	**$M [CAD]**	**32**	**34**	**58**	**61**	**83**	**87**	**104**	**108**	**111**	**115**	**118**	**120**	**122**	**125**

Thickened Tailings	Unit	0	1	2	3	4	5	6	7	8	9	10	11	12	13
Mine Feed Plan	Mt/y	0.0	1.5	2.0	2.0	3.0	4.0	4.0	4.0	4.0	4.0	4.0	4.0	2.4	0.0
Processing Area Capital Costs	$M [CAD]	7.9	0.0	0.0	1.6	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
Tailings Area Capital Costs	$M [CAD]	19.1	0.0	15.1	0.0	15.1	0.0	12.1	0.0	0.0	1.8	0.0	0.0	0.0	5.4
Processing Area Operating Costs	$/t mill feed	0.0	0.2	0.2	0.2	0.1	0.1	0.1	0.1	0.1	0.1	0.1	0.1	0.1	0.0
Tailings Area Operating Costs	$/t mill feed	0.0	1.3	1.6	1.1	1.3	0.9	1.2	1.0	1.0	0.9	0.9	1.0	1.0	0.0
Tailings Area Operating Costs	$M/y [CAD]	0.0	1.9	3.1	2.1	3.9	3.8	4.8	4.2	4.1	3.7	3.7	3.8	2.5	0.0
Total Operating Costs	$M/y [CAD]	0.0	2.3	3.6	2.6	4.2	4.2	5.2	4.6	4.5	4.1	4.2	4.2	2.7	0.0
Annual Cash Flow (Negative)	$M/y [CAD]	27.0	2.3	18.7	4.3	19.3	4.2	17.3	4.6	4.5	5.9	4.2	4.2	2.7	5.4
Discounted Cash Flow (Negative	$M/y [CAD]	27.0	2.2	16.9	3.7	15.9	3.3	12.9	3.3	3.1	3.8	2.5	2.5	1.5	2.9
Cumulative Net Present Costs	**$M [CAD]**	**27**	**29**	**46**	**50**	**66**	**69**	**82**	**85**	**88**	**92**	**95**	**97**	**98**	**101**

Filtered Tailings	Unit	0	1	2	3	4	5	6	7	8	9	10	11	12	13
Mine Feed Plan	Mt/y	0.0	1.5	2.0	2.0	3.0	4.0	4.0	4.0	4.0	4.0	4.0	4.0	2.4	0.0
Processing Area Capital Costs	$M [CAD]	40.2	0.0	0.0	13.4	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
Tailings Area Capital Costs	$M [CAD]	4.2	0.0	2.7	0.0	2.7	0.0	2.2	0.0	0.0	1.8	0.0	0.0	0.0	5.1
Processing Area Operating Costs	$/t mill feed	0.0	1.6	1.6	1.6	1.0	1.0	1.0	1.0	1.0	1.0	0.1	0.1	0.1	0.0
Tailings Area Operating Costs	$/t mill feed	0.0	3.5	2.8	2.7	2.8	2.3	2.3	2.4	2.4	2.2	0.9	0.8	0.9	0.0
Tailings Area Operating Costs	$M/y [CAD]	0.0	5.3	5.6	5.5	8.4	9.1	9.1	9.4	9.5	9.0	3.4	3.4	2.3	0.6
Total Operating Costs	$M/y [CAD]	0.0	7.7	8.8	8.7	11.4	13.1	13.1	13.4	13.5	13.0	3.8	3.8	2.5	0.0
Annual Cash Flow (Negative)	$M/y [CAD]	44.4	7.7	11.6	22.1	14.1	13.1	15.3	13.4	13.5	14.8	3.8	3.8	2.5	5.1
Discounted Cash Flow (Negative	$M/y [CAD]	44.4	7.3	10.5	19.1	11.6	10.2	11.4	9.6	9.1	9.5	2.3	2.2	1.4	2.7
Cumulative Net Present Costs	**$M [CAD]**	**44**	**52**	**62**	**81**	**93**	**103**	**115**	**124**	**133**	**143**	**145**	**147**	**149**	**151**

APPENDIX B

Design Criteria

TECHNICAL MEMORANDUM

DATE 13 March 2020 **Reference No.** 19130660-TM-001

TO Robbert Borst
Marathon Gold

CC Auscenco

FROM Philip Addis and Peter Merry **EMAIL paddis@golder.com**

BASIS OF DESIGN FOR THE VALENTINE GOLD PROJECT TAILINGS STORAGE FACILITY – PRE-FEASIBILITY STUDY

Table 1: Revision History

Revision	Date	Revision Edits
A	17 November 2019	Issued for client review
B	19 December 2019	Dam and filtered stack geometry, mine plan, tailings tonnage
0	13 March 2020	Hydraulic design criteria, earthquake design ground motions

INTRODUCTION

Marathon Gold has retained Golder to complete a pre-feasibility study for a tailings storage facility (TSF) at the Valentine Gold Project. The purpose of this document is to present the basis of design, and the associated design criteria, that Golder will adopt to carry out the design. This basis of design document clarifies the scope of work and input parameters and is comprised of relevant input data and information for use by all parties as a basis for the design and scope of works. This document will be updated during the design as more information is made available, or if the design basis changes.

1.0 BASIS OF DESIGN

1.1 Design Codes, Standards and Guidelines

The engineering design of the TSF will be consistent with any internal requirements of Marathon Gold and Canadian regulatory requirements. These requirements, codes and guidelines comprise of the following documents:

- Canadian Dam Association, 2007, “Dam Safety Guidelines”. Revised 2013, CDA Publication.
- Canadian Dam Association, 2014, “Application of Dam Safety Guidelines to Mining Dams”, CDA Publication.
- The Mining Association of Canada, 2017, “A Guide to the Management of Tailings Storage Facilities”, Third Edition.
- Newfoundland and Labrador Mining Act SNL1999 Chapter M-15.1.

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1.2 Design Parameters and Criteria

The design parameters and criteria are summarised in Table 2, with further details described in Sections 1.3 to 1.4. Numerous items have been left blank in this initial revision of the design basis as they require input from others or will be determined as the project advances.

Table 2: Design Parameters and Criteria Summary

Design Parameter	Design Input	Established by	Reference / Comments
Dam Classification			
Dam classification provides recommendation for return period of the design seismic event and flood	CDA – Very High	Golder	
Production Data			
Life of mine	12 years	Moose Mountain	See Table 3
Total tonnage of tailings produced	40.677 million tonnes	Moose Mountain	See Table 3 for annual breakdown
Total tonnage of tailings to TSF	30.125 million tonnes	Marathon Gold	
Tailings disposal location	Year 1 to 9 – TSF Year 10 to 12 – Leprechaun Pit	Marathon Gold	
Process Plant availability to operate during the year	92.5 %	Golder	Assumed
Moisture content of the ore going into the Process Plant	2%	Golder	Assumed
Process Plant Throughput	See Table 3	Moose Mountain	January 15, 2020
Tailings Characteristics			
Design slurry density (solids concentration) (by mass)	Conventional: 40% solids (Yr 1 and 2) 45% solids (Yr 3 and onwards) Thickened: 65% solids Filtered: 80 to 85%	Ausenco	
Particle size distribution and plasticity	P_{80} =75 µm, non-plastic (Yr 1 and 2) P_{80} =150 µm, non-plastic (Yr 3 and onwards)	Lycopodium 2018	To be revised following additional testing
Average tailings specific gravity	2.68	Ausenco	To be revised following additional testing
Deposited void ratio (e) for tailings	1.0 (Conventional) 0.9 (Thickened) 0.8 (Filtered)	Golder	Assumed
Average in situ dry density	1.34 t/m^3 (Conventional) 1.41 t/m^3 (Thickened) 1.49 t/m^3 (Filtered)	Golder	Assumed based on void ratios

Table 2: Design Parameters and Criteria Summary

Design Parameter	Design Input	Established by	Reference / Comments
Total volume of tailings to TSF	26.1 Mm^3 (Conventional) 24.8 Mm^3 (Thickened) 23.5 Mm^3 (Filtered)	Golder	Assumed based on dry density
Total volume of tailings to Leprechaun Pit	13.5 Mm^3 (Conventional) 12.8 Mm^3 (Thickened) 12.1 Mm^3 (Filtered)	Golder	Assumed based on dry density
Beach slope	Conventional deposition - 0.5% to 1% Thickened – 3%	Golder	Assumed.
Shear strength	No data		
Permeability (Vertical)	No data		
TSF Dam Design (Conventional and Thickened Tailings)			
Crest width	20 m Intermediate 10 m Final	Golder	
Upstream slope	3 (H): 1(V) Intermediate 3.5 (H) :1 (V) Average	Golder	Required for liner installation
Downstream slope	2 (H) :1 (V) Average	Golder	
Maximum embankment height	49 m	Golder	
Maximum embankment elevation	408.3 masl	Golder	
Lining System for Upstream Dam Slopes	1.5 mm Linear Low Density Polyethylene (LLDPE) Geomembrane	Golder	
TSF Basin Liner	To be determined		
TSF decant pond size	0.84 million m^3 - average precipitation conditions 0.91 million m^3 - 25 year wet conditions 0.2 million m^3 – EDF storage	Golder	
Filtered Stack Design			
Bench geometry	10 m width, 10 m height, 40 m slope length (4H:1V)	Golder	
Overall Slope	5H:1V	Golder	
Polishing Pond Design			
Retention time	5 days	Ausenco	
Storage depth for total suspended solids	0.5 m	Golder	Assumed.
Length to Width Ratio	4:1	Golder	

Table 2: Design Parameters and Criteria Summary

Design Parameter	Design Input	Established by	Reference / Comments
Water Treatment Plant Design			
Water Treatment Plant Discharge Rate	170 m^3/h nominal 255 m^3/h maximum	Ausenco	
Water Treatment Plant Operation	8 months per annum (April to December)	Golder	
Design Stormwater Management			
Environmental Design Flood (EDF) – Provide live storage for EDF to prevent activation of spillway	1:100-year 24 hour storm (75 mm)	Golder	
Flood Routing and Inflow Design Flood (IDF) – Spillway capacity designed to safely pass the IDF	Probable Maximum Flood (PMF)	Golder	
24-hour probable maximum precipitation (PMP)	309 mm	Golder	(Hogg, 1985)
Freeboard Requirements – The crest must not be overtopped under wind set-up and wave run-up	Freeboard allowance of 2 m for: - Wind setup and wave run-up - Safety allowance	Golder	
Water Balance Inputs			
Catchment areas	- TSF: 223 ha - Leprechaun Pit: 54 ha - Polishing Pond: 5 ha	Golder	
Runoff coefficients	- Natural ground: 0.72 - Prepared ground: 0.85 - Dry tailings: 0.4 - Ponds and wet tailings: 1.0 - Waste Rock: 0.5	Golder	
TSF natural ground, prepared ground, wet beach, dry beach and pond area	- 10% natural ground - 10% prepared ground - 45% wet tailings and pond - 35% dry tailings surface	Golder	Assumed
Climate Data			
Average annual temperature	3.8 degrees Celsius	Stantec	Stantec 2017
Average annual precipitation	1,236 mm with 359 cm snow and 877 mm of rain	Stantec	Stantec 2017
Annual pan evaporation loss	475 – 500 mm	Stantec	Water Resources Atlas for Newfoundland (1992)
Average ratio of measured pan evaporation to lake evaporation	0.77	Stantec	Stantec 2017
Meteorological station	Buchans Station ID 8400698	Stantec	Stantec 2017

Table 2: Design Parameters and Criteria Summary

Design Parameter	Design Input	Established by	Reference / Comments
Seepage and Runoff Collection Management			
Low permeable element	1.5 mm LLDPE geomembrane on upstream dam slope	Golder	
Collection ditches - return interval	1 in 25-year storm event	Golder	Critical storm duration to be determined during PFS
Collection ditch geometry	Side slopes 2H:1V	Golder	
Ditch erosion protection	Rip-rap	Golder	
Groundwater level	0 to 2 m below ground level	Stantec	
Acceptable Factors of Safety			
Static conditions where loss of containment is possible	1.3 (short term) 1.5 (long term)	Golder	CDA 2014
Post-seismic	1.0	Golder	CDA 2014
Design Earthquake Loadings			
Seismic Event Return Interval	½ between 1:2,475 yr and 1:10,000 yr AEP	Dam Classification is “Very High”	CDA 2014,
Seismic Load	0.038*g* (½ of the design PGA value of 0.76*g* corresponding to the recommended design AEP	Golder	Extrapolated from NBCC 2015
Post Closure	Maximum Credible Earthquake (MCE)	Golder	CDA 2014

Table 3: New Base Case Production Schedule (Ausenco 21 Jan2020)

Year	Tailings Production (Mt)	Mine Waste Production (Mt)
-2	-	-
-1	-	10.16
1	1.875	31.03
2	2.5	36.03
3	2.5	51.49
4	3.25	44.00
5	4	43.53
6	4	35.12
7	4	29.00
8	4	13.48
9	4	4.03
10	4	
11	4	
12	2.55	-

Note: 1. Process Plant throughput is approximate based on 100% availability.

1.3 Additional Design Assumptions

In addition to the basis of the design, the following additional general assumptions will be adopted:

- Limited geotechnical information is available for the site. Based on review of surficial geology mapping for the area and observations made during a site visit, it is assumed that the foundation of the TSF comprises granular till overlying bedrock. Geotechnical investigation is required to confirm this assumption.
- The TSF embankment will be constructed with a waste rock material from pit development. Transition and filter material will be borrowed locally as identified from specifically targeted field investigations and/or processed from waste rock. It is assumed that a low permeable geomembrane liner will be required as a local source of clay / fine grained till is not available at this time. Geotechnical investigation is required to confirm this assumption.
- TSF will be raised in a downstream direction with waste rock from the open pit.
- Excess water will be decanted from the facilities using a barge decant system – to be designed by Ausenco.
- Polishing pond will be constructed with perimeter embankments above the natural topography; therefore, run-off will be diverted away from the pond. The catchment of the pond is only the pond itself.
- All seepage from the TSF is collected and pumped back to the facility (i.e. no net seepage loss)
- Reclaim water from the facility to the process plant occurs year-round.

1.4 Closure Criteria

The proposed TSF closure design criteria are to meet the requirements outlined in Chapter M-15.1 Sections 9 and 10 of the NL *Mining Act.*

2.0 CLOSING REMARKS

We trust that this technical memorandum provides sufficient information regarding the basis of design for the technical studies to support the PFS level design. Please do not hesitate to contact us if you require any further elaboration or clarification.

Golder Associates Ltd.

REGISTERED PROFESSIONAL ENGINEER
PEG Newfoundland and Labrador
W.P.H. MERRY
SIGNATURE
June 30, 2020
DATE
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https://golderassociates.sharepoint.com/sites/117078/project files/6 deliverables/4. pfs design report/appendices/appendix b - design criteria/19130660 valentine gold tsf basis of design - rev0.docx

PROVINCE OF NEWFOUNDLAND AND LABRADOR
PEG Newfoundland and Labrador
PERMIT HOLDER
This Permit Allows
GOLDER ASSOCIATES LTD.
To practice Professional Engineering in Newfoundland and Labrador.
Permit No. as issued by PEGNL D0029
which is valid for the year 2020

REFERENCES

Canadian Dam Association, 2007, “Dam Safety Guidelines”. Revised 2013, CDA Publication.

Canadian Dam Association, 2014. “Application of Dam Safety Guidelines to Mining Dams”, CDA Publication.

Hogg, W.D and D.A. Carr, 1985. Rainfall Frequency Atlas for Canada. Environment Canada 1985.

Stantec, 2017, “Valentine Lake Gold Project – Baseline Hydrology and Surface Water Quality Monitoring Program”. Report prepared for Marathon Gold Corporation, Toronto, ON. December 1, 2017. 46 pp. + Attachments.

Stantec, 2018, “Valentine Lake Gold Project – Preliminary Economic Assessment Operational Mine Site Water Balance (DOC-NO-MEM-001-400-A-19Mar18)”. Memo prepared for Marathon Gold Corporation, Toronto, ON. March 19, 2018. 7 pp. + Attachments.

Lycopodium Minerals Canada Ltd, October 2018, “Technical Report Preliminary Economic Assessment of the Valentine Lake Gold Project”.

Linacre. http://www-das.uwyo.edu/~geerts/cwx/penpan.html.

Department of Environment and Lands, Water Resources Division, Government of Newfoundland and Labrador, 1992, “Water Resources Atlas for Newfoundland”.

Mining Act, 1999. Chapter M-15.1: An Act Respecting the Operation of Mines and Mills in the Province. Department of Mines and Energy, Province of Newfoundland and Labrador. Assented to December 14, 1999.

APPENDIX C

Water Balance

GOLDER

TECHNICAL MEMORANDUM

DATE 30 June 2020 **Reference No.** 19130660-TM-002

TO Robbert Borst
Marathon Gold Corporation

CC

FROM Adwoa Cobbina and Peter Merry **EMAIL Adwoa_Cobbina@golder.com**

VALENTINE GOLD PROJECT – WATER BALANCE AND HYDRAULIC DESIGN FOR TAILINGS MANAGEMENT FACILITY (REVISION 0)

1.0 INTRODUCTION

Golder Associates Ltd. (Golder) has been retained by Marathon Gold Corporation (Marathon) to complete a pre-feasibility Study (PFS) level design of a tailings management facility (TMF) for the Valentine Gold Project, in south central Newfoundland. The following technical memorandum summarizes the water management plan which includes a water balance model and sizing of the water management structures at the TMF.

2.0 WATER BALANCE MODELLING

Tailings management is primarily a water management problem. The precipitation and process flows must pass through a disposal facility in a near constant stream over the entire life of the mine. The challenge is to allow this to safely happen over a wide range of climatic and operating conditions in a facility that is continuously growing and expanding.

A water balance was developed on linked Microsoft Excel spreadsheets to simulate water flows to and from the TMF over a one-year period during operations under both average annual precipitation and 25-year wet conditions. Excess water from the open pit dewatering and run-off from waste rock stockpiles and plant site area are managed separately and are not included in this model.

Golder analyzed two water balance modelling scenarios. The first scenario has the TMF serving as the active tailings disposal location. This is planned for the initial 9 years of the mine life. The second scenario has the Leprechaun Pit serving as the active tailing disposal location. This is planned for years 10 to 12 of the mine life. Although the second scenario has Leprechaun Pit serving as the active tailing facility, water from the original TMF is still available during this period for reclaim to the mill as necessary. Under both scenarios, excess water is discharged to the wastewater treatment plant, and then transferred to a polishing pond before discharging to the environment.

Water balance modelling was carried out for these scenarios to estimate the amount of water collected at the TMF and in Leprechaun Pit, the amount of water available in the facilities for reclaim, the discharge volumes to the wastewater treatment plant, and the storage volume requirements for sizing the TMF pond.

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A flow diagram of the water balance for the site is found in Figure 1.

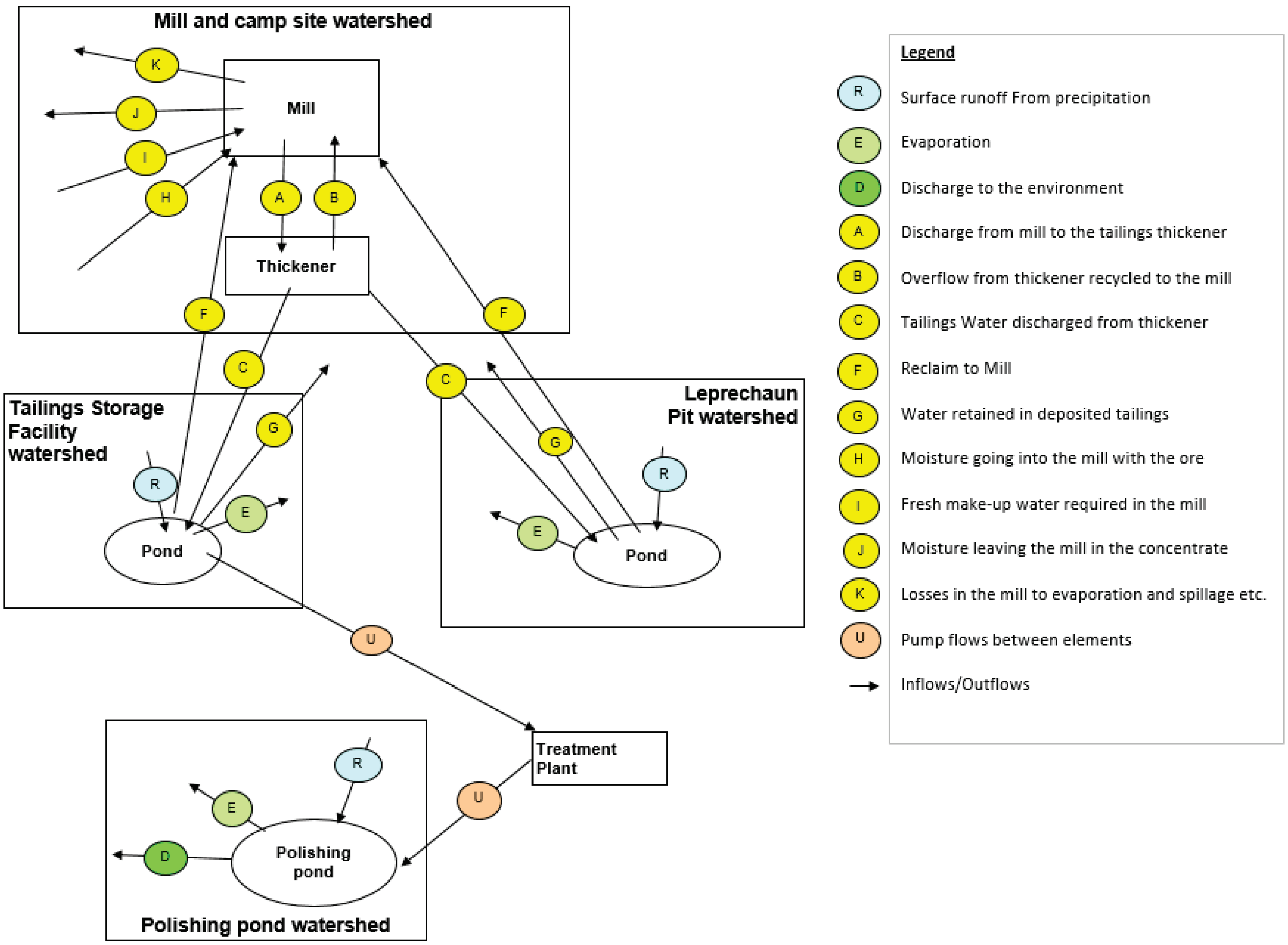

Figure 1: Flow Diagram of the Valentine Gold Water Balance

2.1 Inputs and Assumptions

2.1.1 Catchment Areas

The facilities and watershed areas listed in Table 1 were considered in the water balance. Available 5 m contour mapping was used to define the catchment areas.

Table 1: Valentine Gold Site - Water Balance Catchment Areas

Facility	Area (ha)	Collecting Area (ha)	Collecting Area (ha)
Tailings Storage Facility	223	Natural ground	22
		Prepared ground	22
		Pond & wet tailings	100
		Dry tailings beach	78
Leprechaun Pit	54	Prepared ground	11
		Pond & wet tailings	24
		Dry tailings beach	19
Polishing Pond	5	Pond	5
TOTAL	**282**		

2.1.2 Operating Data

Operating data is required for the water balance modelling. The model considered a production rate of 4 Mtpa. Further details are provided in Attachment A.

2.1.3 Climate Data

Precipitation, temperature and evapotranspiration data for the site were obtained from a 2017 hydrology baseline study (Stantec 2017) and are summarised in Table 2.

Table 2: Site Monthly and Annual Precipitation Data (Climate Normals:1981-2010)

Month	Jan	Feb	Mar	April	May	June	July	Aug	Sep	Oct	Nov	Dec	Annual
Precipitation (mm)	122	98	95	86	87	88	95	123	110	98	112	123	1236
Temperature (°C)	-8.2	-8.4	-4.8	1	7	12	16.3	16.2	11.9	6	0.5	-4.5	3.8
Evapotranspiration*	0	0	0	9	53	88	115	105	68	34	3	0	475

* Only the annual evapotranspiration was provided by Stantec (2017). The monthly distribution was estimated using the Thornthwaite equation.

2.1.4 Estimation of Surface Runoff

The runoff factor is a dimensionless coefficient relating the amount of runoff to the amount of precipitation received. It is a larger value for areas with low infiltration and high runoff, and it is a lower value for permeable and/or well vegetated areas.

The following monthly averaged runoff factors were assumed in the water balance to calculate surface runoff from the various types of land cover encountered for the Valentine Gold Site:

- Natural ground - 72%
- Prepared ground - 85% (built up areas, building roofs, internal roads, parking areas)
- Ponds and wet tailings -100%
- Dry tailings beach - 40%
- Waste rock and ore storage - 50%

2.1.5 Assumptions

The following was assumed in the development of the TMF and Leprechaun Pit water balances:

- Mill's availability to operate during the year is 92.5%
- The mill requires a minimum of 10% of clean make-up water
- Moisture content of the ore going into the mill is 2%
- All seepage from the TMF is collected and pumped back to the facility (i.e. no net seepage loss); and
- The wastewater treatment plant is only active during non-winter months (mid-April – mid-November)
- Reclaim water from the TMF to the mill occurs year-round
- Groundwater flows to Leprechaun pit are excluded

2.2 Water Balance Modelling Results

The water balance results for the first scenario, where the TMF is the active tailings disposal location, indicate that the TMF can meet the full make-up water requirements of the mill. As the water treatment plant will only be active during the non-winter months, the TMF pond needs to be sized to store water accumulated during the non-discharge period. Under average climate conditions, a peak pond capacity of 0.84 Mm^3 is required. Under the 25-year wet conditions, this requirement is raised to 0.91 Mm^3. To store the run-off from the Environmental Design Flood an additional 0.2 Mm^3 is required. It is therefore recommended that the TMF should allow for an operational pond with a capacity of up to 1.1 Mm^3 below the spillway invert. During the discharge period, the recommended treatment rates varies from 116 m^3/hr to 190 m^3/hr for average and 25-year wet conditions, respectively.

The modelling results are summarized in Table 3 for average year precipitation and 25-year wet precipitation. The complete water balance for average climate conditions can be found in Attachment A. A condensed 25-year wet water balance for only the TMF pond is also presented in Attachment A.

Table 3: Water Balance Summary for Tailings Deposited at TMF at 4 Mtpa – Ultimate Configuration

Flows		Annual Precipitation Conditions	
		Mean	25-yr Wet
		Annual Volume (Mm^3/year)	
Inflows	Tailings Water	2.15	2.15
	Surface Runoff	2.06	2.49
	Total Inflows	4.21	4.64
Outflows	Water Retained in Deposited Tailings	1.34	1.34
	Evaporation	0.32	0.32
	Reclaim to Mill	1.88	1.88
	Discharge to Water Treatment Plant	0.67	1.10
	Excess Water Stored	0	0
	Total Outflows	4.21	4.64
Mill Make-up Water Deficit (reclaim water required from an external source)		0	0

In contrast, once Leprechaun Pit becomes the active tailings facility, (Years 10 – 12), its catchment area is insufficient to meet the full make-up water requirements of the mill (on the assumption of zero groundwater inflows). In this case, approximately 35% of the makeup requirements (under average precipitation conditions) must be taken from another source – i.e., the TMF. In this case, the total volume of water treated and discharged to the environment is 30% to 40% higher then when the tailings deposition is in the TMF. The recommended treatment rates will increase to 190 m^3/hr to 280 m^3/hr for average and 25-year wet conditions, respectively. The model results are presented in Table 4.

Table 4: Water Balance Summary for Tailings Deposited at Leprechaun Pit at 4 Mtpa – Ultimate Configuration

Flows		Annual Precipitation Conditions	
		Mean	25-yr Wet
		Annual Volume (Mm^3/year)	
Inflows	Tailings Water	2.15	2.15
	Surface Runoff	Pit: 0.5 TMF: 2.06	Pit: 0.61 TMF: 2.49
	Total Inflows	4.71	5.25
Outflows	Water Retained in Deposited Tailings	1.34	1.34
	Evaporation	Pit: 0.08 TMF: 0.32	Pit: 0.08 TMF: 0.32
	Reclaim to Mill	Pit: 1.24 TMF: 0.64	Pit: 1.34 TMF: 0.54
	Discharge to Water Treatment Plant	TMF: 1.10	TMF: 1.63
	Excess Water Stored	0	0
	Total Outflows	4.71	5.25
Mill Make-up Water Deficit (reclaim water required from an external source)		0	0

3.0 COLLECTION DITCHES AND EMERGENCY SPILLWAY SIZING

Run-off collection ditches are located at the downstream toe of the TMF dam. These ditches collect run-off from the downstream dam slope and shallow seepage flow. The ditches will direct the flow to two collection sumps. The TMF will have an emergency spillway to prevent dam overtopping under extreme climate conditions.

3.1 Design Criteria

The design criteria to support the pre-feasibility level design of the ditches and spillway are provided in Golder (2020). The most relevant criteria are as follows:

- The ditches are designed to safely convey the peak flow resulting from the 1 in 25-year rainfall storm event.
- The TMF emergency spillway must allow safe routing of the Probable Maximum Precipitation (PMP) to maintain a minimum freeboard of 0.5 m from wave run-up.

3.2 Collection Ditches

A 15-minute duration rainfall storm event with a 25-year return period for the site is approximately 13.5 mm. The rainfall intensity is 54 mm/hr. This data was obtained from the Intensity-Duration-Frequency data of an Environment and Climate Change Canada climate station (Deer Lake) provided from a hydrology baseline report (Stantec 2017).

The Rational Method was applied for the estimation of the design flow for the ditches. This method is commonly used to calculate peak flows for small catchments. The intensity, drainage areas, and runoff coefficient were used to compute the peak flow estimates. A runoff coefficient of 0.76 (to represent both natural ground and the downstream dam face) and 0.85 (downstream dam face only) was used for the ditches at the first stage (year -1) and ultimate stage (year 7) respectively.

The ditches will be excavated in original ground and the base of the ditches will be lined with riprap for erosion protection. The depths and base widths for the ditches was determined using the Manning's Equation. The parameters required to calculate the widths and depths for the ditches are the side slopes, the average longitudinal gradient, and a Manning's roughness coefficient

The contact water collection system at the Valentine Gold site includes the ditches to convey the runoff from the east side of the TMF and the south side. The design peak flows and dimensions for the ditches are shown in Table 5 for year -1 with a freeboard of 0.3 m assumed.

Table 5: Contact Water Ditch Design Flows and Dimensions

Ditch Location	Year Built	Design Peak Flow (m^3/s)	Average Channel Slope (%)	Channel Side Slopes (H:1V)	Channel Bottom Width (m)	Channel Depth (m)	Channel Length (m)	Riprap D_{50} (mm)
East	-1	2.9	2.3	2	2	1	1400	150
South	-1	2.7	1.3	2	2	1	1400	150

The ditches will be extended at year 7 to accommodate the larger TMF footprint. The channel lengths will increase to 1,800m, but the cross-sectional ditch dimensions remain unchanged.

3.3 TMF Emergency Spillway and Channel

The PMP is the most severe storm for which a dam and its associated facilities are designed. From a safety design perspective, the PMP is the runoff resulting from the largest discrete storm event that can be safely routed through a basin without overtopping a dam. The PMP was calculated as 309 mm using the Rainfall Frequency Atlas for Canada (Environment Canada 1985). As a conservative measure, the design inflow to the TMF emergency spillway was calculated assuming 100% runoff from the tailings and tailings pond surface.
The Hydrological Engineer Corps – Hydrologic Modelling System (HEC-HMS) software Version 4.3 was used to generate the flows.

The spillway was sized for the ultimate stage (Year 7) of the TMF with a 5 m base width and a 2 m depth, the design flow for these dimensions, the resulting design values are as follows:

- Design flow: 10.5 m^3/s
- Peak flow velocity: 3.3 m/s
- Peak flow depth: 1.2 m
- Freeboard under design flow: 0.8m

Spillway discharge is conveyed to the environment through a channel designed using broad-crested weir equations (Smith 1995) were used to calculate the discharge at varying water levels. The channel was designed for the peak discharge presented above. The required design dimensions for the conveyance channel are as follows:

- Average channel slope: 6.9%
- Channel side slopes: 2 H:1V
- Channel bottom width: 6 m
- Channel depth: 1 m
- Channel length: 1590 m
- Riprap D_{50}: 305 mm

4.0 POLISHING POND

The polishing pond was sized based on the 25-yr wet return period monthly treatment rate of 280 m^3/h from the water balance (Section 2.2). The polishing pond capacity accounts for the following:

- Sediment storage depth of 0.5 m (assumed)
- Minimum retention time of 5 days (Golder 2020)
- An active storage depth of 1.1 m
- Storm storage depth of 0.5 m (assumed)

Based on the requirements above, the polishing pond has been sized with a length of 460 m, a width of 88 m, a maximum depth of 2.5 m and an operating storage capacity of 44 000 m^3. The polishing pond will also be provided with an emergency discharge spillway.

5.0 CONCLUSION

We trust that this technical memorandum provides sufficient information regarding the water balance and hydraulic design for the technical studies to support the PFS level design. Please do not hesitate to contact us if you require any further elaboration or clarification.

Golder Associates Ltd.

Adwoa Cobbina
Water Resources Engineer

Peter Merry, P.Eng. (NL)
Principal, Project Director

AC/PA/WPM/hp

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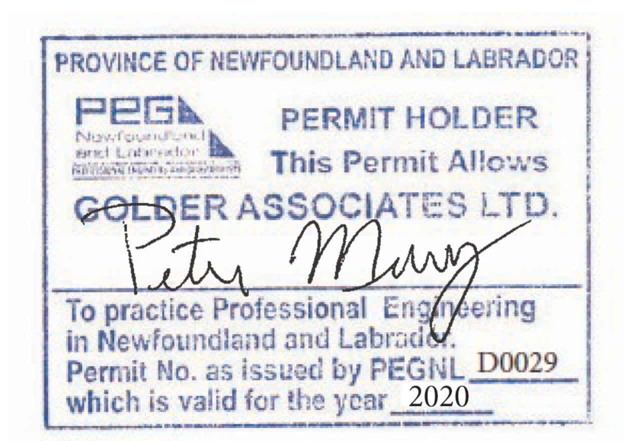

REFERENCES

Environment Canada. (EC) (1985). Rainfall Frequency Atlas for Canada

Golder Associates Ltd. (Golder) (2020). Basis of Design for the Valentine Gold Project Tailings Storage Facility – Pre-Feasibility Study. Golder Doc No.19130660-TM-001. Rev 0

Smith, C.D. (1995) Hydraulic Structures, University of Saskatchewan

Stantec. (2017). Valentine Lake Project: Baseline Hydrology and Surface Water Quality Monitoring Program

Thornthwaite, C. W. (1948). An Approach Toward a Rational Classification of Climate

United States Department of Agriculture. (USDA) (1986). Urban Hydrology for Small Watersheds

ATTACHMENT A

Water Balance Data

Deterministic Mine Site Wide Flow (water balance) Model

Average Climate Conditions

Mine:	Valentine Gold
Owner(s):	Marathon Gold Corporation
Operator:	Marathon Gold Corporation
Location:	Central Western Newfoundland
Product:	Gold
Type of mine:	Open Pit
Tailings streams:	1
Date:	Dec 06, 2019
Level of study:	Pre-Feasibility Study
Golder Project #:	19130660-4000

Notes:

1 Data are only inputted into the orange shaded cells. The relevant data are automatically transferred to other sheets.

2 Each sheet is protected except for the orange shaded cells. The password is simply Golder.

3 The red "tailings streams" at the top of each sheet can be removed from all the sheets by merely deleting it on this cover sheet.

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2020-03-17

Sheet 2
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Sheet

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FLOWS ASSOCIATED WITH PROCESSING THE ORE

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4 Tailings Volume and Flow Calculations Associated with Processing the Ore

PRECIPITATION, EVAPORATION, FLOW LOGIC AND AREAS

5 Precipitation, Runoff, Flood Events & Evaporation Data

6 Flow Logic Diagram

7 List of Flows

8 Collecting Watershed Areas

RUNOFF FLOWS FROM PRECIPITATION

9 - 1 Flows R1 to R4

9 - 2 Flows R5 to R8

FLOWS FROM EVAPORATION, SEEPAGE, MISCELLANEOUS & DRIFTED SNOW MELT

10 Evaporation Losses

ACCUMULATION OF FLOWS IN THE PONDS

11 - 1 Tailings Facility Pond

11 - 2 Pit Mine Pond

11 - 3 Water Treatment Plant

11 - 4 Polishing Pond

Note: The sheet is protected. The password is simply Golder.

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Sheet 3
Operating Data

Definiation: Nominal and design values: The sizing and flow (water balance) modelling for a tailings facility are based on the planned annual mill throughput averaged over 365 days per year. Golder defines this as a nominal production rate. The design of the process facilities, pumps and pipelines must take into account the availability of the mill (% of the year that the mill is available to operate) plus an appropriate factor of safety. A word of caution; sometimes the nominal value is defined differently.

		Symbol	Source (Note 1)	Value	Units (metric)
Mine:	Valentine Gold				
Location:	Central Western Newfoundland				
Product:	Gold				
Tailings streams:	1				
Date:	Dec 06, 2019				
Level of Study:	Pre-Feasibility Study				
Golder project #:	19130660-4000				

	Symbol	Source	Value	Units
Ore processing				
- Ore reserve *(design tonnage)*		Moose Mountain	35.00	Mt
- Planned annual mill throughput *(nominal production rate)*		Moose Mountain	4,000,000	t / y
- Mill availability *(% of the year that the mill is available to operate -usually 90 to 95%)*		assumed	92.5	%
- Factor of safety on the design value		assumed	1.15	-
Tailings				
- Tailings / ore ratio *(the difference is concentrate)*			1.000	-
- Percentages of tailings in each strean			100.0	%
- Slurry density of tailings discharge from thickener to the disposal facilities	S_2	Ausenco	65.0	% solids
- Specfic gravity of tailings particles	G_s	Ausenco	2.7	-
- Density of the process floid *(normally water)*	ρ_ω	Assumed	1.0	t / m^3
- Assumed deposited void ratio *(Void volume / solids volume)*	e	Assumed	0.9	-
- Degree of saturation *(volume of water / volume of voids)*		Assumed	100.0	%
Flows impacting the mill part of the flow model				
- Moisture content of the ore going into the mill *(% mositure / total mass of ore)*	ω_1	Assumed	2.0	%
- Slurry density of the concentrate *(% solids in total mass of concentrate)*	S_3	Assumed	100.0	% solids
- Clean (fresh) make-up water required in the mill *(% of total flow through the mill)*		Assumed	10.00	%
- Water lost in the mill to evaporation and spillage *(% of total flow through mill)*		Assumed	1.00	%
Water Treatment Plant Parameters				
- Operating months per year (April to November)		assumed	8	mo

Notes:

1 The sources of the information could be either the owner, other consultants or assumed by Golder

2 The sheet is protected except for the orange shaded cells. The password is simply Golder.

Input data are only required in the orange shaded cells. The values are then automatically linked to other relevanat sheets where the calculations are carried out.

2020-03-17

Sheet 4

Tailings Volume & Flow Calculations Associated with Processing Ore

		Indicator Letter	Indicator Symbol	Flow No. (Note 1)	Source or Calculation	Value	Units (metric)
Mine:	Valentine Gold						
Location:	Central Western Newfoundland						
Date:	Dec 06, 2019						
Project #:	19130660-4000						
Level of Study:	Pre-Feasibility Study						
Ore production							
- Ore reserve		A			Moose Mountain	35.00	Mt
- Nominal ore production	Planned annual	B			Moose Mountain	4,000,000	t / yr
	Monthly	C			B / 12	333,333	t / mo
	Daily	D			B / 365	10,959	t / day
- Life of mine		E			A / B	8.8	years
- Mill availability *(% of the year the mill is available to operate)*		F			assumed	92.5	%
- Factor of safety on the design value		G			assumed	1.15	-
- Design daily milling rate processing the ore		H			D / (F /100) x G	13,625	t / day
Tailings Production							
- Tailings / ore ratio		I			0	1.000	-
- Concentrate (product) per month		J			C - C x I	0	t / mo
- % of tailings to each stream		K			0	100	%
- Nominal tailings production	Total	L			A x I x (K / 100)	35.00	Mt
	Annual	M			B x I x (K / 100)	4,000,000	t / yr
	Monthly	N			C x I x (K / 100)	333,333	t / mo
	Daily	O			D x I x (K / 100)	10,959	t / day
- Design daily tailings production rate for dewatering, pumps and pipe lines		P			O / (F / 100) x G	13,625	t / day
- Taillings specific gravity			G_s		Ausenco	2.68	-
- Density of *the process flluid (normally water)*			ρ_ω		Assumed	1.00	t / m^3
- Assumed void ratio of the deposited tailings *(void vol. / solid tailings vol,)*			e		Assumed	0.90	-
- Dry density of deposited tailings			ρ_d		G_s x ρ_ω / (1 + e)	1.41	t / m^3
- Volume of deposited tailings *(based on nominal values)*	Total	Q			L / ρ_d	24.81	M-m^3
	Annual	R			M / ρ_d	2,835,821	m^3 / yr
	Monthly	S			N / ρ_d	236,318	m^3 / mo
	Daily	T			O / ρ_d	7,769	m^3 / day
Monthly nominal flows associated with processing the ore							
Water in the tailings being discharged from mill to tailings thickener							
- Discharge slurry density			S_1		Ausenco	65.0	% solids
- Volume of water discharged from the mill		U		P1	N / $(S/100)_1$ - N	179,487	m^3 / mo
Water in the tailings being discharge from the thickener to disposal							
- Discharge slurry density of tailings *(% solids in total mass of tailings)*			S_2		Ausenco	65.0	% solids
- Volume of water discharged with the tailings from the thickener		V			N / (S_2 / 100) - N	179,487	m^3 / mo
- Volume of water discharged from the thickener *to surface disposal*		W		P2 or P10	N / (S_2 / 100) - N	179,487	m^3 / mo
Overflow from thickener recycled to the mill		x		P3	U - V	0	m^3 / mo
Water retained in the deposited tailings							
- Degree of saturation *(volume of water / volume of voids)*			s		Assumed	100.00	%
- Water content of deposited tailings *(mass water / mass tailings)*			ω_1		(s / 100) x e / G_s x 100	33.6	%
- Volume of water retained in the deposited tailings *(not in mine backfill)*		Y		P4 or P11	N x (ω_1/100)	111,940	m^3 / mo
Moisture in the ore going into the mill.						111940.2985	
- Water content of the ore going into the mill *(mass water / mass of ore)*			ω_2		Assumed	2.0	%
- Volume of water entering the mill in the ore		Z		P5	C x (ω_2 / 100)	6,667	m^3 / mo
Water leaving the mill with the concentrate (product)							
- Slurry density of the concentrate *(% solids in total mass of concentrate)*			S_3		Assumed	100.0	% solids
- Volume of water leaving the mill in the concentrate		AA		P6	J / (S_3 / $100)_-$ J	0	m^3 / mo
Clean make-up water required in the mill from an external source							
- % of total flow through the mill *(assume total flow same as volume discharged with the tailings)*		BB			Assumed	10.0	%
- Clean make-up waterr required in the mill		CC		P7	U x (BB / 100)	17,949	m^3 / mo
Water lost in the mill to evaporation and spillage							
- % of total flow through the mill *(assume total flow same as volume discharged with the tailings)*		DD			Assumed	1.0	%
- Volume lost in the mill to evaporation and spillage		EE		P8	U x (DD / 100)	1,795	m^3 / mo
Make-up water that is required to run the mill from the tailings pond or elsewhere *(a positive no.)* **or excess water that cannot be recycled and has to be discharged to the environment** *(a negative no.)* **.**							
- Volume of water		FF		P9 or P12	P1 + P6 + P8 - P3 - P5 - P7	156,667	m^3 / mo

Notes:
1. Input data are are not required on this sheet. It is extracted from previous sheets and the calcuated
2. The sheet is protected. The password is simply Golder.
3. Flow numbers and colours correspond to the flows on the "Schematic Flow Sketch sheet.

181,282	Flows into the mill	**Must be equal**
181,282	Flows out of the mill	

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2020-03-17

Sheet 5
Precipitation, Factore Runoff, Floods & Evaporation Data

Mine:	Valentine Gold	**Location:**	Central Western Newfoundland	**Date:**	Dec 06, 2019
Project #:	19130660-4000	**Level of study:**	Pre-Feasibility Study	**Tailings streams**	1

Precipitation				**Factored Runoff** (Note 1)														
Month	**Annual selected for flow modelling (mm / yr)** →		**1236**	From natural ground		From prepared ground (around mill site etc.)		From ponds and wet tailings		From dry tailings beach		From waste rock & ore storage		From open pit		From membrane lined surfaces		Monthly runoff **(Note 3)**
	Mean	**Monthly Distribution (Note 2)**	**Precip-itation**	Runoff factor	Factored runoff used in the flow model	Runoff factor	Factored runoff used in the flow model	Runoff factor	Factored runoff used in the flow model	Runoff factor	Factored runoff used in the flow model	Runoff factor	Factored runoff used in the flow model	Runoff factor	Factored runoff used in the flow model	Runoff factor	Factored runoff used in the flow model	Expressed as a % of accumu-lation
	(mm)	(% of total)	(mm)		(mm)		(mm)		(mm)		(mm)		(mm)		(mm)		(mm)	(%)
Oct	97.5	7.9	97.5	0.72	70.2	0.85	82.9	1.00	97.5	0.40	39.0	0.50	48.7		0.0		0.0	100
Nov	111.8	9.0	111.8	0.72	80.5	0.85	95.0	1.00	111.8	0.40	44.7	0.50	55.9		0.0		0.0	50
Dec	123.1	10.0	123.1	0.72	88.6	0.85	104.6	1.00	123.1	0.40	49.2	0.50	61.5		0.0		0.0	0
Jan	122.0	9.9	122.0	0.72	87.8	0.85	103.7	1.00	122.0	0.40	48.8	0.50	61.0		0.0		0.0	0
Feb	98.1	7.9	98.1	0.72	70.6	0.85	83.4	1.00	98.1	0.40	39.2	0.50	49.0		0.0		0.0	0
Mar	95.0	7.7	95.0	0.72	68.4	0.85	80.7	1.00	95.0	0.40	38.0	0.50	47.5		0.0		0.0	0
April	85.7	6.9	85.7	0.72	61.7	0.85	72.8	1.00	85.7	0.40	34.3	0.50	42.8		0.0		0.0	50
May	86.6	7.0	86.6	0.72	62.3	0.85	73.6	1.00	86.6	0.40	34.6	0.50	43.3		0.0		0.0	100
June	87.8	7.1	87.8	0.72	63.2	0.85	74.6	1.00	87.8	0.40	35.1	0.50	43.9		0.0		0.0	100
July	95.3	7.7	95.3	0.72	68.6	0.85	81.0	1.00	95.3	0.40	38.1	0.50	47.6		0.0		0.0	100
Aug	123.0	9.9	123.0	0.72	88.6	0.85	104.5	1.00	123.0	0.40	49.2	0.50	61.5		0.0		0.0	100
Sept	110.4	8.9	110.4	0.72	79.5	0.85	93.8	1.00	110.4	0.40	44.2	0.50	55.2		0.0		0.0	100
TOTAL	**1236.3**	**100.0**	**1236.2**	**0.72**	**890.1**	**0.85**	**1050.8**	**1.00**	**1236.2**	**0.40**	**494.5**	**0.50**	**618.1**	**0.00**	**0.0**	**0.00**	**0.0**	

Evaporation (Note 4)					
Month	**Annual selected for flow modelling (mm / yr) [REF 1]** →		**475**	**Lake evaporation used in the flow model**	
	Pan (measured) or calculated lake evaporation				
	Mean month (from study)	**Monthly distribution**	**Value to which the factor is applied**	**Factor from pan to lake** (Note 5) **[REF 1]**	**Used in flow model**
	(mm)	(% of total)	(mm)		(mm)
Oct	48.0	7.11	34	0.67	22.6
Nov	4.6	0.68	3	0.67	2.2
Dec	0.0	0.00	0	0.67	0.0
Jan	0.0	0.00	0	0.67	0.0
Feb	0.0	0.00	0	0.67	0.0
Mar	0.0	0.00	0	0.67	0.0
April	12.3	1.82	9	0.67	5.8
May	75.8	11.24	53	0.67	35.8
June	124.7	18.48	88	0.67	58.8
July	163.3	24.21	115	0.67	77.0
Aug	148.8	22.06	105	0.67	70.2
Sept	97.2	14.40	68	0.67	45.8
TOTAL	**674.7**	**100.00**	**475.0**	**0.67**	**318.3**

Percnt Precipitation & evaporation in years that are wetter or dryer than the mean year (Expressed as factor of the mena)				
Annual Return Period	**Percipitation**		**Evaporation**	
Years	Wetter	Dryer	Wetter	Dryer
mean	1.0			
2				
5	1.01	0.78		
10	1.10	0.74		
25	1.21	0.69		
50	1.29	0.67		
100	1.37	0.66		
200	1.46	0.64		
1,000	**1.64**	**0.61**		

Storm Events		
Return period	**Precip-itation**	**Duration**
Years	(mm)	Hours
2	42.9	24
5	51.9	24
10	57.8	24
25	65.3	24
50	70.9	24
100	76.4	24
200		
500		
1000		
PMP		
Timmins Storm		

PMP - Probable maximum precipitation

NOTES:

1 The runoff factor is the % of the precipitation that runs off. It takes into account evapo-transpiration and infiltration. From natural ground it might be in the order of 20 to 70 % depending on the degree of ground saturation, the size of the rainfall and the time of the year. It will be greater from prepared surfaces and pit walls. For modeling it can be assumed that 100 % of the precipiation that falls on the pond and wet tailings beach ends in the pond. The runoff from a dry tailings beach is less depending on the tailings degree of saturation. Flow measurements have to be made to accurately correlate runoff with with precipitation.

2 For years that are wetter and dryer than the mean year, it is assumed that the monthly distribution of precipitation is the same as the distribution in the mean year.

3 A flow model must be able to account for winter snow accumulation by entering runoff as a percentage of the total accumulated for the month. For example if there is no runoff in the January, February and March and 100% runoff in April then the total winter's accumulation for the three months will enter the inflow side of the model in April. **For the flow model to function properly the input table must start and end in months when the runoff from precipitation and evaporation is 100% .**

4 "Pan evaporation" is a measured value. The evaporation that actually occurs from a water surface is called the "lake evaporation". It is normally about 70 % of the measured pan evaporation but this could vary depending on the climatic conditions and the time of year. Evaporation can also be calculated based on climatic conditions.

5 If calculated lake evaporation is used then the factor in the pan evaporation to lake evaporation colume is zero for each month.

6 Data are only inputted in the orange shaded cells (data input cells). Values used in the flow model.

7 The sheet is protected except for the orange shaded cells. The password is simply Golder.

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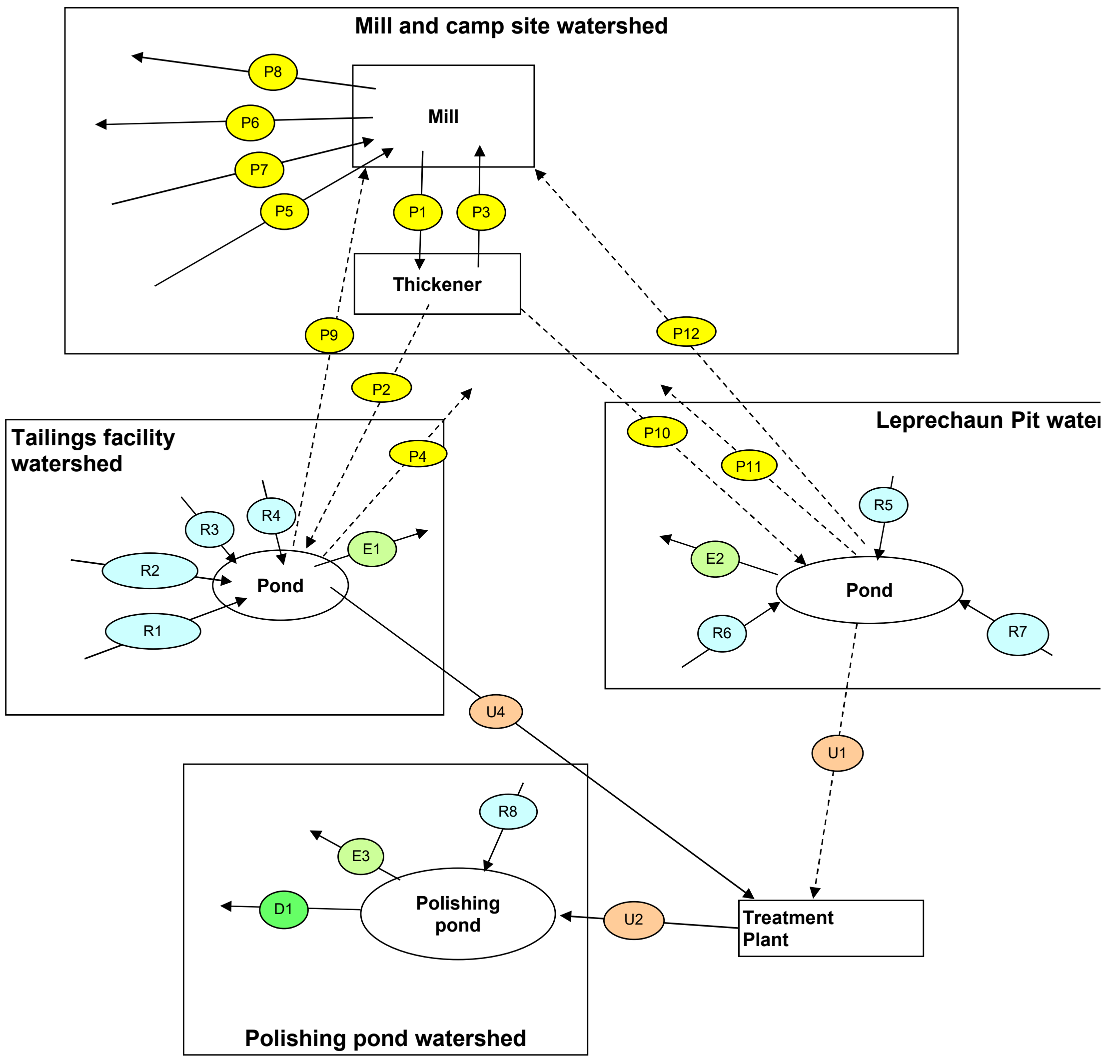
Sheet 6

Flow Logic Diagram

Mine:	Valentine Gold	**Location:**	Central Western Newfoundlan
Project #:	19130660-4000	**Level of study:**	Pre-Feasibility Study
Date:	Dec 06, 2019	**Tailings streams:**	1



Note: Input data are not required on this sheet. The sheet is protected. The password is simply Golder.

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Sheet 8
Collecting Watershed Areas

Mine:	Valentine Gold	**Location:**	Central Western Newfoundla
Project #:	19130660-4000	**Level of study:**	Pre-Feasibility Study
Date:	Dec 06, 2019	**Tailings streams:**	1

Watershed		**Sub Watersheds**			**Flow Number**
Facility	Area (ha)	Collecting area	% of total	(m^2)	
Tailings facility	223	Natural ground	10	223,000	R1
		Prepared ground	10	223,000	R2
		Pond & wet tailings	45	1,003,500	R3, E1
		Dry tailings beach	35	780,500	R4
		TOTAL	100	2,230,000	-
Leprechaun Pit	54	Prepared ground	20	107,200	R5
		Pond & wet tailings	45	241,200	R6, E2
		Dry tailings beach	35	187,600	R7,
		TOTAL	100	536,000	-
Polishing pond	5	Pond	100	50,000	R8, E3
		TOTAL	100	50,000	-
TOTAL	**281.60**				

Notes:

The sub-watersheds are subdivided by percentages which will change as the mine develops.

Data are only inputted into the orange shaded cells. The calculations are carried out the other cells and the relevant data is automantically transferred to other sheets. The sheet is protected except for the orange shaded cells. The password is simply Golder.

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Sheet 7
List of Flows

Area	Flow Number	Description	
Flows associated with the ore and tailings production (P)	P1	discharge from the mill to the tailings thickener	
	P2	discharge from thickener to tailings disposal facilty	
	P3	overflow from the thickener recycled to the mill	
	P4	water retained in the consolidated tailings mass	
	P5	Moisture going into the mill with the ore	
	P6	moisture leaving the mill in the concentrate	
	P7	fresh make-up water required in the mill	
	P8	losses in the mill to evaporation and spillage etc.	
	P9	other make-up water required in the mill from the TMF	
	P10	discharge from thickener to Leprechaun Pit	
	P11	water retained in the consolidated Leprechaun Pit tailings mass	
	P12	other make-up water required in the mill from the Leprechaun Pit	
Runoff (R)	R1	tailings facility	natural ground
	R2	tailings facility	prepared ground
	R3	tailings facility	precipitation on pond & wet tailings
	R4	tailings facility	dry tailings beach
	R5	Leprechaun Pit	prepared ground
	R6	Leprechaun Pit	precipitation on pond & wet tailings
	R7	Leprechaun Pit	dry tailings beach
	R8	polishing pond	precipitation on pond
Evaporation from ponds (E)	E1	tailings pond & wet tailings	
	E2	Leprechaun Pit tailings pond & wet tailings	
	E3	polishing pond after treatment	
Pump Flows Between Elements	U1	from pit pond to the treatment plant	
	U2	from treatment plant to polishing pond	
	U3	from tailings pond to treatment plant	
Discharge to environment (D)	D1	from the polishing pond to the environment	

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Sheet 9 - 1
Runoff from Precipitation

From cover sheet				
	Mine:	Valentine Gold	Location:	Central Western Newfoundland
	Project #:	19130660-4000	Level of study:	Pre-Feasibility Study
	Date:	Dec 06, 2019	Tailings streams:	1

Month	Factored Precipitation (from Sheet 12) (mm)							Monthly runoff expressed as % of the total accumu-lation (If less than 100% it is because of freeze-up)
	From natural ground	From prepared ground	From ponds and wet tailings	From dry tailings beach	From waste rock & ore storage	From open pit	From membrane lined surfaces	
Oct	70.2	82.9	97.5	39.0	48.7	0.0	0.0	100
Nov	80.5	95.0	111.8	44.7	55.9	0.0	0.0	50
Dec	88.6	104.6	123.1	49.2	61.5	0.0	0.0	0
Jan	87.8	103.7	122.0	48.8	61.0	0.0	0.0	0
Feb	70.6	83.4	98.1	39.2	49.0	0.0	0.0	0
Mar	68.4	80.7	95.0	38.0	47.5	0.0	0.0	0
April	61.7	72.8	85.7	34.3	42.8	0.0	0.0	50
May	62.3	73.6	86.6	34.6	43.3	0.0	0.0	100
June	63.2	74.6	87.8	35.1	43.9	0.0	0.0	100
July	68.6	81.0	95.3	38.1	47.6	0.0	0.0	100
Aug	88.6	104.5	123.0	49.2	61.5	0.0	0.0	100
Sept	79.5	93.8	110.4	44.2	55.2	0.0	0.0	100
TOTAL	890.1	1050.8	1236.2	494.5	618.1	0.0	0.0	

	Runoff Flow (m^3 / month)											
Runoff # (from Sheet 15)	R1	natural ground		R2	prepared ground		R3	precipitation on pond & wet tailings		R4	dry tailings beach	
Facility (from Sheet 15)	tailings facility			tailings facility			tailings facility			tailings facility		
Area (m^2) (from Sheet 16)	223,000			223,000			1,003,500			780,500		
Month	Available runoff (area x factored precipitation)	Accumulation left over each month (available runoff + previous months accumulation) - (available runoff + previous months accumulation) x monthy %	(Available runoff +previous monthly accumulation) x monthy %	Available runoff (area x factored precipitation)	Accumulation left over each month (available runoff + previous months accumulation) - (available runoff + previous months accumulation) x monthy %	(Available runoff +previous monthly accumulation) x monthy %	Available runoff (area x factored precipitation)	Accumulation left over each month (available runoff + previous months accumulation) - (available runoff + previous months accumulation) x monthy %	(Available runoff +previous monthly accumulation) x monthy %	Available runoff (area x factored precipitation)	Accumulation left over each month (available runoff + previous months accumulation) - (available runoff + previous months accumulation) x monthy %	(Available runoff +previous monthly accumulation) x monthy %
Oct	15,653	0	15,653	18,480	0	18,480	97,833	0	97,833	30,437	0	30,437
Nov	17,949	8,975	8,975	21,190	10,595	10,595	112,182	56,091	56,091	34,901	17,451	17,451
Dec	19,763	28,738	0	23,332	33,927	0	123,521	179,612	0	38,429	55,879	0
Jan	19,587	48,325	0	23,123	57,050	0	122,417	302,029	0	38,085	93,965	0
Feb	15,750	64,074	0	18,593	75,643	0	98,435	400,464	0	30,624	124,589	0
Mar	15,252	79,326	0	18,006	93,649	0	95,325	495,789	0	29,657	154,246	0
April	13,759	46,543	46,543	16,243	54,946	54,946	85,993	290,891	290,891	26,753	90,499	90,499
May	13,903	0	60,446	16,414	0	71,360	86,896	0	377,787	27,034	0	117,534
June	14,096	0	14,096	16,641	0	16,641	88,100	0	88,100	27,409	0	27,409
July	15,300	0	15,300	18,063	0	18,063	95,626	0	95,626	29,750	0	29,750
Aug	19,747	0	19,747	23,313	0	23,313	123,421	0	123,421	38,397	0	38,397
Sept	17,724	0	17,724	20,925	0	20,925	110,777	0	110,777	34,464	0	34,464
TOTAL	198,484		198,484	234,322		234,322	1,240,527		1,240,527	385,942		385,942

Notes:
1. Input data are not required on this sheet. The information is automatically transferred from other sheets or calculated on this sheet.
2. The blue shaded columns are the calculated monthly runoffs that are automatically transferred to the accumulation and summary sheets.
3. The sheet is protected. The password is simply Golder.
4. The table must start in a month with 100 % runoff - not a month when freezing temperatures prevents partial or zero runoff. The starting months must be the same on all sheets.

Sheet 9 - 2
Runoff from Precipitation

From cover sheet				
	Mine:	Valentine Gold	Location:	Central Western Newfoundland
	Project #:	19130660-4000	Level of study:	Pre-Feasibility Study
	Date:	Dec 06, 2019	Tailings streams:	1

Factored Precipitation (from Sheet 12) (mm)

Month	From natural ground	From prepared ground	From ponds and wet tailings	From dry tailings beach	From waste rock & ore storage	From open pit	From membrane lined surfaces	Monthly runoff expressed as % of the total accumu-lation (If less than 100% it is because of freeze-up)
Oct	70.2	82.9	97.5	39.0	48.7	0.0	0.0	100
Nov	80.5	95.0	111.8	44.7	55.9	0.0	0.0	50
Dec	88.6	104.6	123.1	49.2	61.5	0.0	0.0	0
Jan	87.8	103.7	122.0	48.8	61.0	0.0	0.0	0
Feb	70.6	83.4	98.1	39.2	49.0	0.0	0.0	0
Mar	68.4	80.7	95.0	38.0	47.5	0.0	0.0	0
April	61.7	72.8	85.7	34.3	42.8	0.0	0.0	50
May	62.3	73.6	86.6	34.6	43.3	0.0	0.0	100
June	63.2	74.6	87.8	35.1	43.9	0.0	0.0	100
July	68.6	81.0	95.3	38.1	47.6	0.0	0.0	100
Aug	88.6	104.5	123.0	49.2	61.5	0.0	0.0	100
Sept	79.5	93.8	110.4	44.2	55.2	0.0	0.0	100
TOTAL	890.1	1050.8	1236.2	494.5	618.1	0.0	0.0	

Runoff Flow (m^3 / month)

Runoff # (from Sheet 15)	R5	prepared ground		R6	precipitation on pond & wet tailings		R7	dry tailings beach		R8	precipitation on pond	
Facility (from Sheet 15)	Leprechaun Pit			Leprechaun Pit			Leprechaun Pit			polishing pond		
Area (m^2) (from Sheet 16)	107,200			241,200			187,600			50,000		
Month	Available runoff (area x factored precipitation)	Accumulation left over each month (available runoff + previous months accumulation) - (available runoff + previous months accumulation) x monthy %	(Available runoff +previous monthly accumulation) x monthy %	Available runoff (area x factored precipitation)	Accumulation left over each month (available runoff + previous months accumulation) - (available runoff + previous months accumulation) x monthy %	(Available runoff +previous monthly accumulation) x monthy %	Available runoff (area x factored precipitation)	Accumulation left over each month (available runoff + previous months accumulation) - (available runoff + previous months accumulation) x monthy %	(Available runoff +previous monthly accumulation) x monthy %	Available runoff (area x factored precipitation)	Accumulation left over each month (available runoff + previous months accumulation) - (available runoff + previous months accumulation) x monthy %	(Available runoff +previous monthly accumulation) x monthy %
Oct	8,883	0	8,883	23,515	0	23,515	7,316	0	7,316	3,510	0	3,510
Nov	10,186	5,093	5,093	26,964	13,482	13,482	8,389	4,194	4,194	4,024	2,012	2,012
Dec	11,216	16,309	0	29,689	43,171	0	9,237	13,431	0	4,431	6,443	0
Jan	11,116	27,425	0	29,424	72,595	0	9,154	22,585	0	4,392	10,835	0
Feb	8,938	36,363	0	23,660	96,255	0	7,361	29,946	0	3,531	14,366	0
Mar	8,656	45,019	0	22,912	119,167	0	7,128	37,074	0	3,420	17,786	0
April	7,808	26,414	26,414	20,669	69,918	69,918	6,430	21,752	21,752	3,085	10,436	10,436
May	7,890	0	34,304	20,886	0	90,804	6,498	0	28,250	3,117	0	13,553
June	8,000	0	8,000	21,176	0	21,176	6,588	0	6,588	3,161	0	3,161
July	8,683	0	8,683	22,985	0	22,985	7,151	0	7,151	3,431	0	3,431
Aug	11,207	0	11,207	29,665	0	29,665	9,229	0	9,229	4,428	0	4,428
Sept	10,059	0	10,059	26,626	0	26,626	8,284	0	8,284	3,974	0	3,974
TOTAL	112,643		112,643	298,171		298,171	92,764		92,764	44,503		44,503

Notes:

1. Input data are not required on this sheet. The information is automatically transferred from other sheets or calculated on this sheet.
2. The blue shaded columns are the calculated monthly runoffs that are automatically transferred to the accumulation and summary sheets.
3. The sheet is protected. The password is simply Golder.
4. The table must start in a month with 100 % runoff - not a month when freezing temperatures prevents partial or zero runoff. The starting month must be the same on all sheets.

2020-03-17

Sheet 10
Evaporation Losses

From cover sheet	Mine:	Valentine Gold	Location:	Central Western Newfoundland
	Project #:	19130660-4000	**Level of study:**	Pre-Feasibility Study
	Date:	Dec 06, 2019	**Tailings streams:**	1

Lake Evaporation (from Sheet 12) (mm)	Evaporation Losses (m^3 / month)											
	Location (from Sheet 15)	**tailings pond & wet tailings**	**Leprechaun Pit tailings pond & wet tailings**	**polishing pond after treatment**								**Total**
	Flow # (from Sheet 15)	**E1**	**E2**	**E3**								
	Area (m^2) (from Sheet 16) →	1,003,500	241,200	50,000								
22.6	**Oct**	22,712	5,459	1,132	0	0	0	0	0	0	0	29,302
2.2	**Nov**	2,182	525	109	0	0	0	0	0	0	0	2,816
0.0	**Dec**	0	0	0	0	0	0	0	0	0	0	0
0.0	**Jan**	0	0	0	0	0	0	0	0	0	0	0
0.0	**Feb**	0	0	0	0	0	0	0	0	0	0	0
0.0	**Mar**	0	0	0	0	0	0	0	0	0	0	0
5.8	**April**	5,800	1,394	289	0	0	0	0	0	0	0	7,482
35.8	**May**	35,890	8,626	1,788	0	0	0	0	0	0	0	46,304
58.8	**June**	59,012	14,184	2,940	0	0	0	0	0	0	0	76,136
77.0	**July**	77,313	18,583	3,852	0	0	0	0	0	0	0	99,748
70.2	**Aug**	70,455	16,934	3,510	0	0	0	0	0	0	0	90,900
45.8	**Sept**	46,001	11,057	2,292	0	0	0	0	0	0	0	59,350
318.3	**TOTAL**	**319,364**	**76,762**	**15,913**	**0**	**0**	**0**	**0**	**0**	**0**	**0**	**412,038**

Notes:

1 Input data are not required on this sheet. The information is automatically transferred from other sheets or calculated on this sheet.

2 The green shaded columns are the calculated monthly evaporations that are automatically transferred to accumulation and summary sheets.

3 The sheet is protected. The password is simply Golder.

4 The table must start in a month with 100 % runoff - not a month when freezing temperatures prevents partial or zero runoff. The starting month should be the same on all the sheets.

2020-03-17

Sheet 11 - 1

Accumulated Flow

Collecting Facility: Tailings Facility Pond

From cover sheet				
Mine:	Valentine Gold	Location:		Central Western Newfoundland
Project #:	19130660-4000	Level of study:		Pre-Feasibility Study
Date:	Dec 06, 2019	Tailings streams:		1

Initial Pond Volume: 590,000

Flow # (Sheet 15)	R1 natural ground	R2 prepared ground	R3 precipitation on pond & wet tailings	R4 dry tailings beach	E1 tailings pond & wet tailings	P2 discharge from thickener to tailings disposal facilty	P4 water retained in the consolidated tailings mass	$P9_{actual}$ other make-up water required in the mill from the TMF	$P9_{required}$ other make-up water required in the mill from the TMF	P9 Deficit	Net Inflow	Pond Volume Before Discharge to WTP	U3 from tailings pond to treatment plant	Pond Volume After Discharge to WTP
Source sheet (from Sheet 2)	9 - 1	9 - 2	9 - 2	9 - 2	10	4	4	4	4					
Month (Sheet 12)	Flows (m^3/month)													
Oct	15,653	18,480	97,833	30,437	-22,712	179,487	-111,940	-156,667	-156,667	0	50,572	640,572	83,809	556,763
Nov	8,975	10,595	56,091	17,451	-2,182	179,487	-111,940	-156,667	-156,667	0	1,809	558,572	83,809	474,763
Dec	0	0	0	0	0	179,487	-111,940	-156,667	-156,667	0	-89,120	385,643	0	385,643
Jan	0	0	0	0	0	179,487	-111,940	-156,667	-156,667	0	-89,120	296,523	0	296,523
Feb	0	0	0	0	0	179,487	-111,940	-156,667	-156,667	0	-89,120	207,403	0	207,403
Mar	0	0	0	0	0	179,487	-111,940	-156,667	-156,667	0	-89,120	118,284	0	118,284
April	46,543	54,946	290,891	90,499	-5,800	179,487	-111,940	-156,667	-156,667	0	387,960	506,243	83,809	422,434
May	60,446	71,360	377,787	117,534	-35,890	179,487	-111,940	-156,667	-156,667	0	502,117	924,551	83,809	840,742
June	14,096	16,641	88,100	27,409	-59,012	179,487	-111,940	-156,667	-156,667	0	-1,885	838,857	83,809	755,048
July	15,300	18,063	95,626	29,750	-77,313	179,487	-111,940	-156,667	-156,667	0	-7,694	747,354	83,809	663,545
Aug	19,747	23,313	123,421	38,397	-70,455	179,487	-111,940	-156,667	-156,667	0	45,304	708,848	83,809	625,039
Sept	17,724	20,925	110,777	34,464	-46,001	179,487	-111,940	-156,667	-156,667	0	48,770	673,809	83,809	590,000
TOTAL	**198,484**	**234,322**	**1,240,527**	**385,942**	**-319,364**	**2,153,846**	**-1,343,284**	**-1,880,000**	**-1,880,000**	**0**	**670,473**		**670,473**	

Notes:

1 Input data are not required on this sheet. The information is linked from other sheets.

2 The total "F" flow is automatically transferred to the other relevant sheets.

3 All the flows are summarized on the "Summary of Flows" sheet.

4 The sheet is protected. The password is simply Golder.

5 The table must start in a month with 100 % runoff - not a month when freezing temperatures prevent partial or zero runoff. The starting month should be the same as all the other sheets in the model.

Sheet 11 - 2

Accumulated Flow

Collecting Facility: Pit Mine Pond

From cover sheet				
	Mine:	Valentine Gold	**Location:**	**Central Western Newfoundland**
	Project #:	19130660-4000	**Level of study:**	Pre-Feasibility Study
	Date:	Dec 06, 2019	**Tailings streams:**	1

Initial Pond Volume: 0

Flow # (from Sheet 15) →	R5	R6	R7	E2	P10	P11	'P12$_{actual}$	'P12$_{required}$	P12 Deficit	Net Inflow	Pond Volume Before Discharge to WTP	U1	Pond Volume After Discharge to WTP
	prepared ground	precipitation on pond & wet tailings	dry tailings beach	Leprechaun Pit tailings pond & wet tailings	discharge from thickener to Leprechaun Pit	water retained in the consolidated Leprechaun Pit tailings mass	other make-up water required in the mill from the Leprechaun Pit	other make-up water required in the mill from the TMF				from pit pond to the treatment plant	
Source sheet (from Sheet 2) →	9 - 2	#REF!	#REF!	10	4	4	4	4					
Month (Sheet 12) ↓	**Flows (m^3/month)**												
Oct	8,883	23,515	7,316	-5,459	0	0	0	0	0	34,255	34,255	0	34,255
Nov	5,093	13,482	4,194	-525	0	0	0	0	0	22,245	56,500	0	56,500
Dec	0	0	0	0	0	0	0	0	0	0	56,500	0	56,500
Jan	0	0	0	0	0	0	0	0	0	0	56,500	0	56,500
Feb	0	0	0	0	0	0	0	0	0	0	56,500	0	56,500
Mar	0	0	0	0	0	0	0	0	0	0	56,500	0	56,500
April	26,414	69,918	21,752	-1,394	0	0	0	0	0	116,690	173,191	0	173,191
May	34,304	90,804	28,250	-8,626	0	0	0	0	0	144,732	317,923	0	317,923
June	8,000	21,176	6,588	-14,184	0	0	0	0	0	21,579	339,502	0	339,502
July	8,683	22,985	7,151	-18,583	0	0	0	0	0	20,235	359,738	0	359,738
Aug	11,207	29,665	9,229	-16,934	0	0	0	0	0	33,167	392,904	0	392,904
Sept	10,059	26,626	8,284	-11,057	0	0	0	0	0	33,912	426,817	0	426,817
TOTAL	**112,643**	**298,171**	**92,764**	**-76,762**	**0**	**0**	**0**	**0**	**0**	**426,817**		**0**	

Notes:

1. Input data are not required on this sheet. The information is linked from other sheets.
2. The total "F" flow is automatically transferred to the other relevant sheets.
3. All the flows are summarized on the "Summary of Flows" sheet.
4. The sheet is protected. The password is simply Golder.
5. The table must start in a month with 100 % runoff - not a month when freezing temperatures prevent partial or zero runoff. The starting mon same as all the other sheets in the model.

Sheet 11 - 3

Accumulated Flow

Collecting Facility: Water Treatment Plant

From cover sheet	Mine:	Valentine Gold	Location:	Central Western Newfoundland
	Project #:	19130660-4000	Level of study:	Pre-Feasibility Study
	Date:	Dec 06, 2019	Tailings streams:	1

Flow # (from Sheet 15) →	U1	U3	U2
	from pit pond to the treatment plant	from tailings pond to treatment plant	from treatment plant to polishing pond
Source sheet (from Sheet 2) →	11 - 2	11 - 1	
Month (Sheet 12) ↓	Flows (m^3/month)		
Oct	0	83,809	83,809
Nov	-	83,809	83,809
Dec	-	-	-
Jan	-	-	-
Feb	-	-	-
Mar	-	-	-
April	-	83,809	83,809
May	0	83,809	83,809
June	0	83,809	83,809
July	0	83,809	83,809
Aug	0	83,809	83,809
Sept	0	83,809	83,809
TOTAL	0	670,473	670,473

Notes:

1. Input data are not required on this sheet. The information is linked from other sheets.
2. The total "F" flow is automatically transferred to the other relevant sheets.
3. All the flows are summarized on the "Summary of Flows" sheet.
4. The sheet is protected. The password is simply Golder.
5. The table must start in a month with 100 % runoff - not a month when freezing temperatures prevent partial or zero runoff. The starting month should be the same as all the other sheets in the model.

Sheet 11 - 4
Accumulated Flow

Collecting Facility: Polishing Pond

From cover sheet	Mine:	Valentine Gold	Location:	Central Western Newfoundland
	Project #:	19130660-4000	Level of study:	Pre-Feasibility Study
	Date:	Dec 06, 2019	Tailings streams:	1

Initial Pond Volume: 44,000

Flow # (from Sheet 15)	R8 precipitation on pond	U2 from treatment plant to polishing pond	E3 polishing pond after treatment	Net Inflow	Pond Volume Before Discharge to Environment	D1 from the polishing pond to the environment	Pond Volume After Discharge to WTP
Source sheet (from Sheet 2)	9 - 2	11 - 3	10				
Month (Sheet 12)	Flows (m³/mont						
Oct	3,510	83,809	-1,132	86,187	130,187	86,187	44,000
Nov	2,012	83,809	-109	85,713	129,713	85,713	44,000
Dec	0	0	0	0	44,000	0	44,000
Jan	0	0	0	0	44,000	0	44,000
Feb	0	0	0	0	44,000	0	44,000
Mar	0	0	0	0	44,000	0	44,000
April	10,436	83,809	-289	93,956	137,956	93,956	44,000
May	13,553	83,809	-1,788	95,574	139,574	95,574	44,000
June	3,161	83,809	-2,940	84,029	128,029	84,029	44,000
July	3,431	83,809	-3,852	83,387	127,387	83,387	44,000
Aug	4,428	83,809	-3,510	84,726	128,726	84,726	44,000
Sept	3,974	83,809	-2,292	85,491	129,491	85,491	44,000
TOTAL	44,503	670,473	-15,913	699,064		699,064	

Notes:

1. Input data are not required on this sheet. The information is linked from other sheets.
2. The total "F" flow is automatically transferred to the other relevant sheets.
3. All the flows are summarized on the "Summary of Flows" sheet.
4. The sheet is protected. The password is simply Golder.
5. The table must start in a month with 100 % runoff - not a month when freezing temperatures prevent partial or zero ru same as all the other sheets in the model.

Deterministic Mine Site Wide Flow (water balance) Model

25-year Wet Climate Conditions

Mine:	Valentine Gold
Owner(s):	Marathon Gold Corporation
Operator:	Marathon Gold Corporation
Location:	Central Western Newfoundland
Product:	Gold
Type of mine:	Open Pit
Tailings streams:	1
Date:	Dec 06, 2019
Level of study:	Pre-Feasibility Study
Golder Project #:	19130660-4000

Notes:

1 Data are only inputted into the orange shaded cells. The relevant data are automatically transferred to other sheets.

2 Each sheet is protected except for the orange shaded cells. The password is simply Golder.

3 The red "tailings streams" at the top of each sheet can be removed from all the sheets by merely deleting it on this cover sheet.

Golder Associates

2020-03-17

Sheet 11 - 1

Accumulated Flow

Collecting Facility: Tailings Facility Pond

From cover sheet				
	Mine:	Valentine Gold	**Location:**	**Central Western Newfoundland**
	Project #:	19130660-4000	**Level of study:**	Pre-Feasibility Study
	Date:	Dec 06, 2019	**Tailings streams:**	1

Initial Pond Volume: 590,000

Flow # (Sheet 15) →	R1	R2	R3	R4	E1	P2	P4	$P9_{actual}$	$P9_{required}$	P9 Deficit	Net Inflow	Pond Volume <u>Before</u> Discharge to WTP	U3	Pond Volume <u>After</u> Discharge to WTP
	natural ground	prepared ground	precipitation on pond & wet tailings	dry tailings beach	tailings pond & wet tailings	discharge from thickener to tailings disposal facilty	water retained in the consolidated tailings mass	other make-up water required in the mill from the TMF	other make-up water required in the mill from the TMF				from tailings pond to treatment plant	
Source sheet (from Sheet 2) →	9 - 1	9 - 2	9 - 2	9 - 2	10	4	4	4	4					
Month (Sheet 12) ↓	**Flows (m^3/month)**													
Oct	18,935	22,354	118,347	36,819	-22,712	179,487	-111,940	-156,667	-156,667	0	84,624	674,624	137,781	536,842
Nov	10,856	12,816	67,852	21,110	-2,182	179,487	-111,940	-156,667	-156,667	0	21,332	558,175	137,781	420,393
Dec	0	0	0	0	0	179,487	-111,940	-156,667	-156,667	0	-89,120	331,273	0	331,273
Jan	0	0	0	0	0	179,487	-111,940	-156,667	-156,667	0	-89,120	242,154	0	242,154
Feb	0	0	0	0	0	179,487	-111,940	-156,667	-156,667	0	-89,120	153,034	0	153,034
Mar	0	0	0	0	0	179,487	-111,940	-156,667	-156,667	0	-89,120	63,914	0	63,914
April	56,301	66,467	351,884	109,475	-5,800	179,487	-111,940	-156,667	-156,667	0	489,208	553,122	137,781	415,340
May	73,120	86,322	457,000	142,178	-35,890	179,487	-111,940	-156,667	-156,667	0	633,610	1,048,950	137,781	911,169
June	17,052	20,130	106,573	33,156	-59,012	179,487	-111,940	-156,667	-156,667	0	28,779	939,948	137,781	802,166
July	18,508	21,850	115,676	35,988	-77,313	179,487	-111,940	-156,667	-156,667	0	25,590	827,756	137,781	689,974
Aug	23,888	28,201	149,299	46,448	-70,455	179,487	-111,940	-156,667	-156,667	0	88,261	778,236	137,781	640,454
Sept	21,441	25,312	134,005	41,690	-46,001	179,487	-111,940	-156,667	-156,667	0	87,327	727,781	137,781	590,000
TOTAL	**240,101**	**283,453**	**1,500,634**	**466,864**	**-319,364**	**2,153,846**	**-1,343,284**	**-1,880,000**	**-1,880,000**	**0**	**1,102,251**		**1,102,251**	

Notes:

1. Input data are not required on this sheet. The information is linked from other sheets.
2. The total "F" flow is automatically transferred to the other relevant sheets.
3. All the flows are summarized on the "Summary of Flows" sheet.
4. The sheet is protected. The password is simply Golder.
5. The table must start in a month with 100 % runoff - not a month when freezing temperatures prevent partial or zero runoff. The starting month should be the same as all the other sheets in the model.

APPENDIX D

Slope Stability

TECHNICAL MEMORANDUM

DATE 30 June 2020 **Project No.** 19130660

TO Robbert Borst
Marathon Gold

FROM Matt Soderman and Peter Merry **EMAIL msoderman@golder.com**

VALENTINE GOLD PROJECT – TAILINGS MANAGEMENT FACILITY SLOPE STABILITY (REVISON 0)

1.0 INTRODUCTION

This appendix summarizes the stability analyses of the proposed Tailings Management Facility (TMF) perimeter dam slopes. Stability analyses were carried out in support of the prefeasibility design of the TMF. Limited subsurface geotechnical information is available for the TMF location as a specific site investigation has not yet been carried out. It is recommended that a geotechnical investigation be undertaken at the TMF during the next phase (feasibility study level) of the project. The preliminary stability analyses herein include a sensitivity analysis of the foundation soil strength parameters to provide guidance on the dam design and should be updated based on the results of future geotechnical investigation and laboratory testing.

2.0 METHOD OF ANALYSIS

Two-dimensional limit equilibrium slope stability analyses were performed using the commercially available program SLOPE/W, developed by GEOSLOPE International Ltd., employing the Morgenstern Price method of analysis. Morgenstern Price is a general method of slices which is based on equilibrium of forces and moments acting on each slice of soil mass above the potential failure surface. For all analyses, the Factors of Safety of numerous potential failure surfaces were computed in order to establish the minimum Factor of Safety. The Factor of Safety is defined as the ratio of the forces tending to resist failure to the driving forces tending to cause failure. The software was used to locate the most critical failure surfaces and determine the lowest factor of safety (FOS) using a various (circular and non-circular) failure mechanism.

2.1 Analyzed Sections

The TMF dam will be developed using the downstream construction method in stages which will require incremental raising of the dam crest from a start-up configuration to a final ultimate elevation as outlined in Table 1 below. The dam profile features a non-uniform crest which slopes down from the right (west) abutment for approximately half the length of the dam after which it remains flat (constant elevation) to the left (east) abutment.

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Table 1: TMF Dam Staging

Description	Unit	Stage 1A (Start-up)	Stage 1B	Stage 2	Stage 3	Stage 4 (Ultimate)
Year of Construction	Year	0	2	3	5	7
Year of Operation	Year	1-2	3	4-5	6-7	8-9
Dam Crest Elevation	masl	385.7 – 373.0	388.8 – 375.0	394.9 – 382.0	402.3 – 386.0	408.3 – 390.5

Stability analyses were conducted for one representative design section, namely Section A. The section represents a location where the dam height is the greatest (approximately 49 m at the Stage 4 – Ultimate Configuration) and where the existing ground slopes relatively steeply towards the river, which is considered to be critical to the stability of the dam slopes. Figure D1 shows the approximate location of Section A on plan with respect to the alignment of the ultimate TMF dam.

The model section was developed based on the typical design section for the dam. An overall upstream slope of 3.5H:1V and downstream slope of 2H:1V was used in the analysis. The proposed HDPE liner on the upstream slope of the dam was not included in the model as it will have negligible impact on the stability analysis. The existing ground generally slopes towards the Victoria River from the ridge providing natural containment at the north limit of the TMF.

2.2 Foundation Stratigraphy

No geotechnical investigations have been carried out in the specific footprint area of the proposed TMF. Based on review of available surface geology mapping for the project area (Smith, 2011), the dominant subsurface material in the TMF footprint is glacial till occurring mainly as hummocky and blanket deposits with thicknesses up to 15 m or as thin discontinuous veneer (typically less than 1.5 m thick) overlying bedrock. The area of high ground along the crest of the ridge at the north limit of the TMF is characterized by bedrock outcrop either exposed above the till veneer or concealed by vegetation. Bogs are present in poorly drained areas.

The foundation stratigraphy developed for the representative design section modelled in the slope stability analyses was developed based findings from limited investigation undertaken at the overall Valentine Gold mine site as part of a baseline hydrogeological characterization study carried out by Gemtec Consulting Engineers and Scientists Ltd. (GEMTEC) in October of 2019. Standpipe Monitoring wells were installed in two boreholes drilled in proximity to the TMF footprint, namely borehole MW-2 and MW-4. Boreholes MW-2 and MW-4 were advanced to depths of approximately 5.6 m and 7.5 m, respectively and were terminated in bedrock following coring.

At both locations, a layer of granular till comprising silty sand and gravel containing cobbles and boulders was encountered immediately below ground surface and overlying bedrock, and ranged in thickness from about to 2.7 m to 4.5 m. Standard Penetration Test (SPT) ‘N’ values measured in the till ranged from 8 blows to 55 blows per 0.3 m of penetration suggesting a loose to very dense level of compactness. No peat / organics were logged but are certainly present (based on site visit observations) at varying thicknesses at the site. The groundwater level measured within the monitoring wells (50 mm dia. PVC standpipe piezometers screened across both the till and bedrock layers) immediately following well installation ranged from a depth of 0.04 m above ground surface (suggesting artesian conditions) to 1.2 m below ground surface.

The foundation stratigraphy used for model Section A is a representative simplified model which comprises a 5 m-thick layer of granular till overlying bedrock. The bedrock surface has been assumed to be coincident with the existing ground. It was assumed that peat / organics will be completely stripped from within the dam footprint and would remain downstream of the dam (assumed thickness of 0.3 m). Peat on the upstream side of the dam within the tailings basin will consolidate under the loading stress of the overlying tailings and was ignored in the model section.

2.3 Geotechnical Parameters

From the available information, it has been assumed that the till is non-cohesive, dilative material which will not experience excess porewater generation during construction and effective stress parameters were employed in the analyses (drained conditions). The construction materials are granular, free draining materials and effective stress parameters were also used in the analyses. Geotechnical parameters were estimated from correlations based on the results of the in-situ SPTs recorded in the GEMTEC boreholes (till only) and our engineering judgement based on precedent experience with similar soils / fill materials. The geotechnical parameters used in the stability analyses for the TMF dam are summarized in Table 2. It is noted that given the lack of data and uncertainty surrounding the foundation conditions in the TMF, a sensitivity analysis was carried out by varying the strength of the foundation material, as discussed further in Section 3.0.

Table 2: Summary of Soil Properties used in Slope Stability Analyses

Soil Deposits used in Analyses at Simplified Representative Section A	Bulk Unit Weight, γ (kN/m^3)	Effective Friction Angle, φ' (°)	Cohesion, c' (kPa)	Undrained Shear Strength, s_u (kPa)
Fill / Construction Materials				
Tailings	19	29	-	-
Filter (sand)	21	32	-	-
Transition (sand and gravel)	21	34	-	-
Waste rock fill	22	40	-	-
Foundation Materials				
Peat	14	26	-	-
Granular Till	20	32[1.]	-	-
Bedrock	Set as impenetrable in the model			

Note: 1. A sensitivity analysis was carried out by varying to the shear strength of the till. Refer to Section 3.0 for discussion and results.

2.4 Loading Conditions

The following conditions were simulated for the analysis:

- End of Construction (EoC) condition: Following completion of construction of the start-up dam or subsequent raises wherein no tailings and / or water is impounded against the newly constructed portion of the dam
- Long-term condition: Steady state seepage conditions considering the TMF pond at its normal operating water level (ex. El. 388.5 m for the ultimate dam).

Both static and pseudo-static conditions were analyzed. It was assumed that native overburden and dam fill materials are not susceptible to liquefaction at any stages of the TMF dam's life assuming any surficial unsuitable / very loose to loose soils will be removed during foundation preparation.

A rapid drawdown loading scenario was not considered in the analysis as any leakage through the upstream HDPE liner is expected to drain quickly through the downstream rockfill zone of the embankment and a steady state seepage regime with an elevated piezometric level within the fill is not likely to be sustained.

2.5 Target Factors of Safety

In accordance with the dam classification methodology presented in the CDA Dam Safety Guidelines (CDA, 2013, 2014), the proposed TMF dams for the project have been provisionally classified as a "Very High" consequence of failure during the operations.

The target FOS required the stability of the upstream and downstream slopes of the TMF dams, in accordance with the CDA guidelines are provided in Table 3.

Table 3: Target Factors of Safety for Slope Stability (CDA, 2014)

Loading Condition	Minimum Factor of Safety	Slope
Static Assessment		
During or at end of construction	1.3	Upstream and downstream
Long term (steady state seepage, normal reservoir level)	1.5	Downstream
Seismic Assessment		
Pseudo-static	1.0	Downstream

2.6 Phreatic Surface

For most cases, two piezometric levels were applied to the soil layers in the model. Upstream of the HDPE liner, the phreatic surface in the tailings was modelled at the same elevation as the TMF pond, except for Stage 1A at the end of construction when no TSF pond is present (in this case, one piezometric level located at the existing ground surface was modelled). Downstream of the HDPE liner, a second phreatic surface was assumed to be coincident with the existing ground surface, as suggested from groundwater level reading from the existing nearby GEMTEC monitoring well MW-4 and based on the free draining properties of the granular filter, transition and rockfill materials.

2.7 Seismicity

The Earthquake Design Ground Motion (EDGM) is defined as the level of earthquake ground motion at the location of the dam for which a dam structure is designed and evaluated. The suggested annual exceedance probability (AEP) earthquake level for the EDGM is based on the dam consequence classifications and phase of TMF as summarized in the CDA guidelines (CDA, 2014). For a mining dam in the Construction, Operation and Transition phases of its life which is classified as having a "Very High" consequence of failure, the CDA guidelines recommend a design earthquake corresponding to halfway between the 1:2,475 year event and the 1:10,000 year event or Maximum Credible Earthquake (MCE).

A site-specific hazard assessment has not been carried out for the site. The 2015 National Building Code of Canada (NBCC) provides seismic hazard information relating to a maximum AEP for seismic “Site Class C” with an average shear wave velocity, V_s, of 450 m/s for probabilities of 1:2,475 or higher, and are attached to this appendix and summarized in the table below.

Table 4: Peak Ground Acceleration values for the Valentine Gold TMF site (NBCC, 2015)

Annual Exceedance Probability (AEP)	1:100	1:475	1:1,000	1:2,475
PGA (g)	0.006	0.019	0.028	0.046

Seismic hazard values for probabilities lower than 1:2,475 years are not available in the current NBCC. Extrapolation of the hazard model to lower probability, though possible (NBCC recommends log-log scale extrapolation), represents a degree of uncertainty and may be unreliable. However, at this stage of the project, extrapolation was deemed appropriate. The PGA for design was estimated by extrapolation to be approximately 0.098 *g* for seismic “Site Class C”, as shown in Figure D11. The foundation conditions at site are assumed to be consistent with “Site Class C” (V_s between 360 m/s and 760 m/s) therefore no amplification factor is suggested.

2.8 Pseudo-Static Analysis

The stability of the TMF dam against dynamic loads was analysed using the pseudo-static approach considering a PGA of 0.076 *g* corresponding to the recommended guidance from CDA for a “Very High” consequence of failure classification (halfway between the 1:2,475 year event and the 1:10,000 year event or MCE). The horizontal seismic loading co-efficient of ½ of the PGA (0.038 *g*) was used in the analyses as per the method suggested by Hynes-Griffin and Franklin (Hynes-Griffin & Franklin, 1984).

It is assumed that the materials are not susceptible to potential strength degradation from cyclic loading during a seismic event given their assumed compact to very dense (dilatant) state and free-draining, granular nature.

3.0 STABILITY ANALYSES RESULTS

For each of the loading conditions, a number of failure surfaces were simulated to determine the most critical failure mode and the associated lowest calculated FoS. Table 5 below summarizes the minimum calculated FoS calculated for each loading condition.

Table 5: Stability Assessment Factors of Safety

Dam Stage	Loading Condition	Target Minimum FOS	Minimum Calculated FOS	Figure No.
Stage 1A – Start-Up	End of Construction - Static	1.3	2.32 (U/S)	D2
			1.58 (D/S)	D3
	Long-term steady state - Static	1.5	1.57 (D/S)	D4
	Pseudo-static (PGA=0.038 *g*)	1.0	1.44 (D/S)	D5
Stage 4 – Ultimate	End of Construction - Static	1.5[1.]	3.01 (U/S)	D6
			1.59 (D/S)	D7
	Long-term steady state - Static	1.5	1.55 (D/S)	D8
	Pseudo-static (PGA=0.038 *g*)	1.0	1.42 (D/S)	D9

Note: 1. A FoS of 1.5 was selected for the End of Construction phase for Stage 4 given that tailings and water are impounded in the TMF during staged raise construction.

The calculated FoS against potential slope failure for each of the loading conditions for the analysed Section A and geotechnical parameters assumed adequately satisfy the target minimum FoS requirements recommended in the CDA guidelines.

However, due to limited subsurface information and lack of advanced laboratory testing to define the shear strength and behaviour of the foundation and embankment fill materials, a sensitivity analysis was carried out whereby the effective stress strength parameters of the till foundation soil were varied to evaluate the impact on the FoS against deep seated failure for a given geometry (i.e. 5 m thick till layer, typical dam section geometry, piezometric conditions). The range of strength was selected based on the results from the GEMTEC investigations carried out nearby as well as engineering judgement based on experience with similar till materials. A summary of the sensitivity analysis for static global stability in relation to the minimum target FoSs is presented graphically on Figure D10. Based on the results of the sensitivity analysis, the following preliminary conclusions can be made:

- For the range of strengths analyzed, the minimum target FoS of 1.3 for the End of Construction loading condition is adequately satisfied.
- A minimum drained strength for the till foundation soil of about 31° to 32° would be required to exceed the minimum target FoS of 1.5 for the long-term static loading condition.
- Should the TMF foundation comprise very loose to loose non-cohesive foundation soils, measures may be required to mitigate slope instability (ex. flatter dam slopes or implementation of toe berms).

The analyses should be updated at the next phase of the project and following completion of a geotechnical investigation and laboratory testing campaign at the TMF site location. For example, if very loose to loose non-cohesive foundation soils are encountered, a liquefaction assessment should be carried out, or if cohesive soils are encountered, undrained conditions should be considered in the analysis.

4.0 CLOSURE

We trust the above meets your present requirements. If you have any questions or comments, please contact the undersigned.

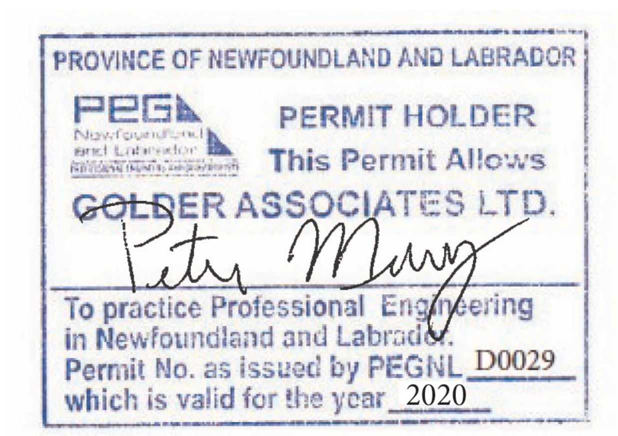
Golder Associates Ltd.

Matt Soderman, P.Eng. (ON)
Project Engineer

Peter Merry, P.Eng. (NL)
Project Director

MS/WPM/PA/hp

https://golderassociates.sharepoint.com/sites/117078/project files/5 technical work/4000 pre-feasibility design/geotechnical/1. slope stability/19130660 valentine gold tmf slope stability 30jun_20.docx



REFERENCES

Canadian Dam Association (CDA), 2014, “Technical Bulletin: Application of Dam Safety Guidelines to Mining Dams”, published by Canadian Dam Association.

GEMTEC, 2019, “Preliminary Summary, Baseline Hydrogeological Characterization Study, Valentine Lake Project, Central Newfoundland”, draft memo submitted to Marathon Gold Corporation dated December 4, 2019.

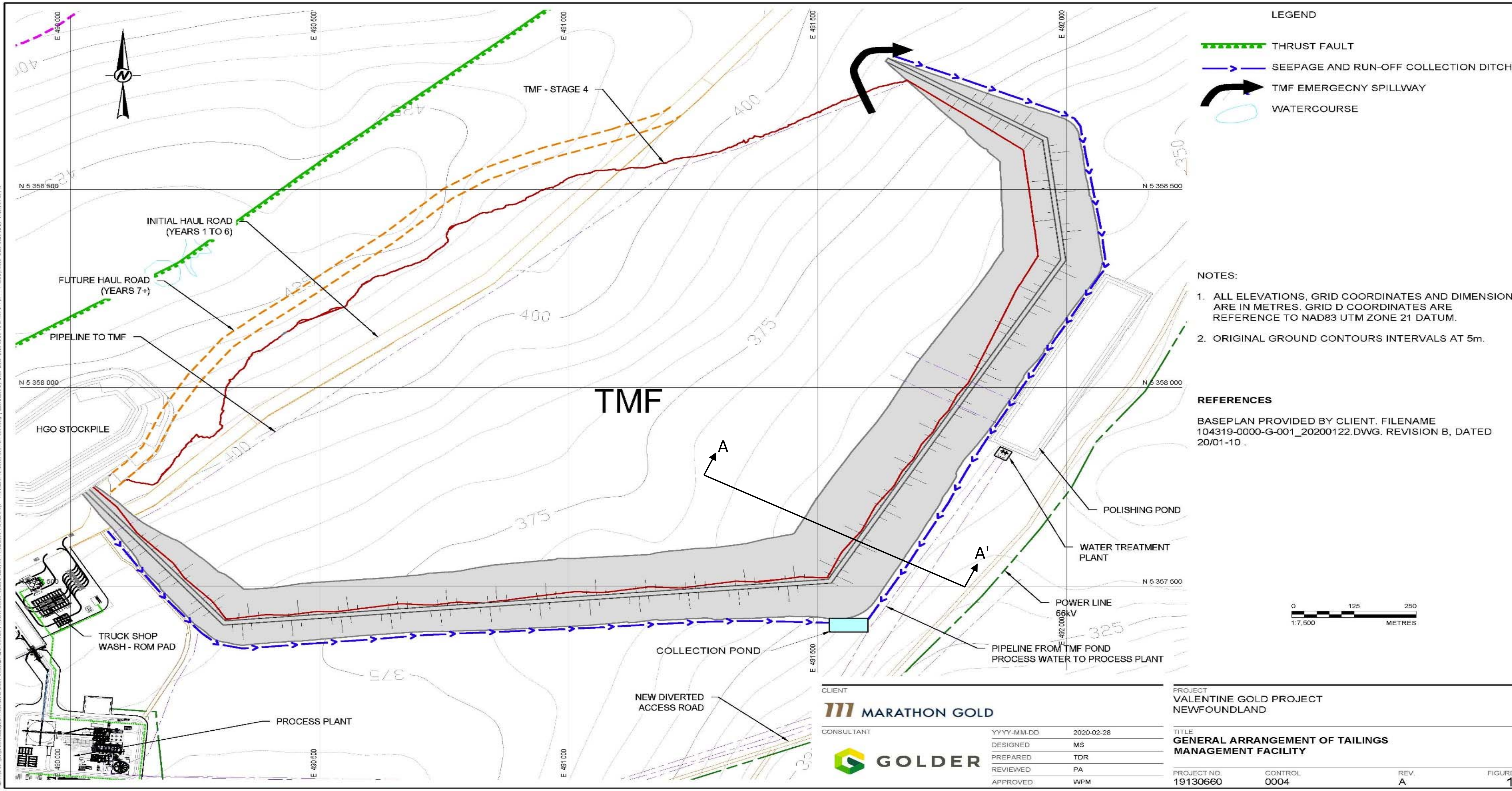
Hynes-Griffin ME, Franklin AG, 1984, “Rationalizing the seismic coefficient method. U.S. Army Corps of Engineers Waterways Experiment Station, Vicksburg, Mississippi”, Miscellaneous Paper GL-84-13, 21 pp.

Smith, J.S., 2011, “Surficial geology of the Victoria Lake map area (NTS 12A/06)”, Geological Survey, Department of Natural Resources, Government of Newfoundland and Labrador, Map 2011-03, Open File 12A/06/1509.

National Building Code Canada (NBCC), 2015, “National Building Code Seismic Hazard Calculation”, website “http://www.earthquakescanada.ca/hazard-alea/interpolat/index_2015-en.php” accessed January 23, 2020.

ATTACHMENT A

Figures



CLIENT **MARATHON GOLD CORPORATION**		PROJECT **Valentine Gold Pre-feasibility Study**	
CONSULTANT GOLDER	YYYY-MM-DD: 2020-06-30 DESIGNED: PREPARED: HM REVIEWED: MS APPROVED: PM	TITLE **Slope Stability Location - Section A-A'**	

TITLE	PHASE	REV	FIGURE
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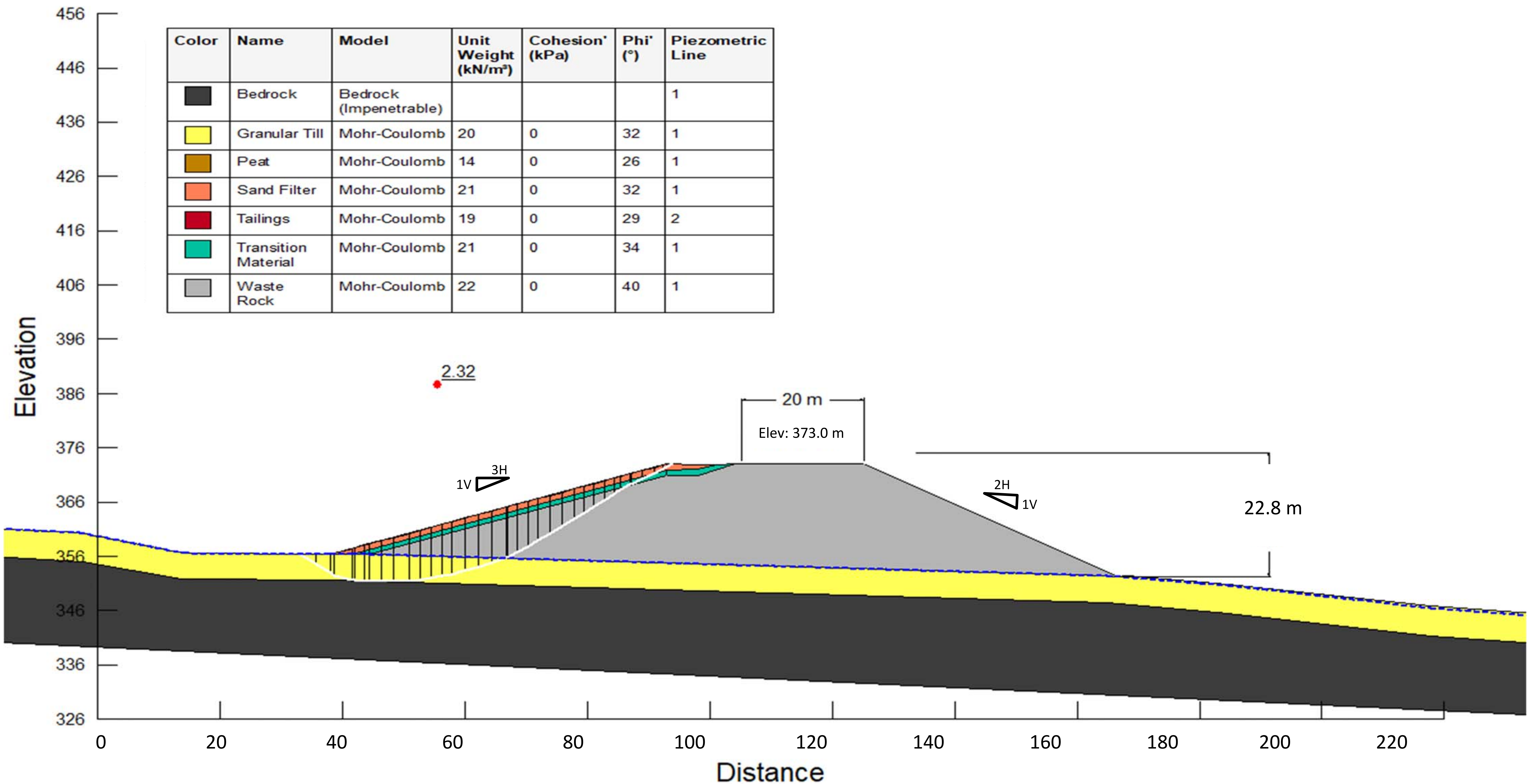

Color	Name	Model	Unit Weight (kN/m³)	Cohesion' (kPa)	Phi' (°)	Piezometric Line
	Bedrock	Bedrock (Impenetrable)				1
	Granular Till	Mohr-Coulomb	20	0	32	1
	Peat	Mohr-Coulomb	14	0	26	1
	Sand Filter	Mohr-Coulomb	21	0	32	1
	Tailings	Mohr-Coulomb	19	0	29	2
	Transition Material	Mohr-Coulomb	21	0	34	1
	Waste Rock	Mohr-Coulomb	22	0	40	1

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Valentine Gold Pre-feasibility Study

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GOLDER

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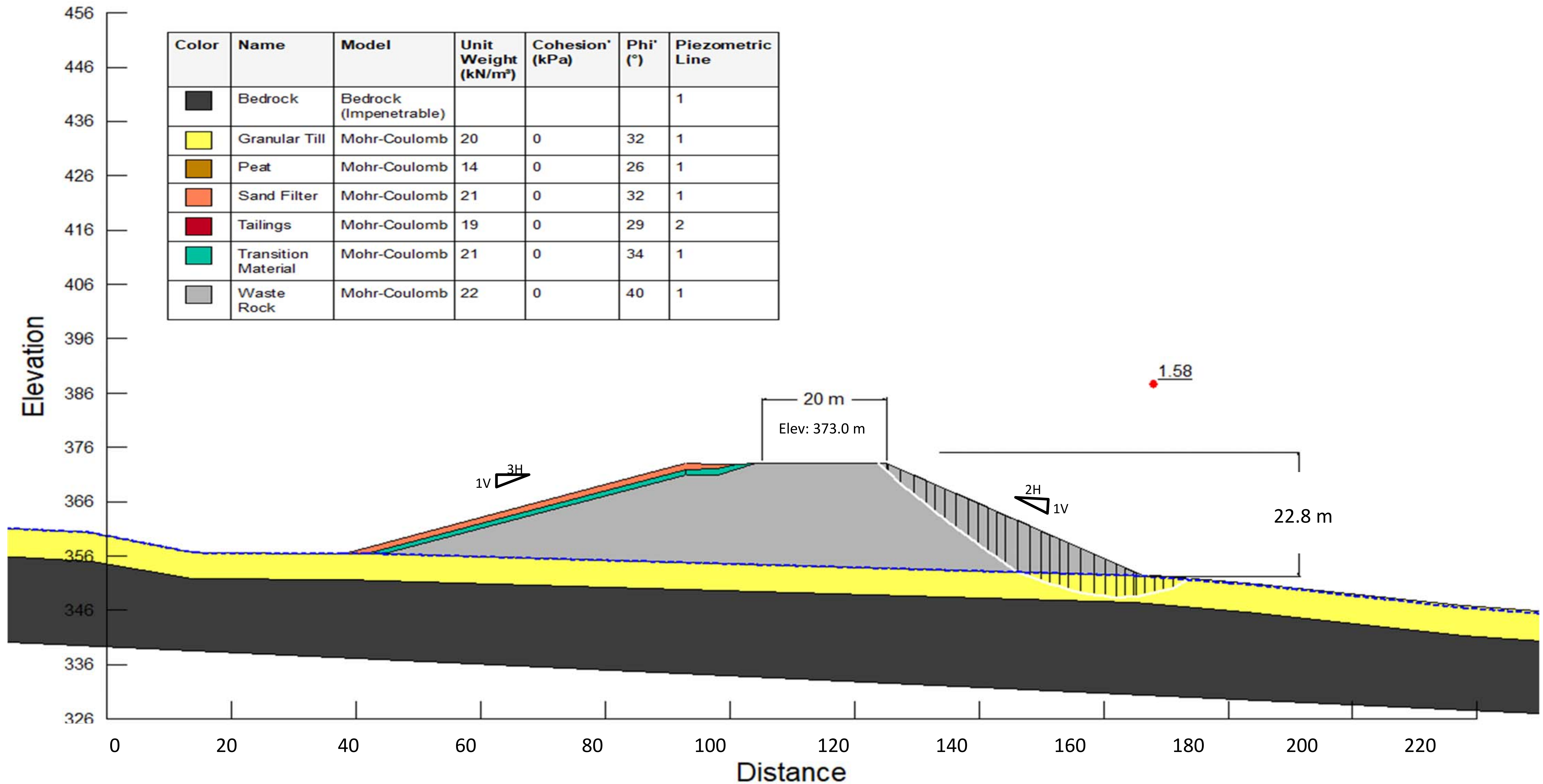
PREPARED	HM
REVIEWED	MS
APPROVED	PM

TITLE
Stage 1A End of Construction - Static U/S

TITLE	PHASE	REV	FIGURE
19130660	**4000**	**B**	**D2**



Color	Name	Model	Unit Weight (kN/m³)	Cohesion' (kPa)	Phi' (°)	Piezometric Line
	Bedrock	Bedrock (Impenetrable)				1
	Granular Till	Mohr-Coulomb	20	0	32	1
	Peat	Mohr-Coulomb	14	0	26	1
	Sand Filter	Mohr-Coulomb	21	0	32	1
	Tailings	Mohr-Coulomb	19	0	29	2
	Transition Material	Mohr-Coulomb	21	0	34	1
	Waste Rock	Mohr-Coulomb	22	0	40	1

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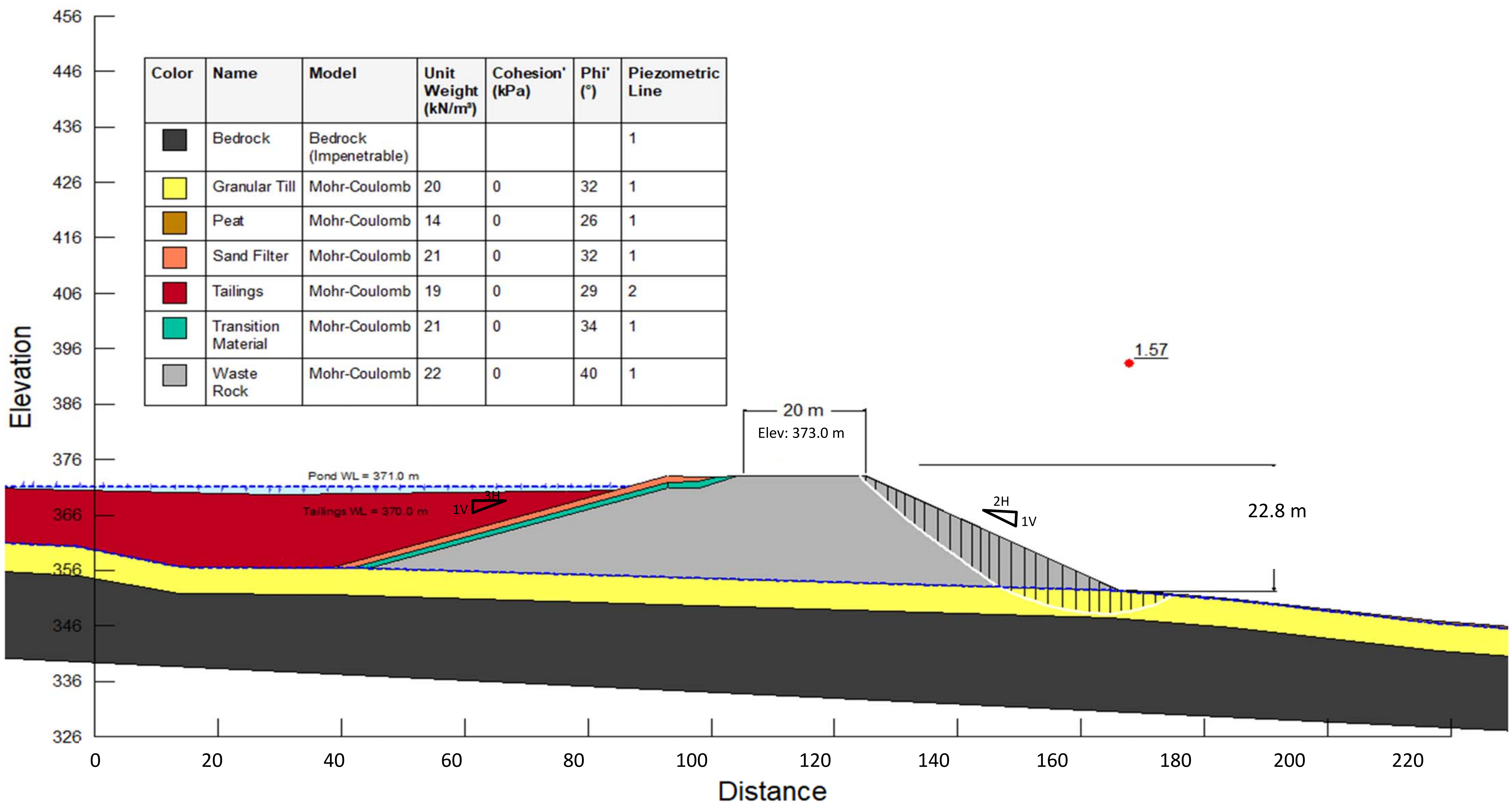
TITLE

Stage 1A End of Construction - Static D/S

TITLE	PHASE	REV	FIGURE
19130660	**4000**	**B**	**D3**



Color	Name	Model	Unit Weight (kN/m³)	Cohesion' (kPa)	Phi' (°)	Piezometric Line
	Bedrock	Bedrock (Impenetrable)				1
	Granular Till	Mohr-Coulomb	20	0	32	1
	Peat	Mohr-Coulomb	14	0	26	1
	Sand Filter	Mohr-Coulomb	21	0	32	1
	Tailings	Mohr-Coulomb	19	0	29	2
	Transition Material	Mohr-Coulomb	21	0	34	1
	Waste Rock	Mohr-Coulomb	22	0	40	1

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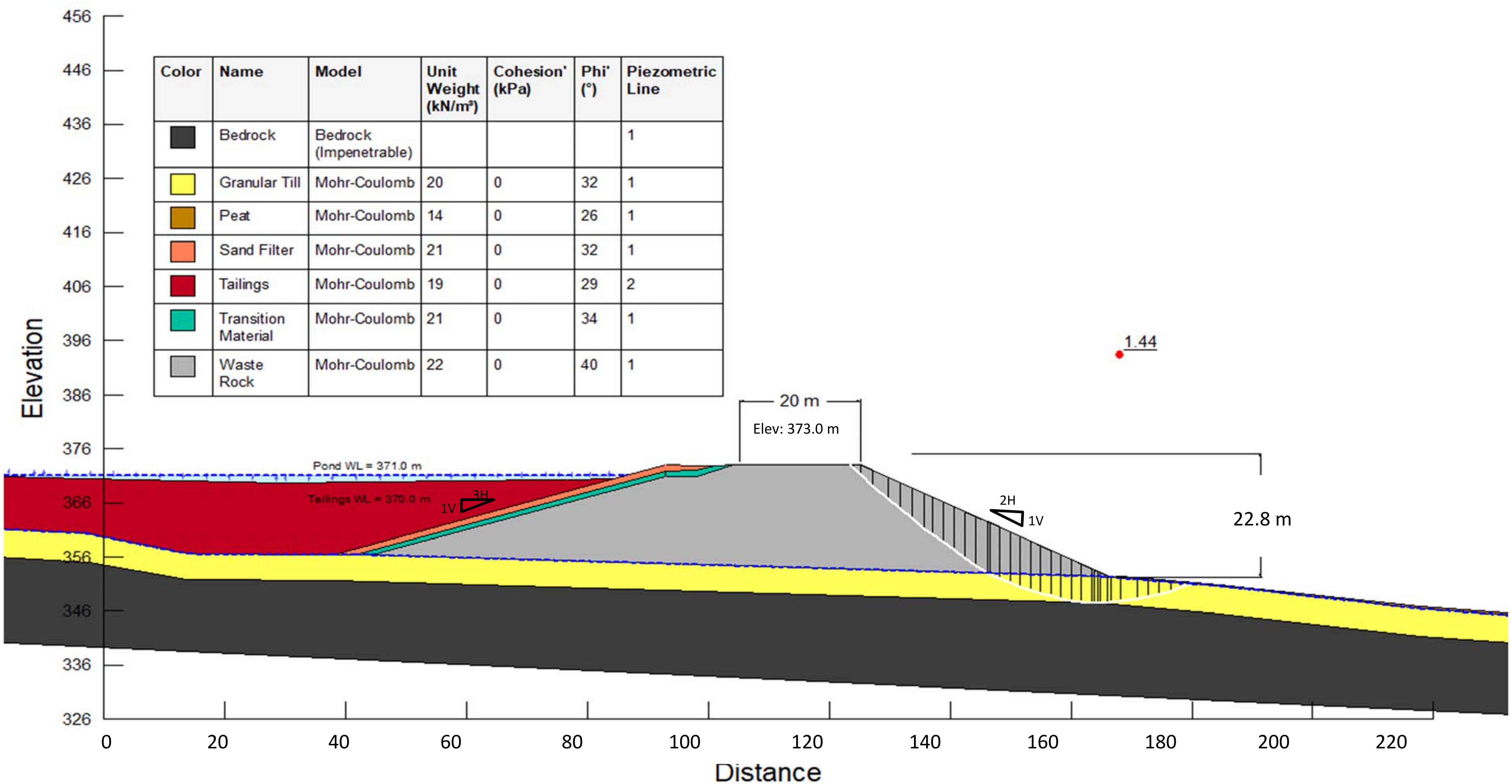
TITLE

Stage 1A Long-term steady state - Static D/S

TITLE	PHASE	REV	FIGURE
19130660	**4000**	**B**	**D4**



Color	Name	Model	Unit Weight (kN/m³)	Cohesion' (kPa)	Phi' (°)	Piezometric Line
	Bedrock	Bedrock (Impenetrable)				1
	Granular Till	Mohr-Coulomb	20	0	32	1
	Peat	Mohr-Coulomb	14	0	26	1
	Sand Filter	Mohr-Coulomb	21	0	32	1
	Tailings	Mohr-Coulomb	19	0	29	2
	Transition Material	Mohr-Coulomb	21	0	34	1
	Waste Rock	Mohr-Coulomb	22	0	40	1

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TITLE
Stage 1A - Pseudo-static (PGA = 0.038 g) D/S

TITLE	PHASE	REV	FIGURE
19130660	**4000**	**B**	**D5**

Color	Name	Model	Unit Weight (kN/m³)	Cohesion' (kPa)	Phi' (°)	Piezometric Line
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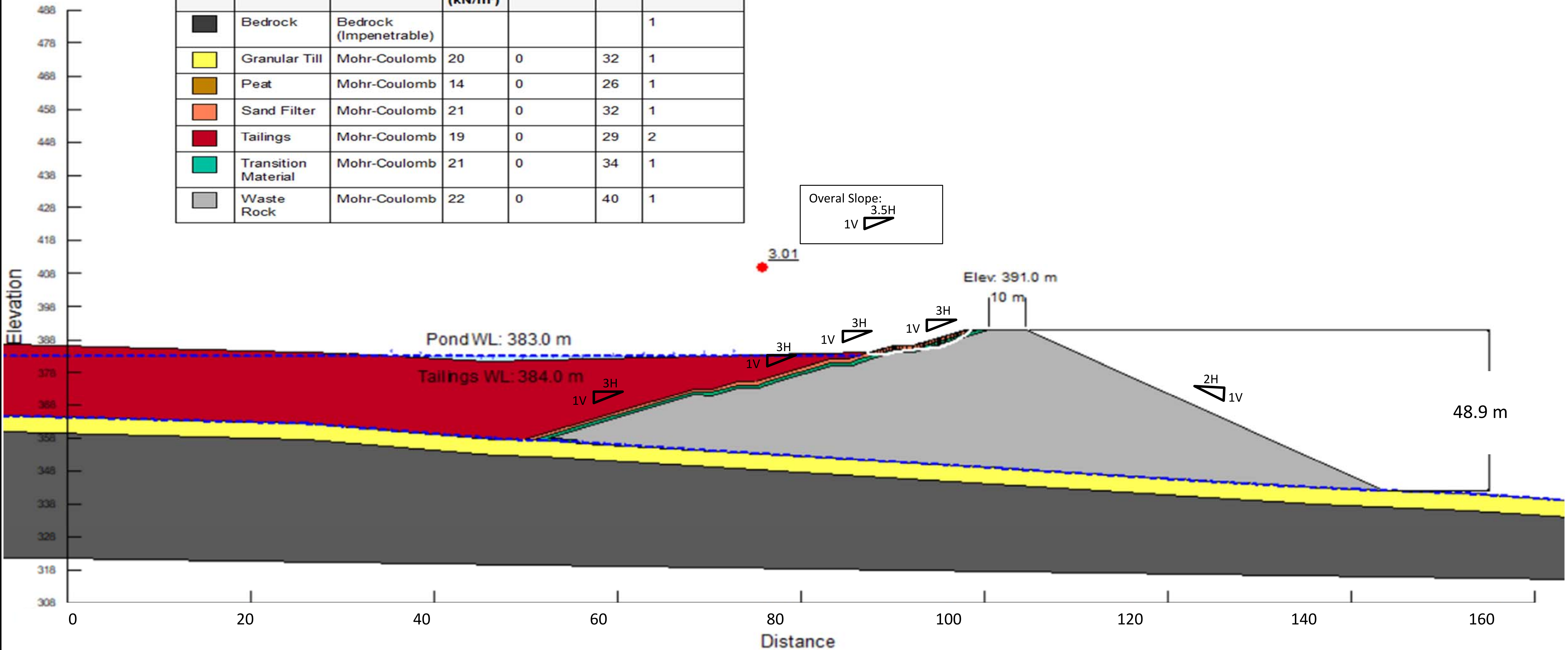
	Peat	Mohr-Coulomb	14	0	26	1
	Sand Filter	Mohr-Coulomb	21	0	32	1
	Tailings	Mohr-Coulomb	19	0	29	2
	Transition Material	Mohr-Coulomb	21	0	34	1
	Waste Rock	Mohr-Coulomb	22	0	40	1



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TITLE
Stage 4 - Ultimate End of Construction - Static U/S

TITLE	PHASE	REV	FIGURE
19130660	**4000**	**B**	**D6**

Color	Name	Model	Unit Weight (kN/m³)	Cohesion' (kPa)	Phi' (°)	Piezometric Line
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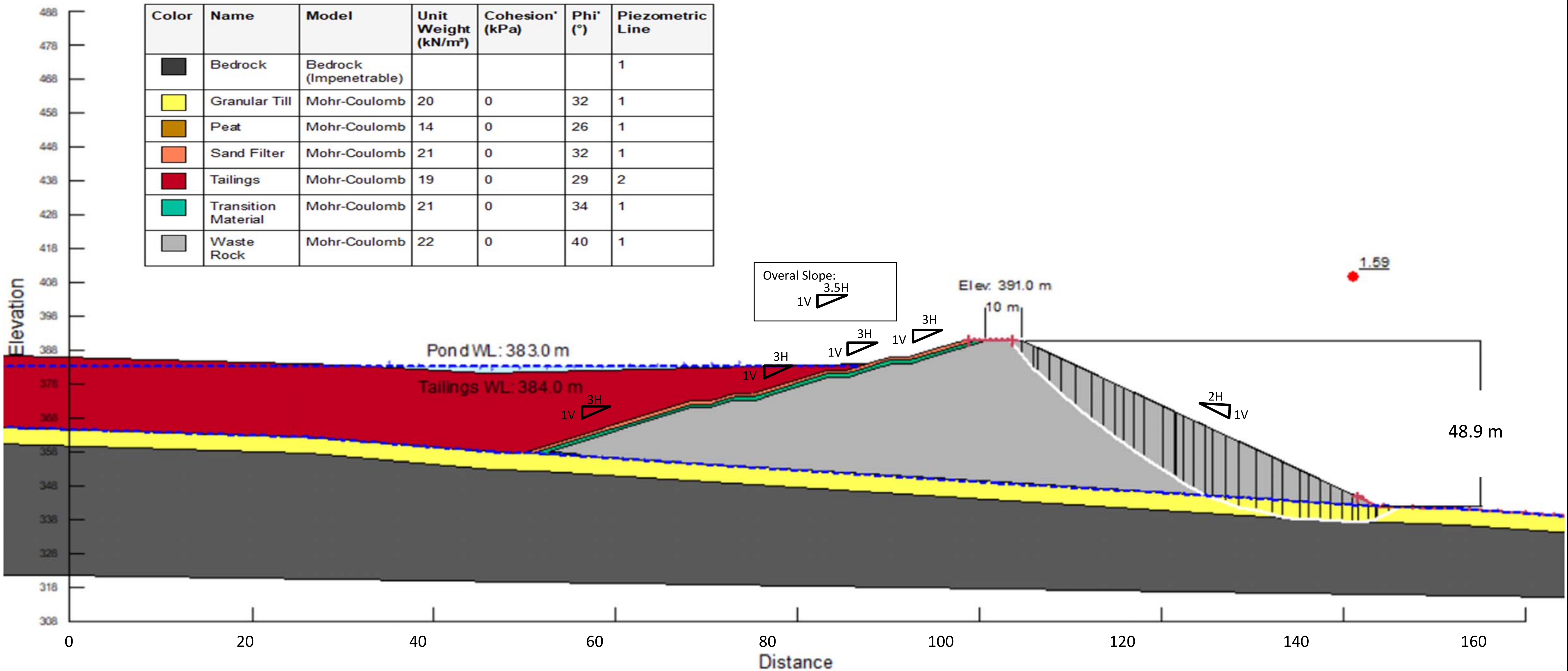
	Peat	Mohr-Coulomb	14	0	26	1
	Sand Filter	Mohr-Coulomb	21	0	32	1
	Tailings	Mohr-Coulomb	19	0	29	2
	Transition Material	Mohr-Coulomb	21	0	34	1
	Waste Rock	Mohr-Coulomb	22	0	40	1



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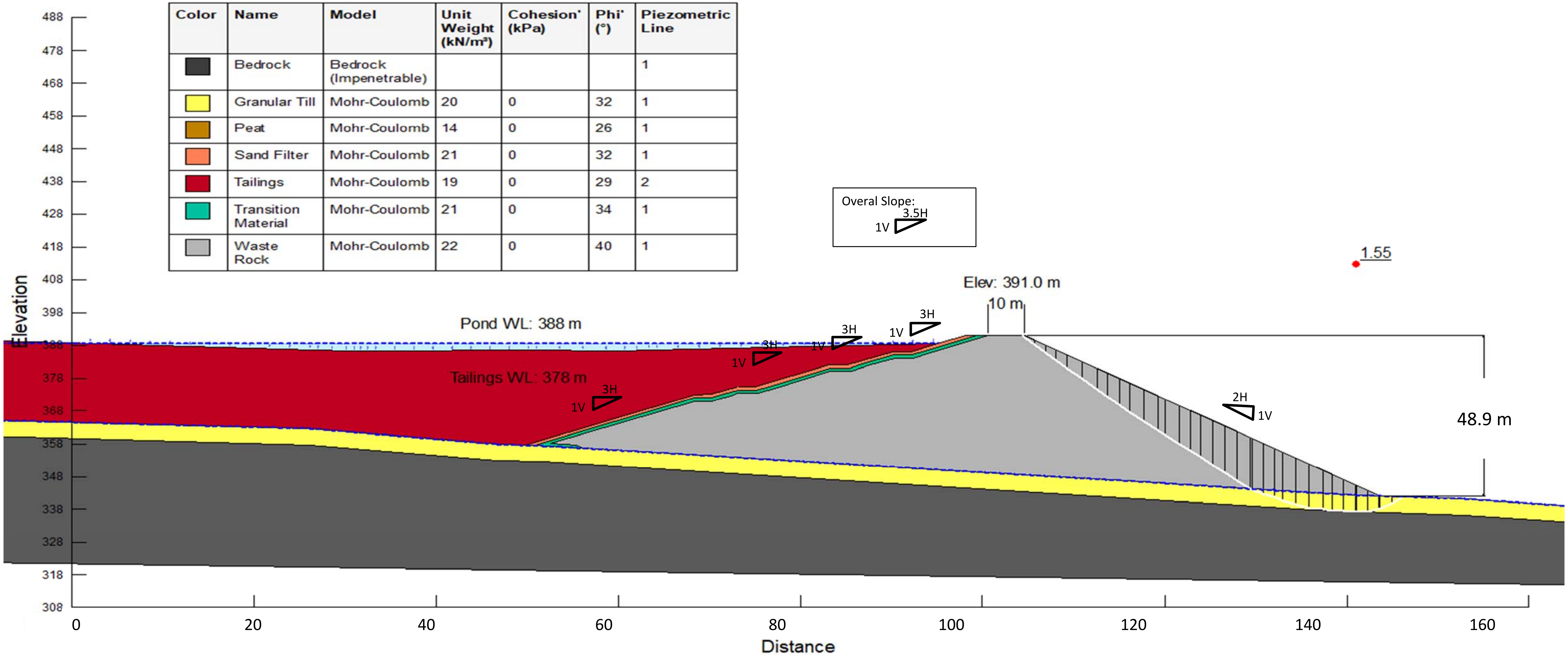
TITLE
Stage 4 - Ultimate End of Construction - Static D/S

TITLE	PHASE	REV	FIGURE
19130660	**4000**	**B**	**D7**



Color	Name	Model	Unit Weight (kN/m³)	Cohesion' (kPa)	Phi' (°)	Piezometric Line
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	Peat	Mohr-Coulomb	14	0	26	1
	Sand Filter	Mohr-Coulomb	21	0	32	1
	Tailings	Mohr-Coulomb	19	0	29	2
	Transition Material	Mohr-Coulomb	21	0	34	1
	Waste Rock	Mohr-Coulomb	22	0	40	1

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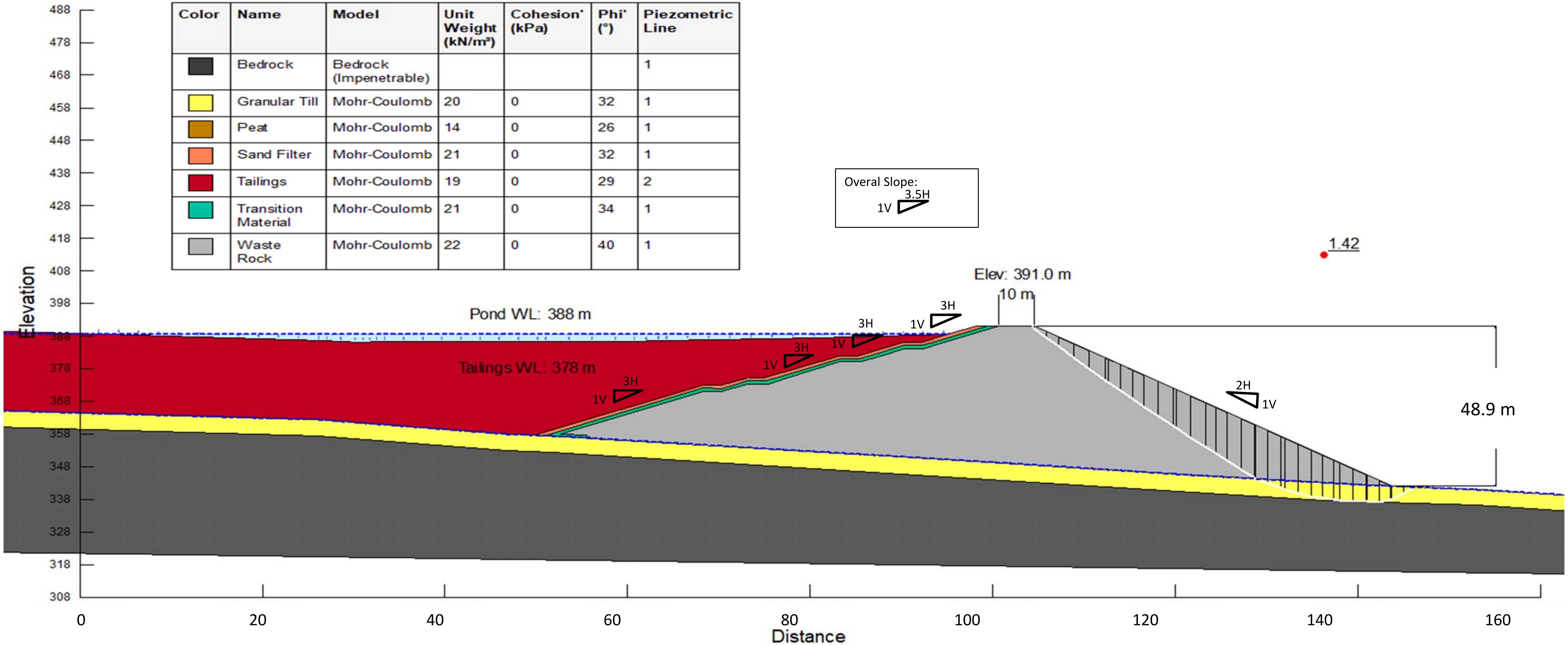
TITLE
Stage 4 - Ultimate Long-term steady state - Static D/S

TITLE	PHASE	REV	FIGURE
19130660	**4000**	**B**	**D8**



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	Granular Till	Mohr-Coulomb	20	0	32	1
	Peat	Mohr-Coulomb	14	0	26	1
	Sand Filter	Mohr-Coulomb	21	0	32	1
	Tailings	Mohr-Coulomb	19	0	29	2
	Transition Material	Mohr-Coulomb	21	0	34	1
	Waste Rock	Mohr-Coulomb	22	0	40	1

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APPROVED	PM

PROJECT
Valentine Gold Pre-feasibility Study

TITLE
Stage 4 - Ultimate - Pseudo static (PGA=0.038 g) D/S

TITLE	PHASE	REV	FIGURE
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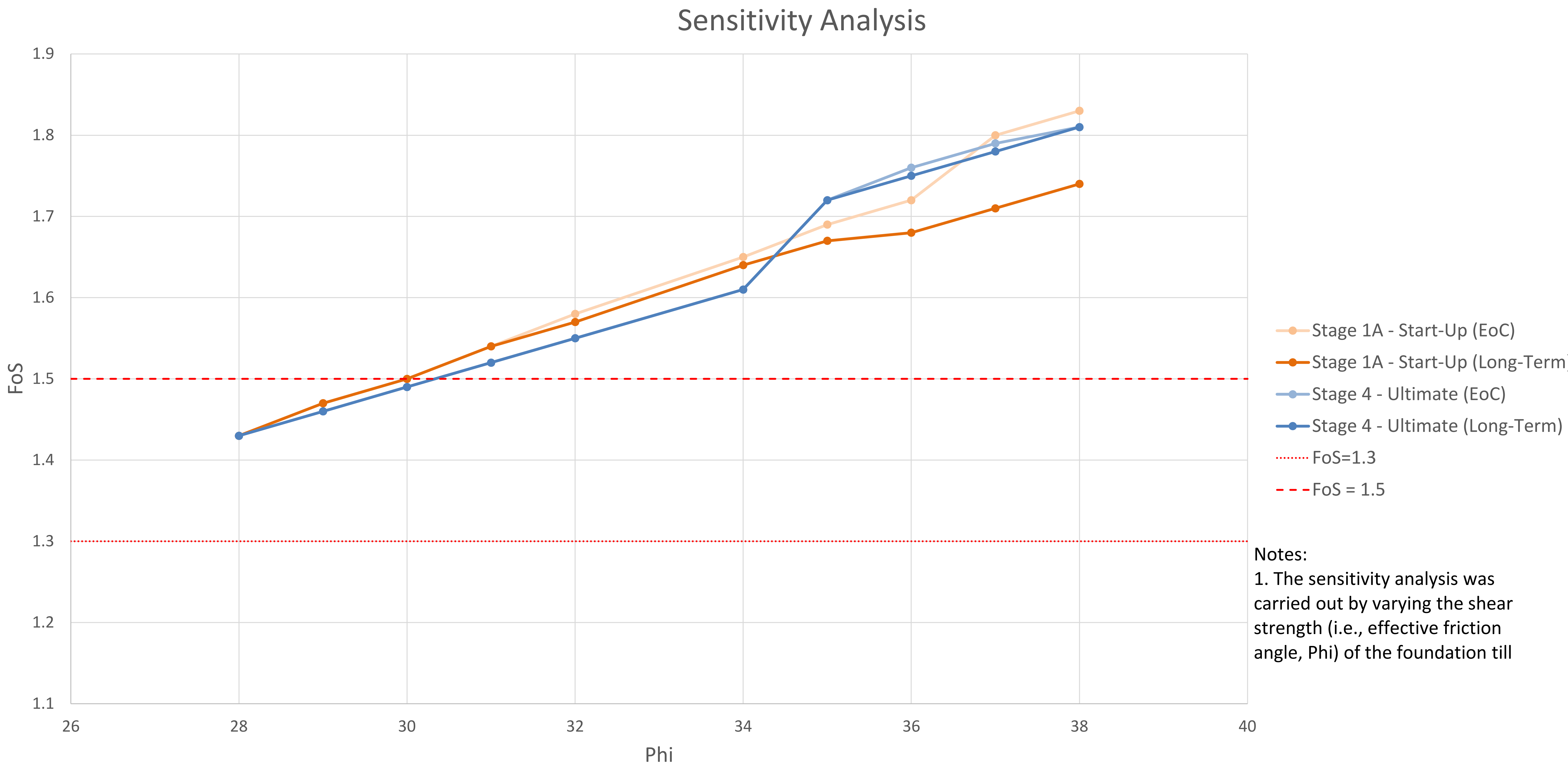

Notes:
1. The sensitivity analysis was carried out by varying the shear strength (i.e., effective friction angle, Phi) of the foundation till

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REVIEWED	MS
APPROVED	PM

TITLE
Sensitivity Analysis

TITLE	PHASE	REV	FIGURE
19130660	**4000**	**B**	**D10**

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: 48.360N 57.129W

2020-01-24 18:08 UT

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)	0.054	0.031	0.020	0.007
Sa (0.1)	0.078	0.047	0.031	0.012
Sa (0.2)	0.081	0.052	0.036	0.014
Sa (0.3)	0.074	0.049	0.035	0.014
Sa (0.5)	0.067	0.046	0.032	0.012
Sa (1.0)	0.046	0.031	0.021	0.006
Sa (2.0)	0.025	0.017	0.011	0.003
Sa (5.0)	0.007	0.004	0.003	0.001
Sa (10.0)	0.003	0.002	0.001	0.000
PGA (g)	0.046	0.028	0.019	0.006
PGV (m/s)	0.058	0.037	0.024	0.007

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

Natural Resources Canada   Ressources naturelles Canada

Canada

APPENDIX E

Liner Leakage Rate Estimates

MEMORANDUM

DATE March 5, 2020 **Project No.** 19130660

TO Matt Soderman, P.Eng (ON), Golder Associates Ltd.

CC

FROM Santosh Rimal, P.Eng. (ON) (Golder Associates Ltd.) **EMAIL** srimal@golder.com

PREFEASIBILITY STUDY – VALENTINE GOLD – TMF LINER LEAKAGE RATE ESTIMATE

The potential leakage rate through the proposed side-slope geomembrane liner is limited by the size and frequency of defects in the geomembrane liner that are not detected by the Construction Quality Assurance (CQA) inspection/testing program. Based on various case studies, Rowe et.al (2004) indicate that even with intensive CQA inspection/testing, one should expect/assume that an average of 5 small defects (2 mm approximate diameter) per hectare of geomembrane liner may not be detected. These defects represent the primary pathway through which leakage through the proposed side-slope geomembrane may occur.

The potential leakage rate per defect in the geomembrane liner was calculated for two different areas of the tailings management facility i.e.:

1. Area where the side-slope geomembrane is in contact with water (Ponded Area); and
2. Area where the side-slope geomembrane is in contact with tailings (Tailings Area).

Leakage Estimate – TMF Ponded Area

For the side-slope geomembrane in contact with water the leakage rate per defect in the geomembrane was calculated using the method presented by Giroud et. al. (1997) for leakage through a geomembrane underlain by a semi-permeable medium. In this case, the semi-permeable medium is the liner bedding layer beneath the geomembrane liner. The key assumptions used for the calculation are as follows:

- 5 holes (defects) per hectare of geomembrane liner;
- hole diameter of 2.0 mm;
- finished surface of the bedding layer is smooth and there is uniform contact with the overlying geomembrane;
- average head on the geomembrane at the midslope of the ponded area is 3.5 m; and
- hydraulic conductivity of the liner bedding layer is assumed to be 5 x 10^{-4} m/s.

Detailed calculations are presented in Attachment A. The results are summarized herein.

- **Ponded Area (20,400 m^2)**

Leakage Rate Per Defect	490 m^3/year/defect
Total Leakage Rate for 5 Defects Per Hectare	2,450 m^3/year/hectare

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Equivalent Uniform Leakage Flux through the Geomembrane in Contact with Water in the Ponded Area	0.245 m^3/m^2/year
Total leakage through the Geomembrane in the Ponded Area	0.245 m^3/m^2/year x 20,400 m^2 = 4998 m^3/year =13.7 m^3/day

Leakage Estimate – TMF Tailings Area

For the side-slope geomembrane in contact with tailings the leakage rate per defect in the geomembrane was calculated using the method presented by Badu-Tweneboah and Giroud, J.P. (2018) for leakage through a geomembrane overlain by the tailings. The key assumptions used for the calculation are as follows:

- 5 holes (defects) per hectare of geomembrane liner;
- hole radius of 1 mm (i.e. 0.001 m);
- geomembrane thickness of 1.5 mm (i.e. 0.0015 m);
- average head on the geomembrane at the midslope of the tailings area of 15.6 m; and
- hydraulic conductivity of the tailings of 1 x 10^{-6} m/s (assumed).

Detailed calculations are presented in Attachment A. The results are summarized herein.

- **Tailings Area (265,350 m^2)**

Leakage Rate Per Defect	0.676 m^3/year/defect
Total Leakage Rate for 5 Defects Per Hectare	3.4 m^3/year/hectare
Equivalent Uniform Leakage Flux through the Geomembrane in Contact with Water in the Ponded Area	0.00034 m^3/m^2/year
Total leakage through the Geomembrane in the Ponded Area	0.00034 m^3/m^2/year x 265,350 m^2 = 90 m^3/year = 0.25 m^3/day

Total Leakage Estimate – TMF

Therefore, the total leakage rate from the holes in the geomembrane in the Ponded Area and Tailings Area area combined is approximately 14 m^3/day.

https://golderassociates.sharepoint.com/sites/117078/project files/6 deliverables/4. pfs design report/appendices/appendix e - liner leakage/pfs_app_d_liner leakage rate.docx

ATTACHMENT A **Leakage Rate through Sideslope Geomembrane in Contact with Water at the Ponded Area** **Valentine Gold Project, Marathon Gold, Newfoundland**		
Project No.: 19130660	Prepared by: S. Rimal Reviewed by: F. Barone	Date: March 2020

References:

Ref.1 - Giroud J.P., King, T.D., Sanglerat, T.R., Hadj-Hamou, T., and Khire, M.V. (1997). "Rate of Liquid Migration Through Defects in a Geomembrane Placed on a Semi-Permeable Medium." Geosynthetics International, Vol. 4, Nos. 3-4, pp. 349-372.

Ref. 2 - Rowe, R.K., Quigley, R.M., Brachman, R.W.I., and Booker, J.R. (2004). Barrier Systems for Waste Disposal Facilities, 2nd Edition, Spon Press, 587p.

Assumptions:

(a) 5 holes per hectare of geomembrane (Ref. 2)
(b) Hole diameter = 2.0 mm (Ref. 2)
(c) Surface of the liner bedding layer beneath geomembrane is smooth and there is uniform contact with the geomembrane.
(d) Average head on the geomembrane at the midslope of the ponded area is 3.5 m.
(e) Hydraulic conductivity of the liner bedding is $5\text{x}10^{-4}$ m/s.

The leakage rate per hole can be estimated using the following equations (Ref. 1):

$$\log Q = 0.3195 + 2\log d + 0.5\log h - 0.74\left(\frac{5 + 2\log d - \log k_{um}}{n}\right)^{n}$$

$$n = 5.5540 - 0.4324\log d + 0.5405\log h + 1.3514\log C_{qo} + 1.3514\log\left[1 + 0.1\left(\frac{h}{t_{um}}\right)^{0.95}\right]$$

Where:

Q = leakage rate per hole (m^3/s)
D = hole diameter = 0.002 m
h = water pressure head on geomembrane surface at the midslope of the ponded area = 3.5 m
k_{um} = hydraulic conductivity of the liner bedding layer beneath the geomembrane = $5\text{x}10^{-4}$ m/s
C_{qo} = contact quality factor for geomembrane/bedding layer interface = 0.21 (for good uniform contact based on above reference)
t_{um} = thickness of liner bedding layer = 0.15 m (average)

Therefore, substituting the above input values in above equations we get the leakage rate per hole:

$Q \quad = 1.55 \times 10^{-5}\ m^3/s$
$\quad\quad = 490\ m^3/year$

The total leakage rate per hectare, assuming 5 holes per hectare (Ref. 2):

$Q_{total} \quad = 5$ holes/hectare x $490\ m^3/year$
$\quad\quad = 2450\ m^3/year/hectare$

Leakage flux i.e. equivalent uniform leakage rate per unit surface area

$q \quad = Q_{Total}/$ Area (Area = 1 Ha = $10{,}000\ m^2$)
$\quad\quad = 2450\ m^3/year\ /\ 10000\ m^2$
$\quad\quad = 0.245\ m^3/m^2/year$

Area of the sideslope geomembrane in contact with water at ponded area, A = $20{,}400\ m^2$

Therefore, the total leakage rate through sideslope geomembrane in contact with water at the ponded area:

$Q_{pond} \quad = q \times A$
$\quad\quad = 0.245\ m^3/m^2/year \times 20{,}400\ m^2$
$\quad\quad = 4998\ m^3/year$
$\quad\quad = 13.7\ m^3/day$

References:

Ref.1 – Rowe, R. K., Joshi, P., Brachman, R.W.I. and McLeod, H. (2017). "Leakage through Holes in Geomembrane below Saturated Tailings." Journal of Geotechnical and Geoenvironmental Engineering, 143(2).

Ref. 2 – Badu-Tweneboah, K. and Giroud, J.P. (2018). "Discussion of: Leakage through Holes in Geomembrane below Saturated Tailings by R. Kerry Rowe, Prabeen Joshi, R.W.I. Brachman and H. McLeod." Journal of Geotechnical and Geoenvironmental Engineering, 144(4).

Ref. 3 - Rowe, R.K., Quigley, R.M., Brachman, R.W.I., and Booker, J.R. (2004). Barrier Systems for Waste Disposal Facilities, 2nd Edition, Spon Press, 587p.

The leakage rate per hole can be estimated using the following equations (Ref. 2):

$$Q = \frac{4k_T h\, r}{1 + \frac{4}{\pi}\left(\frac{t_{GM}}{r}\right)}$$

Where:

Q = leakage rate per hole (m^3/s)
k_T = hydraulic conductivity of tailings = 1 x 10^{-6} m/s (assumed)
h = average head on geomembrane surface at the midslope of the geomembrane / tailings contact area =15.6 m
r = hole radius = 1 mm = 0.001 m
t_{GM} = thickness of geomembrane liner = 1.5 mm = 0.0015 m

Therefore, substituting the above input values in above equations we get the leakage rate per hole:

Q = 2.14 x 10^{-8} m^3/s
= 0.676 m^3/year

The total leakage rate per hectare, assuming 5 holes per hectare (Ref. 3):

Q_{total} = 5 holes/hectare x 0.676 m^3/year

$= 3.4\ m^3/year/hectare$

Leakage flux i.e. equivalent uniform leakage rate per unit surface area

$$
\begin{aligned}
q &= Q_{Total} / \text{Area (Area = 1 Ha = 10,000 } m^2) \\
&= 3.4\ m^3/year / 10000\ m^2 \\
&= 0.00034\ m^3/m^2/year
\end{aligned}
$$

Area of the sideslope geomembrane in contact with tailing, A = 265,350 m^2

Therefore, the total leakage rate through the geomembrane in contact with tailings:

$$
\begin{aligned}
Q_{pond} &= q \times A \\
&= 0.00034\ m^3/m^2/year \times 265,350\ m^2 \\
&= 90\ m^3/year \\
&= 0.25\ m^3/day
\end{aligned}
$$

APPENDIX F

Quantity and Cost Estimates

Ausenco No: 104319-MA-EST-003
Rev No: A

Marathon Gold Valentine PFS

TMF Scope
Capital Cost Estimate

Rev	Date	Description	Prepared	Checked	Approved
A	12-Feb-20	Issued for Client/Consultant Review	AT	AN	JPD

Ausenco Comm. Code	Description	Unit Rate	Rate Reference	Description	Comments
BBB	Clearing and grubbing borrow pit	1.3	Pennecon		
BBD	Stripping of topsoil within borrow pit (150 mm depth)	6.6	Pennecon	Strip topsoil (150 mm depth) and remove to topsoil stockpile area as directed	
****	Tree clearing TSF footprint excluding embankment	0.6	Pennecon		half the cost of BBB
BBB	Clearing and grubbing TSF embankment, spillway, ditches, and sediment pond footprints	1.3	Pennecon		
BBD	Stripping of topsoil (150 mm depth) for TSF embankments	6.6	Pennecon	Strip topsoil (150 mm depth) and remove to topsoil stockpile area as directed	
BCA	Excavation and disposal of unsuitable surficial soils within embankment footprint	4.4	Pennecon	Bulk Excavation Type 1 (Common)	
BGF	Waste rock to form embankment: Spread, moisture condition, compact, grade slopes to design profile (load, haul to site and dump is excluded as it is part of mining operation)	2.1	Pennecon	Zone 4 Rockfill - Downstream Shell	
BSB	Produce coarse filter material for embankment construction: load waste rock from mine stockpile, haul to crushing plant, crush and temporarily stockpile	16.4	Pennecon	m3	
BGB	Form coarse filter on upstream embankment slope with crushed material from stockpile in 0.3 m thick lifts: load from temporary stockpile at crusher, haul to TSF site, dump, spread, compact, and grade		Pennecon	Incl in BSB cost	
BGB1	Form fine filter on upstream embankment slope in 0.3 m thick lifts: excavate from borrow area, screen, load, haul to TSF site, place in layers, moisture condition, compact and grade to final design profile	17.3	Pennecon		
BLCB	Supply and install non-woven geotextile (400 g/m²) on embankment slope as cushion for geomembrane	5.2	Pennecon	Supply and install Bidim A24 geotextile below rip rap material at inlet and outlet	
BLDC	Supply and install geomembrane on upstream embankment slope, anchor on dam crest and tie in with concrete plinth at toe (mm LLDPE White, textured)	12.0	Pennecon		
BHAA	Trench excavation to bedrock. Excavate, load, haul, and stockpile excavated material outside TSF footprint	16.0	Pennecon	Excavate culvert trenches to finished profile, load, haul up to 1km	
****	Bedrock cleaning for initial slush grouting	63.3	GOLDER	use golder	
****	Place 20 Mpa, Type HS or HSb dental concrete for plinth foundation to form foundation for concrete plinth.	980.0	Pennecon	Dental Concrete	
****	Place Reinforcing Steel Dowling for Concrete Plinth - 15M Hilti HIT HY + Dowels 1000 mm long, embedded 500 mm into bedrock at 1500 mm intervals.		Pennecon	Epoxy for CRBA	
****	Place slush grout over dental concrete and around steel doweling to form smooth surface. Slush grout is to flow freely and consist of 1 part cement (Type HS or HSb) to 2 parts sand by volume	502.0	Pennecon	Slush Grout	
CRBA	Place reinforcing steel for placement of concrete. - CAN/CSA G30.18M, Gr. 400	21423.1	Pennecon	Reinforcing Steel Dowling for Concrete Plinth	0.0013t
CDBC	Supply and install concrete to form concrete plinth for liner connection. 35 Mpa/19 mm, Type HS or HSb Structural Concrete, Includes formwork, supply, placement, finishing.	980.0	Pennecon	Dental Concrete	
BBD	Spillway footprint stripping of topsoil (150 mm depth)	6.6	Pennecon	Strip topsoil (150 mm depth) and remove to topsoil stockpile area as directed	
BCA	Excavation and disposal of material	4.4	Pennecon	Bulk Excavation Type 1 (Common)	
BSB	Produce coarse riprap material for lining spillway channel in overburden: load waste rock from mine stockpile, haul to crushing plant, crush and temporarily stockpile	16.4	Pennecon	m3	
BLEB	Place coarse riprap material on base of spillway	19.0	GOLDER	use golder	
BDC	Blast bedrock. Includes blasting, loading, hauling, and stockpiling	23.9	Pennecon	Bulk Excavation (Rock Blast)	
****	Place Reinforcing Steel Dowling for spillway weir beam - 15M Hilti HIT HY + Dowels 1000 mm long, embedded 500 mm into bedrock at 1500 mm intervals.		Pennecon	Epoxy for CRBA	
CRBA	Place reinforcing steel for placement of concrete. - CAN/CSA G30.18M, Gr. 400	21423.1	Pennecon	Reinforcing Steel Dowling for Concrete Plinth	0.0013t
****	Place concrete to form concrete weir beam. 35 Mpa/19 mm, Type HS or HSb Structural Concrete. Includes formwork, supply, placement, and finishing	1500.0	Pennecon		
BBB	Polishing pond footprint clearing and grubbing	1.3	Pennecon		
BBD	Polishing pond footprint stripping (150 mm deep)	6.6	Pennecon	Strip topsoil (150 mm depth) and remove to topsoil stockpile area as directed	
BGC	Form polishing pond embankments. Excavate soil from pond footprint and constru perimeter embankment in 0.3 m thick compacted lifts. Includes moisture conditioning, compaction and grading to final design profile	23.0	GOLDER	use golder	
BDAB	Excavate polishing pond and dispose of soil	6.0	Pennecon	Excavate culvert trenches to finished profile, load, haul up to 1km	
BGB1	Form liner bedding (0.2 m) thick on upstream slopes and bottom of polishing pond: excavate from borrow area, screen, load, haul to TSF site, moisture condition, compact and grade to final design profile	16.4	Pennecon	m3	
BLCB	Supply and install non-woven geotextile (400 g/m²) on polishing pond base and slopes as cushion for geomembrane	5.2	Pennecon	Supply and install Bidim A24 geotextile below rip rap material at inlet and outlet	
BLDC	Supply and install geomembrane on polishing pond base, upstream slope and anchor on crest (1.5 mm LLDPE white, textured)	12.0	Pennecon	Supply and install Bidim A24 geotextile below rip rap material at inlet and outlet	
BDAB	Excavate sedimentation pond and dispose of soil	5.0	Pennecon	Excavate culvert trenches to finished profile, load, haul up to 1km	
BGC	Form sedimentation pond embankments. Excavate soil from pond footprint and construct perimeter embankment in 0.3 m thick compacted lifts. Includes moisture conditioning, compaction and grading to final design profile	23.0	GOLDER	use golder	
BKCAA	Excavate seepage collection ditches to 1 m depth with 2H:1V side slopes, 2 m base width	85.0	GOLDER	use golder	
BSB	Produce coarse riprap material for lining seepage collection ditches: load waste rock from mine stockpile, haul to crushing plant, crush and temporarily stockpile	16.4	Pennecon	m3	
BLEB	Place coarse riprap material on base of seepage collection ditches	19.0	GOLDER	use golder	
BCA	Soil Cover: load, haul, place, 0.3 m thick	1.3	Estimate	Estimate	
BVC	Hydroseed cover area	1.3	GOLDER	use golder	

Stage 1A - Year 0

1. TSF CAPITAL COSTS

Level	WBS	ComLev1	Ausenco Comm. Code	Item	Description	Quantity	Unit	Unit Rate	Cost	Total Cost	Rate Reference
					Indirect Capital Costs						
					Indirect Capital Costs					$ -	
3	341	B	****	1	Mobilization and Demobilization	-	lump sum.		$ -		Golder
					Direct Capital Costs						
Level	**WBS**	**ComLev1**	**Ausenco Comm. Code**	**Item**	**Description**	**Quantity**	**Unit**	**Unit Rate**	**Cost**	**Total Cost**	**Rate Reference**
					Borrow Pit Development					$ 90,400	
3	341	B	BBB	2	Clearing and grubbing borrow pit	40,000	m^2	$ 1.27	$ 50,800		Pennecon
3	341	B	BBD	3	Stripping of topsoil within borrow pit (150 mm depth)	6,000	m^3	$ 6.60	$ 39,600		Pennecon
Level	**WBS**	**ComLev1**	**Ausenco Comm. Code**	**Item**	**Description**	**Quantity**	**Unit**	**Unit Rate**	**Cost**	**Total Cost**	**Rate Reference**
					TSF Dam Construction					$ 10,787,866	
3	341	B	****	4	Tree clearing TSF footprint excluding embankment	701,100	m^2	$ 0.64	$ 445,199		Pennecon
3	341	B	BBB	5	Clearing and grubbing TSF embankment, spillway, ditches, and sediment pond footprints	263,810	m^2	$ 1.27	$ 335,039		Pennecon
3	341	B	BBD	6	Stripping of topsoil (150 mm depth) for TSF embankments	31,980	m^3	$ 6.60	$ 211,068		Pennecon
3	341	B	BCA	7	Excavation and disposal of unsuitable surficial soils within embankment footprint	74,620	m^3	$ 4.40	$ 328,328		Pennecon
3	341	B	BGF	8	Waste rock to form embankment: Spread, moisture condition, compact, grade slopes to design profile (load, haul to site and dump is excluded as it is part of mining operation)	1,825,300	m^3	$ 2.10	$ 3,833,130		Pennecon
3	341	B	BSB	9	Produce coarse filter material for embankment construction: load waste rock from mine stockpile, haul to crushing plant, crush and temporarily stockpile	103,260	m^3	$ 16.40	$ 1,693,464		Pennecon
3	341	B	BGB	10	Form coarse filter on upstream embankment slope with crushed material from stockpile in 0.3 m thick lifts: load from temporary stockpile at crusher, haul to TSF site, dump, spread, compact, and grade	103,260	m^3	$ -	$ -		Pennecon
3	341	B	BGB1	11	Form fine filter on upstream embankment slope in 0.3 m thick lifts: excavate from borrow area, screen, load, haul to TSF site, place in layers, moisture condition, compact and grade to final design profile	103,260	m^3	$ 17.30	$ 1,786,398		Pennecon
3	341	B	BLCB	12	Supply and install non-woven geotextile (400 g/m^2) on embankment slope as cushion for geomembrane	125,670	m^2	$ 5.15	$ 647,201		Pennecon
3	341	B	BLDC	13	Supply and install geomembrane on upstream embankment slope, anchor on dam crest and tie in with concrete plinth at toe (1.5 mm LLDPE White, textured)	125,670	m^2	$ 12.00	$ 1,508,040		Pennecon
Level	**WBS**	**ComLev1**	**Ausenco Comm. Code**	**Item**	**Description**	**Quantity**	**Unit**	**Unit Rate**	**Cost**	**Total Cost**	**Rate Reference**
					Liner Foundation Tie In					$ 1,183,193	
3	341	B	BHAA	14	Trench excavation to bedrock. Excavate, load, haul, and stockpile excavated material outside TSF footprint	29,880	m^3	$ 16.00	$ 478,080		Pennecon
3	341	B	****	15	Bedrock cleaning for initial slush grouting	747	m^2	$ 63.32	$ 47,300		GOLDER
3	341	C	****	16	Place 20 Mpa, Type HS or HSb dental concrete for plinth foundation to form foundation for concrete plinth.	100	m^3	$ 980.00	$ 98,000		Pennecon
3	341	C	****	17	Place Reinforcing Steel Dowling for Concrete Plinth - 15M Hilti HIT HY + Dowels 1000 mm long, embedded 500 mm into bedrock at 1500 mm intervals.	1,660	each	$ -	$ -		Pennecon
3	341	C	****	18	Place slush grout over dental concrete and around steel doweling to form smooth surface. Slush grout is to flow freely and consist of 1 part cement (Type HS or HSb) to 2 parts sand by volume	747	m^2	$ 502.00	$ 374,994		Pennecon
3	341	C	CRBA	19	Place reinforcing steel for placement of concrete. - CAN/CSA G30.18M, Gr. 400	2	tonnes	$ 21,423.08	$ 38,407		Pennecon
3	341	C	CDBC	20	Supply and install concrete to form concrete plinth for liner connection. 35 Mpa/19 mm, Type HS or HSb Structural Concrete, Includes formwork, supply, placement, finishing.	149	m^3	$ 980.00	$ 146,412		Pennecon
Level	**WBS**	**ComLev1**	**Ausenco Comm. Code**	**Item**	**Description**	**Quantity**	**Unit**	**Unit Rate**	**Cost**	**Total Cost**	**Rate Reference**
					Spillway					$ 72,213	
3	341	B	BBD	21	Spillway footprint stripping of topsoil (150 mm depth)	797	m^2	$ 6.60	$ 5,257		Pennecon
3	341	B	BCA	22	Excavation and disposal of material	3,138	m^3	$ 4.40	$ 13,805		Pennecon
3	341	B	BSB	23	Produce coarse riprap material for lining spillway channel in overburden: load waste rock from mine stockpile, haul to crushing plant, crush and temporarily stockpile	1,245	m^3	$ 16.40	$ 20,418		Pennecon
3	341	B	BLEB	24	Place coarse riprap material on base of spillway	1,245	m^3	$ 19.00	$ 23,655		GOLDER
3	341	B	BDC	25	Blast bedrock. Includes blasting, loading, hauling, and stockpiling	248	m^3	$ 23.90	$ 5,915		Pennecon
3	341	C	****	26	Place Reinforcing Steel Dowling for spillway weir beam - 15M Hilti HIT HY + Dowels 1000 mm long, embedded 500 mm into bedrock at 1500 mm intervals.	5	each	$ -	$ -		Pennecon
3	341	C	CRBA	27	Place reinforcing steel for placement of concrete. - CAN/CSA G30.18M, Gr. 400	0.02	tonnes	$ 21,423.08	$ 463		Pennecon
3	341	C	****	28	Place concrete to form concrete weir beam. 35 Mpa/19 mm, Type HS or HSb Structural Concrete. Includes formwork, supply, placement, and finishing	2	m^3	$ 1,500.00	$ 2,700		Pennecon
Level	**WBS**	**ComLev1**	**Ausenco Comm. Code**	**Item**	**Description**	**Quantity**	**Unit**	**Unit Rate**	**Cost**	**Total Cost**	**Rate Reference**
					Polishing Pond					$ 1,552,628	
3	341	B	BBB	29	Polishing pond footprint clearing and grubbing	53,800	m^2	$ 1.27	$ 68,326		Pennecon
3	341	B	BBD	30	Polishing pond footprint stripping (150 mm deep)	8,070	m^3	$ 6.60	$ 53,262		Pennecon
3	341	B	BGC	31	Form polishing pond embankments. Excavate soil from pond footprint and construct perimeter embankment in 0.3 m thick compacted lifts. Includes moisture conditioning, compaction and grading to final design profile	5,600	m^3	$ 23.00	$ 128,800		GOLDER
3	341	B	BDAB	32	Excavate polishing pond and dispose of soil	65,900	m^3	$ 6.00	$ 395,400		Pennecon
3	341	B	BGB1	33	Form liner bedding (0.2 m) thick on upstream slopes and bottom of polishing pond: excavate from borrow area, screen, load, haul to TSF site, moisture condition, compact and grade to final design profile	8,800	m^3	$ 17.30	$ 152,240		Pennecon
3	341	B	BLCB	34	Supply and install non-woven geotextile (400 g/m^2) on polishing pond base and slopes as cushion for geomembrane	44,000	m^2	$ 5.15	$ 226,600		Pennecon
3	341	B	BLDC	35	Supply and install geomembrane on polishing pond base, upstream slope and anchor on crest (1.5 mm LLDPE white, textured)	44,000	m^2	$ 12.00	$ 528,000		Pennecon
Level	**WBS**	**ComLev1**	**Ausenco Comm. Code**	**Item**	**Description**	**Quantity**	**Unit**	**Unit Rate**	**Cost**	**Total Cost**	**Rate Reference**
					Seepage and Runoff Collection System					$ 473,006	
3	341	B	BDAB	36	Excavate sedimentation pond and dispose of soil	7,500	m^3	$ 6.00	$ 45,000		Pennecon
3	341	B	BGC	37	Form sedimentation pond embankments. Excavate soil from pond footprint and construct perimeter embankment in 0.3 m thick compacted lifts. Includes moisture conditioning, compaction and grading to final design profile	750	m^3	$ 23.00	$ 17,250		GOLDER
3	341	B	BKCAA	38	Excavate seepage collection ditches to 1 m depth with 2H:1V side slopes, 2 m base width	2,900	m	$ 85.00	$ 246,500		GOLDER
3	341	B	BSB	39	Produce coarse riprap material for lining seepage collection ditches: load waste rock from mine stockpile, haul to crushing plant, crush and temporarily stockpile	4,640	m^3	$ 16.40	$ 76,096		Pennecon
3	341	B	BLEB	40	Place coarse riprap material on base of seepage collection ditches	4,640	m^3	$ 19.00	$ 88,160		GOLDER
							TOTAL COSTS - PRE CONTINGENCY - DIRECTS			**$ 14,159,306**	

Stage 1 - Year 2

1. TSF CAPITAL COSTS

Level	WBS	ComLev1	Ausenco Comm. Code	Item	Description	Quantity	Unit	Unit Rate	Cost	Total Cost	Rate Reference
					Indirect Capital Costs						
					Indirect Capital Costs					$ -	
3	341	B	****	1	Mobilization and Demobilization	1	lump sum.		$0		Golder
					Direct Capital Costs						
Level	**WBS**	**ComLev1**	**Ausenco Comm. Code**	**Item**	**Description**	**Quantity**	**Unit**	**Unit Rate**	**Cost**	**Total Cost**	**Rate Reference**
					Borrow Pit Development					$ 20,340	
3	341	B	BBB	2	Clearing and grubbing borrow pit	9,000	m^2	$ 1.27	$ 11,430		Pennecon
3	341	B	BBD	3	Stripping of topsoil within borrow pit (150 mm depth)	1,350	m^3	$ 6.60	$ 8,910		Pennecon
Level	**WBS**	**ComLev1**	**Ausenco Comm. Code**	**Item**	**Description**	**Quantity**	**Unit**	**Unit Rate**	**Cost**	**Total Cost**	**Rate Reference**
					TSF Dam Construction					$ 2,615,978	
3	341	B	****	4	Tree clearing TSF footprint excluding embankment	82,000	m^2	$ 0.64	$ 52,070		Pennecon
3	341	B	BBB	5	Clearing and grubbing TSF embankment and spillway footprints	37,280	m^2	$ 1.27	$ 47,346		Pennecon
3	341	B	BBD	6	Stripping of topsoil (150 mm depth) for TSF embankments	4,995	m^3	$ 6.60	$ 32,967		Pennecon
3	341	B	BCA	7	Excavation and disposal of unsuitable surficial soils within embankment footprint	11,655	m^3	$ 4.40	$ 51,282		Pennecon
3	341	B	BGF	8	Waste rock to form embankment: Spread, moisture condition, compact, grade slopes to design profile (load, haul to site and dump is excluded as it is part of mining operation)	603,700	m^3	$ 2.10	$ 1,267,770		Pennecon
3	341	B	BSB	9	Produce coarse filter material for embankment construction: load waste rock from mine stockpile, haul to crushing plant, crush and temporarily stockpile	22,340	m^3	$ 16.40	$ 366,376		Pennecon
3	341	B	BGB	10	Form coarse filter on upstream embankment slope with crushed material from stockpile in 0.3 m thick lifts: load from temporary stockpile at crusher, haul to TSF site, dump, spread, compact, and grade	22,340	m^3	$ -	$ -		Pennecon
3	341	B	BGB1	11	Form fine filter on upstream embankment slope in 0.3 m thick lifts: excavate from borrow area, screen, load, haul to TSF site, place in layers, moisture condition, compact and grade to final design profile	22,340	m^3	$ 17.30	$ 386,482		Pennecon
3	341	B	BLCB	12	Supply and install non-woven geotextile (400 g/m^2) on embankment slope as cushion for geomembrane	24,005	m^2	$ 5.15	$ 123,626		Pennecon
3	341	B	BLDC	13	Supply and install geomembrane on upstream embankment slope, anchor on dam crest and tie in with concrete plinth at toe (1.5 mm LLDPE White, textured)	24,005	m^2	$ 12.00	$ 288,060		Pennecon
Level	**WBS**	**ComLev1**	**Ausenco Comm. Code**	**Item**	**Description**	**Quantity**	**Unit**	**Unit Rate**	**Cost**	**Total Cost**	**Rate Reference**
					Liner Foundation Tie In					$ 100,227	
3	341	B	BHAA	14	Trench excavation to bedrock. Excavate, load, haul, and stockpile excavated material outside TSF footprint	2,220	m^3	$ 16.00	$35,520		Pennecon
3	341	B	****	15	Bedrock cleaning for initial slush grouting	56	m^2	$ 63.32	$3,514		GOLDER
3	341	C	****	16	Place 20 Mpa, Type HS or HSb dental concrete for plinth foundation to form foundation for concrete plinth.	20	m^3	$ 980.00	$19,600		Pennecon
3	341	C	****	17	Place Reinforcing Steel Dowling for Concrete Plinth - 15M Hilti HIT HY + Dowels 1000 mm long, embedded 500 mm into bedrock at 1500 mm intervals.	123	each	$ -	$0		Pennecon
3	341	C	****	18	Place slush grout over dental concrete and around steel doweling to form smooth surface. Slush grout is to flow freely and consist of 1 part cement (Type HS or HSb) to 2 parts sand by volume	56	m^2	$ 502.00	$27,861		Pennecon
3	341	C	CRBA	19	Place reinforcing steel for placement of concrete. - CAN/CSA G30.18M, Gr. 400	0.13	tonnes	$ 21,423.08	$2,854		Pennecon
3	341	C	CDBC	20	Supply and install concrete to form concrete plinth for liner connection. 35 Mpa/19 mm, Type HS or HSb Structural Concrete, Includes formwork, supply, placement, finishing.	11	m^3	$ 980.00	$10,878		Pennecon
Level	**WBS**	**ComLev1**	**Ausenco Comm. Code**	**Item**	**Description**	**Quantity**	**Unit**	**Unit Rate**	**Cost**	**Total Cost**	**Rate Reference**
					Spillway					$ 31,698	
3	341	B	BBD	21	Spillway footprint stripping of topsoil (150 mm depth)	324	m^2	$ 6.60	$ 2,138		Pennecon
3	341	B	BCA	22	Excavation and disposal of material	1,650	m^3	$ 4.40	$ 7,260		Pennecon
3	341	B	BSB	23	Produce coarse riprap material for lining spillway channel in overburden: load waste rock from mine stockpile, haul to crushing plant, crush and temporarily stockpile	374	m^3	$ 16.40	$ 6,125		Pennecon
3	341	B	BLEB	24	Place coarse riprap material on base of spillway	374	m^3	$ 19.00	$ 7,097		GOLDER
3	341	B	BDC	25	Blast bedrock. Includes blasting, loading, hauling, and stockpiling	248	m^3	$ 23.90	$ 5,915		Pennecon
3	341	C	****	26	Place Reinforcing Steel Dowling for spillway weir beam - 15M Hilti HIT HY + Dowels 1000 mm long, embedded 500 mm into bedrock at 1500 mm intervals.	5	each	$ -	$ -		Pennecon
3	341	C	CRBA	27	Place reinforcing steel for placement of concrete. - CAN/CSA G30.18M, Gr. 400	0.02	tonnes	$ 21,423.08	$ 463		Pennecon
3	341	C	****	28	Place concrete to form concrete weir beam. 35 Mpa/19 mm, Type HS or HSb Structural Concrete. Includes formwork, supply, placement, and finishing	2	m^3	$ 1,500.00	$ 2,700		Pennecon
							TOTAL COSTS - PRE CONTINGENCY - DIRECTS			**$ 2,768,243**	

Stage 2 - Year 3

1. TSF CAPITAL COSTS

Level	WBS	ComLev1	Ausenco Comm. Code	Item	Description	Quantity	Unit	Unit Rate	Cost	Total Cost	Rate Reference
					Indirect Capital Costs						
					Indirect Capital Costs					$ -	
3	341		****	1	Mobilization and Demobilization	1	lump sum.		$0		Golder
					Direct Capital Costs						
					Borrow Pit Development					$ 56,500	
3	341	B	BBB	2	Clearing and grubbing borrow pit	25,000	m^2	$ 1.27	$ 31,750		Pennecon
3	341	B	BBD	3	Stripping of topsoil within borrow pit (150 mm depth)	3,750	m^3	$ 6.60	$ 24,750		Pennecon
Level	**WBS**	**ComLev1**	**Ausenco Comm. Code**	**Item**	**Description**	**Quantity**	**Unit**	**Unit Rate**	**Cost**	**Total Cost**	**Rate Reference**
					TSF Dam Construction					$ 8,383,585	
3	341	B	****	4	Tree clearing TSF footprint excluding embankment	224,630	m^2	$ 0.64	$ 142,640		Pennecon
3	341	B	BBB	5	Clearing and grubbing TSF embankment and spillway footprints	115,800	m^2	$ 1.27	$147,066		Pennecon
3	341	B	BBD	6	Stripping of topsoil (150 mm depth) for TSF embankments	16,575	m^3	$ 6.60	$109,395		Pennecon
3	341	B	BCA	7	Excavation and disposal of unsuitable surficial soils within embankment footprint	38,675	m^3	$ 4.40	$170,170		Pennecon
3	341	B	BGF	8	Waste rock to form embankment: Spread, moisture condition, compact, grade slopes to design profile (load, haul to site and dump is excluded as it is part of mining operation)	2,205,700	m^3	$ 2.10	$4,631,970		Pennecon
3	341	B	BSB	9	Produce coarse filter material for embankment construction: load waste rock from mine stockpile, haul to crushing plant, crush and temporarily stockpile	61,900	m^3	$ 16.40	$1,015,160		Pennecon
3	341	B	BGB	10	Form coarse filter on upstream embankment slope with crushed material from stockpile in 0.3 m thick lifts: load from temporary stockpile at crusher, haul to TSF site, dump, spread, compact, and grade	61,900	m^3	$ -	$0		Pennecon
3	341	B	BGB1	11	Form fine filter on upstream embankment slope in 0.3 m thick lifts: excavate from borrow area, screen, load, haul to TSF site, place in layers, moisture condition, compact and grade to final design profile	61,900	m^3	$ 17.30	$1,070,870		Pennecon
3	341	B	BLCB	12	Supply and install non-woven geotextile (400 g/m²) on embankment slope as cushion for geomembrane	63,925	m^2	$ 5.15	$329,214		Pennecon
3	341	B	BLDC	13	Supply and install geomembrane on upstream embankment slope, anchor on dam crest and tie in with concrete plinth at toe (1.5 mm LLDPE White, textured)	63,925	m^2	$ 12.00	$767,100		Pennecon
Level	**WBS**	**ComLev1**	**Ausenco Comm. Code**	**Item**	**Description**	**Quantity**	**Unit**	**Unit Rate**	**Cost**	**Total Cost**	**Rate Reference**
					Liner Foundation Tie In					$ 131,135	
3	341	B	BHAA	14	Trench excavation to bedrock. Excavate, load, haul, and stockpile excavated material outside TSF footprint	2,700	m^3	$ 16.00	$43,200		Pennecon
3	341	B	****	15	Bedrock cleaning for initial slush grouting	68	m^2	$ 63.32	$4,274		GOLDER
3	341	C	****	16	Place 20 Mpa, Type HS or HSb dental concrete for plinth foundation to form foundation for concrete plinth.	34	m^3	$ 980.00	$33,075		Pennecon
3	341	C	****	17	Place Reinforcing Steel Dowling for Concrete Plinth - 15M Hilti HIT HY + Dowels 1000 mm long, embedded 500 mm into bedrock at 1500 mm intervals.	150	each	$ -	$0		Pennecon
3	341	C	****	18	Place slush grout over dental concrete and around steel doweling to form smooth surface. Slush grout is to flow freely and consist of 1 part cement (Type HS or HSb) to 2 parts sand by volume	68	m^2	$ 502.00	$33,885		Pennecon
3	341	C	CRBA	19	Place reinforcing steel for placement of concrete. - CAN/CSA G30.18M, Gr. 400	0.16	tonnes	$ 21,423.08	$3,471		Pennecon
3	341	C	CDBC	20	Supply and install concrete to form concrete plinth for liner connection. 35 Mpa/19 mm, Type HS or HSb Structural Concrete, Includes formwork, supply, placement, finishing.	14	m^3	$ 980.00	$13,230		Pennecon
Level	**WBS**	**ComLev1**	**Ausenco Comm. Code**	**Item**	**Description**	**Quantity**	**Unit**	**Unit Rate**	**Cost**	**Total Cost**	**Rate Reference**
					Spillway					$ 43,274	
3	341	B	BBD	21	Spillway footprint stripping of topsoil (150 mm depth)	459	m^3	$ 6.60	$ 3,029		Pennecon
3	341	B	BCA	22	Excavation and disposal of material	2,075	m^3	$ 4.40	$ 9,130		Pennecon
3	341	B	BSB	23	Produce coarse riprap material for lining spillway channel in overburden: load waste rock from mine stockpile, haul to crushing plant, crush and temporarily stockpile	623	m^3	$ 16.40	$ 10,209		Pennecon
3	341	B	BLEB	24	Place coarse riprap material on base of spillway	623	m^3	$ 19.00	$ 11,828		GOLDER
3	341	B	BDC	25	Blast bedrock. Includes blasting, loading, hauling, and stockpiling	248	m^3	$ 23.90	$ 5,915		Pennecon
3	341	C	****	26	Place Reinforcing Steel Dowling for spillway weir beam - 15M Hilti HIT HY + Dowels 1000 mm long, embedded 500 mm into bedrock at 1500 mm intervals.	5	each	$ -	$ -		Pennecon
3	341	C	CRBA	27	Place reinforcing steel for placement of concrete. - CAN/CSA G30.18M, Gr. 400	0.02	tonnes	$ 21,423.08	$ 463		Pennecon
3	341	C	****	28	Place concrete to form concrete weir beam. 35 Mpa/19 mm, Type HS or HSb Structural Concrete. Includes formwork, supply, placement, and finishing	2	m^3	$ 1,500.00	$ 2,700		Pennecon
						TOTAL COSTS - PRE CONTINGENCY - DIRECTS				**$ 8,614,493**	

Stage 3 - Year 5

1. TSF CAPITAL COSTS

Level	WBS	ComLev1	Ausenco Comm. Code	Item	Description						
					Indirect Capital Costs						
Level	WBS	ComLev1	Ausenco Comm. Code	Item	Description						
					Indirect Capital Costs					$ -	
3	341		****	1	Mobilization and Demobilization	1	lump sum.		$ -		Golder
					Direct Capital Costs						
Level	WBS	ComLev1	Ausenco Comm. Code	Item	Description						
					Borrow Pit Development					$ 49,720	
3	341	B	BBB	2	Clearing and grubbing borrow pit	22,000	m^2	$ 1.27	$ 27,940		Pennecon
3	341	B	BBD	3	Stripping of topsoil within borrow pit (150 mm depth)	3,300	m^3	$ 6.60	$ 21,780		Pennecon
Level	WBS	ComLev1	Ausenco Comm. Code	Item	Description						
					TSF Dam Construction					$ 8,129,345	
3	341	B	****	4	Tree clearing TSF footprint excluding embankment	318,100	m^2	$ 0.64	$ 201,994		Pennecon
3	341	B	BBB	5	Clearing and grubbing TSF embankment, ditches, and spillway footprint	119,750	m^2	$ 1.27	$ 152,083		Pennecon
3	341	B	BBD	6	Stripping of topsoil (150 mm depth) for TSF embankments	14,175	m^3	$ 6.60	$ 93,555		Pennecon
3	341	B	BCA	7	Excavation and disposal of unsuitable surficial soils within embankment footprint	33,075	m^3	$ 4.40	$ 145,530		Pennecon
3	341	B	BGF	8	Waste rock to form embankment: Spread, moisture condition, compact, grade slopes to design profile (load, haul to site and dump is excluded as it is part of mining operation)	2,247,600	m^3	$ 2.10	$ 4,719,960		Pennecon
3	341	B	BSB	9	Produce coarse filter material for embankment construction: load waste rock from mine stockpile, haul to crushing plant, crush and temporarily stockpile	54,700	m^3	$ 16.40	$ 897,080		Pennecon
3	341	B	BGB	10	Form coarse filter on upstream embankment slope with crushed material from stockpile in 0.3 m thick lifts: load from temporary stockpile at crusher, haul to TSF site, dump, spread, compact, and grade	54,700	m^3	$ -	$ -		Pennecon
3	341	B	BGB1	11	Form fine filter on upstream embankment slope in 0.3 m thick lifts: excavate from borrow area, screen, load, haul to TSF site, place in layers, moisture condition, compact and grade to final design profile	54,700	m^3	$ 17.30	$ 946,310		Pennecon
3	341	B	BLCB	12	Supply and install non-woven geotextile (400 g/m^2) on embankment slope as cushion for geomembrane	56,725	m^2	$ 5.15	$ 292,134		Pennecon
3	341	B	BLDC	13	Supply and install geomembrane on upstream embankment slope, anchor on dam crest and tie in with concrete plinth at toe (1.5 mm LLDPE White, textured)	56,725	m^2	$ 12.00	$ 680,700		Pennecon
Level	WBS	ComLev1	Ausenco Comm. Code	Item	Description						
					Liner Foundation Tie In					$ 130,910	
3	341	B	BHAA	14	Trench excavation to bedrock. Excavate, load, haul, and stockpile excavated material outside TSF footprint	2,700	m^3	$ 16.00	$ 43,200		Pennecon
3	341	B	****	15	Bedrock cleaning for initial slush grouting	68	m^2	$ 63.32	$ 4,274		GOLDER
3	341	C	****	16	Place 20 Mpa, Type HS or HSb dental concrete for plinth foundation to form foundation for concrete plinth.	34	m^3	$ 980.00	$ 33,075		Pennecon
3	341	C	****	17	Place Reinforcing Steel Dowling for Concrete Plinth - 15M Hilti HIT HY + Dowels 1000 mm long, embedded 500 mm into bedrock at 1500 mm intervals.	149	each	$ -	$ -		Pennecon
3	341	C	****	18	Place slush grout over dental concrete and around steel doweling to form smooth surface. Slush grout is to flow freely and consist of 1 part cement (Type HS or HSb) to 2 parts sand by volume	67	m^2	$ 502.00	$ 33,734		Pennecon
3	341	C	CRBA	19	Place reinforcing steel for placement of concrete. - CAN/CSA G30.18M, Gr. 400	0.16	tonnes	$ 21,423.08	$ 3,455		Pennecon
3	341	C	CDBC	20	Supply and install concrete to form concrete plinth for liner connection. 35 Mpa/19 mm, Type HS or HSb Structural Concrete, Includes formwork, supply, placement, finishing.	13	m^3	$ 980.00	$ 13,171		Pennecon
Level	WBS	ComLev1	Ausenco Comm. Code	Item	Description						
					Spillway					$ 43,274	
3	341	B	BBD	21	Spillway footprint stripping of topsoil (150 mm depth)	459	m^3	$ 6.60	$ 3,029		Pennecon
3	341	B	BCA	22	Excavation and disposal of material	2,075	m^3	$ 4.40	$ 9,130		Pennecon
3	341	B	BSB	23	Produce coarse riprap material for lining spillway channel in overburden: load waste rock from mine stockpile, haul to crushing plant, crush and temporarily stockpile	623	m^3	$ 16.40	$ 10,209		Pennecon
3	341	B	BLEB	24	Place coarse riprap material on base of spillway	623	m^3	$ 19.00	$ 11,828		GOLDER
3	341	B	BDC	25	Blast bedrock. Includes blasting, loading, hauling, and stockpiling	248	m^3	$ 23.90	$ 5,915		Pennecon
3	341	C	****	26	Place Reinforcing Steel Dowling for spillway weir beam - 15M Hilti HIT HY + Dowels 1000 mm long, embedded 500 mm into bedrock at 1500 mm intervals.	5	each	$ -	$ -		Pennecon
3	341	C	CRBA	27	Place reinforcing steel for placement of concrete. - CAN/CSA G30.18M, Gr. 400	0.02	tonnes	$ 21,423.08	$ 463		Pennecon
3	341	C	****	28	Place concrete to form concrete weir beam. 35 Mpa/19 mm, Type HS or HSb Structural Concrete. Includes formwork, supply, placement, and finishing	2	m^3	$ 1,500.00	$ 2,700		Pennecon
Level	WBS	ComLev1	Ausenco Comm. Code	Item	Description						
					Seepage and Runoff Collection System					$ 470,953	
3	341	B	BKCAA	29	Excavate seepage collection ditches to 1 m depth with 2H:1V side slopes, 2 m base width	3,325	m	$ 85.00	$ 282,625		GOLDER
3	341		BSB	30	Produce coarse riprap material for lining seepage collection ditches: load waste rock from mine stockpile, haul to crushing plant, crush and temporarily stockpile	5,320	m^3	$ 16.40	$ 87,248		Pennecon
3	341		BLEB	31	Place coarse riprap material on base of seepage collection ditches	5,320	m^3	$ 19.00	$ 101,080		GOLDER
					TOTAL COSTS - PRE CONTINGENCY - DIRECTS					$ 8,824,201	

Stage 4 - Year 7

1. TSF CAPITAL COSTS

					Indirect Capital Costs						
Level	**WBS**	**ComLev1**	**Ausenco Comm. Code**	**Item**	**Description**						
					Indirect Capital Costs					**$ -**	
3	341		****	1	Mobilization and Demobilization	1	lump sum.		$ -		Golder
					Direct Capital Costs						
Level	**WBS**	**ComLev1**	**Ausenco Comm. Code**	**Item**	**Description**						
					Borrow Pit Development					**$ 51,980**	
3	341	B	BBB	2	Clearing and grubbing borrow pit	23,000	m^2	$ 1.27	$ 29,210		Pennecon
3	341	B	BBD	3	Stripping of topsoil within borrow pit (150 mm depth)	3,450	m^3	$ 6.60	$ 22,770		Pennecon
Level	**WBS**	**ComLev1**	**Ausenco Comm. Code**	**Item**	**Description**						
					TSF Dam Construction					**$ 7,018,356**	
3	341	B	****	4	Tree clearing TSF footprint excluding embankment	219,800	m^2	$ 0.64	$ 139,573		Pennecon
3	341	B	BBB	5	Clearing and grubbing TSF embankment and spillway footprint	65,100	m^2	$ 1.27	$ 82,677		Pennecon
3	341	B	BBD	6	Stripping of topsoil (150 mm depth) for TSF embankments	9,210	m^3	$ 6.60	$ 60,786		Pennecon
3	341	B	BCA	7	Excavation and disposal of unsuitable surficial soils within embankment footprint	21,490	m^3	$ 4.40	$ 94,556		Pennecon
3	341	B	BGF	8	Waste rock to form embankment: Spread, moisture condition, compact, grade slopes to design profile (load, haul to site and dump is excluded as it is part of mining operation)	1,769,800	m^3	$ 2.10	$ 3,716,580		Pennecon
3	341	B	BSB	9	Produce coarse filter material for embankment construction: load waste rock from mine stockpile, haul to crushing plant, crush and temporarily stockpile	56,900	m^3	$ 16.40	$ 933,160		Pennecon
3	341	B	BGB	10	Form coarse filter on upstream embankment slope with crushed material from stockpile in 0.3 m thick lifts: load from temporary stockpile at crusher, haul to TSF site, dump, spread, compact, and grade	56,900	m^3	$ -	$ -		Pennecon
3	341	B	BGB1	11	Form fine filter on upstream embankment slope in 0.3 m thick lifts: excavate from borrow area, screen, load, haul to TSF site, place in layers, moisture condition, compact and grade to final design profile	56,900	m^3	$ 17.30	$ 984,370		Pennecon
3	341	B	BLCB	12	Supply and install non-woven geotextile (400 g/m²) on embankment slope as cushion for geomembrane	58,697	m^2	$ 5.15	$ 302,290		Pennecon
3	341	B	BLDC	13	Supply and install geomembrane on upstream embankment slope, anchor on dam crest and tie in with concrete plinth at toe (1.5 mm LLDPE White, textured)	58,697	m^2	$ 12.00	$ 704,364		Pennecon
Level	**WBS**	**ComLev1**	**Ausenco Comm. Code**	**Item**	**Description**						
					Liner Foundation Tie In					**$ 116,564**	
3	341	B	BHAA	14	Trench excavation to bedrock. Excavate, load, haul, and stockpile excavated material outside TSF footprint	2,400	m^3	$ 16.00	$38,400		Pennecon
3	341	B	****	15	Bedrock cleaning for initial slush grouting	60	m^2	$ 63.32	$3,799		GOLDER
3	341	C	****	16	Place 20 Mpa, Type HS or HSb dental concrete for plinth foundation to form foundation for concrete plinth.	30	m^3	$ 980.00	$29,400		Pennecon
3	341	C	****	17	Place Reinforcing Steel Dowling for Concrete Plinth - 15M Hilti HIT HY + Dowels 1000 mm long, embedded 500 mm into bedrock at 1500 mm intervals.	133	each	$ -	$0		Pennecon
3	341	C	****	18	Place slush grout over dental concrete and around steel doweling to form smooth surface. Slush grout is to flow freely and consist of 1 part cement (Type HS or HSb) to 2 parts sand by volume	60	m^2	$ 502.00	$30,120		Pennecon
3	341	C	CRBA	19	Place reinforcing steel for placement of concrete. - CAN/CSA G30.18M, Gr. 400	0.14	tonnes	$ 21,423.08	$3,085		Pennecon
3	341	C	CDBC	20	Supply and install concrete to form concrete plinth for liner connection. 35 Mpa/19 mm, Type HS or HSb Structural Concrete, Includes formwork, supply, placement, finishing.	12	m^3	$ 980.00	$11,760		Pennecon
Level	**WBS**	**ComLev1**	**Ausenco Comm. Code**	**Item**	**Description**						
					Spillway					**$ 25,910**	
3	341	B	BBD	21	Spillway footprint stripping of topsoil (150 mm depth)	257	m^2	$ 6.60	$ 1,693		Pennecon
3	341	B	BCA	22	Excavation and disposal of material	1,438	m^3	$ 4.40	$ 6,325		Pennecon
3	341	B	BSB	23	Produce coarse riprap material for lining spillway channel in overburden: load waste rock from mine stockpile, haul to crushing plant, crush and temporarily stockpile	249	m^3	$ 16.40	$ 4,084		Pennecon
3	341	B	BLEB	24	Place coarse riprap material on base of spillway	249	m^3	$ 19.00	$ 4,731		GOLDER
3	341	B	BDC	25	Blast bedrock. Includes blasting, loading, hauling, and stockpiling	248	m^3	$ 23.90	$ 5,915		Pennecon
3	341	C	****	26	Place Reinforcing Steel Dowling for spillway weir beam - 15M Hilti HIT HY + Dowels 1000 mm long, embedded 500 mm into bedrock at 1500 mm intervals.	5	each	$ -	$ -		Pennecon
3	341	C	CRBA	27	Place reinforcing steel for placement of concrete. - CAN/CSA G30.18M, Gr. 400	0.02	tonnes	$ 21,423.08	$ 463		Pennecon
3	341	C	****	28	Place concrete to form concrete weir beam. 35 Mpa/19 mm, Type HS or HSb Structural Concrete. Includes formwork, supply, placement, and finishing	2	m^3	$ 1,500.00	$ 2,700		Pennecon
					TOTAL COSTS - PRE CONTINGENCY - DIRECTS					**$ 7,212,810**	

Closure

Level	WBS	ComLev1	Ausenco Comm. Code	Item	Description	Quantity	Unit	Unit Rate	Cost	Total Cost	Rate Reference
1. TSF CAPITAL COSTS											
				Closure Cost						$ 2,598,154	
3	515	B	BCA	101	Soil Cover: load, haul, place, 0.3 m thick	463,680	m^3	$ 1.27	$588,874		Pennecon
3	515	B	BVC	102	Hydroseed cover area	1,545,600	m^2	$ 1.30	$2,009,280		GOLDER
						TOTAL COSTS - PRE CONTINGENCY				$ 2,598,154	

Yearly Operating and Engineering Costs - Thickened

	Yr 0	Yr 1	Yr 2	Yr 3	Yr 4	Yr 5	Yr 6	Yr 7	Yr 8	Yr 9	Yr 10	Yr 11	Yr 12	Yr 13	TOTAL
Contractors/Consultants															
Full Time Construction CQA	$1,000,000	-		$1,000,000		$1,000,000		$1,000,000	-	-	-	-	-	-	$4,000,000
Engineering - Raise Design	$250,000	-	$250,000	-	$250,000	-	$250,000	-	-	-	-	-	-	-	$1,000,000
InPit Disposal Studies + Design	-	-	-	-	-	-	-		$700,000		-	-	-	-	$700,000
InPit Disposal CQA	-	-	-	-	-	-	-	-		$350,000	-	-	-	-	$350,000
Dam Safety Review	-	$100,000	-	-	-	-	$100,000	-	-	-	-	$100,000	-	-	$300,000
Dam Safety Inspection	-	$30,000	$30,000	$30,000	$30,000	$30,000	$30,000	$30,000	$30,000	$30,000	$30,000	$30,000	$30,000	$30,000	$390,000
OMS Manual udpate	-	$5,000	$5,000	$5,000	$5,000	$5,000	$5,000	$5,000	$30,000	$5,000	$5,000	$5,000	$5,000	$5,000	$90,000
Vegetation control	-	-	$10,000	-	$10,000	-	$10,000	-	$10,000	-	$10,000	-	$10,000	$10,000	$70,000
TMF Operations															
Tailings Engineer / Responsible person	-	$200,000	$200,000	$200,000	$200,000	$200,000	$200,000	$200,000	$200,000	$200,000	$200,000	$200,000	$200,000	$200,000	$2,600,000
TMF Technician	-	$60,000	$60,000	$60,000	$60,000	$60,000	$60,000	$60,000	$60,000	$60,000	$60,000	$60,000	$60,000	$60,000	$780,000
Light Vehicle (purchase, maintenance, fuel, insurance)	-	$110,000	$50,000	$50,000	$50,000	$110,000	$50,000	$50,000	$50,000	$50,000	$50,000	$50,000	$50,000	$50,000	$770,000
Water Quality Monitoring	-	$140,000	$140,000	$140,000	$140,000	$140,000	$140,000	$140,000	$140,000	$140,000	$140,000	$140,000	$140,000	$140,000	$1,820,000
Camp/Lodging	-	$80,000	$80,000	$80,000	$80,000	$80,000	$80,000	$80,000	$80,000	$80,000	$80,000	$80,000	$80,000	$80,000	$1,040,000
CAD	$1,250,000	$725,000	$825,000	$1,565,000	$825,000	$1,625,000	$925,000	$1,565,000	$1,300,000	$915,000	$575,000	$665,000	$575,000	$575,000	$13,910,000

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APPENDIX 2C

Prefeasibility Geotechnical Investigation: Marathon and Leprechaun Pits (Terrane)

TERRANE
GEOSCIENCE INC.

MARATHON GOLD CORPORATION
VALENTINE GOLD PROJECT

PRE-FEASIBILITY GEOTECHNICAL INVESTIGATION: MARATHON & LEPRECHAUN DEPOSITS

PREPARED FOR:
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SUMMARY

Terrane Geoscience Inc. (Terrane) was engaged by Marathon Gold Corp., (Marathon) to complete a pre-feasibility geotechnical assessment for their Valentine Gold Project located in Central Newfoundland, Newfoundland and Labrador, Canada. The Valentine Gold Project is in the west-central region of the island of Newfoundland, Newfoundland and Labrador, Canada. The project is situated approximately 90 km southwest of Millertown, NL and is accessible year-round by gravel road.

Four gold deposits have been discovered on the property, from south to north these are: The Leprechaun, Marathon, Sprite, and Victory deposits. This geotechnical study is focusing only on the Marathon and Leprechaun deposits.

Terrane completed the following Scope of Work as part of this pre-feasibility project:

- Review of background information,
- Geotechnical oriented drill program and field data collection,
- Performed geomechanical laboratory testing,
- Developed a 3D geomechanical fault model,
- Completed hydrogeology field data collection, analysis and reporting,
- Rock mass characterization and divided the pit into geotechnical domains,
- Pit slope design recommendations.

The geotechnical field data collection program was designed with the aim of characterizing the rock mass, structural fabrics, and major structures associated with both the Marathon and Leprechaun proposed open pits. The field data collection program consisted of geotechnical logging, index strength testing (i.e. point load testing and/or rebound hammer), packer testing, scan-line mapping, geomechanical sample collection, and optical/acoustic televiewer surveying. The geotechnical field program was completed between July 7 and August 23 of 2019.

The Marathon deposit is underlain by three primary rock types; mafic intrusive, quartz eye porphyry, and conglomerate. An analysis of the RMR_{76} data for each rock type indicates that the rock mass at the Marathon deposit displays a normal distribution and ranges from 55 – 77 with a mean value of 67. This mean corresponds to rock mass quality of Good.

Bedrock in the Leprechaun deposit area primarily consists of trondhjemite and conglomerate lithologies. An analysis of the RMR_{76} data for each rock type indicates that the rock mass at the Leprechaun deposit display a normal distribution and range from 60 – 82 with a mean value of 71. This mean corresponds to rock mass quality of Good.

Our slope design recommendations are summarized below for the Marathon deposit.

Summary Open Pit Mine Recommendations - Marathon

Design Sector	Design Area	Bench Face Angle (BFA°)	Inter-Ramp Angle (IRA°)	Overall Slope Angle (OSA°)1.	Catch-Bench Width (m)2.	Overall Height (m)
1	NW	71	51.5	47.0	8.1	295
2	NW	71	51.5	47.0	8.1	295
3	NW	71	51.5	47.0	8.1	295
4	SE	71	51.5	47.0	8.1	295
5	SE	71	51.5	47.0	8.1	295
6	SE	71	51.5	47.0	8.1	295
7	SW	75	54.3	52.0	8.1	270
8	SW	75	54.3	52.0	8.1	270
9	SW	75	54.3	52.0	8.1	270

Notes: 1. A geotechnical berm or a ramp after a vertical height of 90 m. OSA assumed to be equal to or less than value presented.

2. Based in 18 m (triple 6 m) benches; using Ryan and Pryor (2000), Bench width (m)=0.2 x height +4.5.

Our slope design recommendations are summarized below for the Leprechaun deposit.

Summary Open Pit Mine Recommendations - Leprechaun

Design Sector	Design Area	Bench Face Angle (BFA°)	Inter-Ramp Angle (IRA°)	Overall Slope Angle (OSA°)1.	Catch-Bench Width (m)2.	Overall Height (m)
1	NW	75	54.3	50.6	8.1	285
2	NW	75	54.3	50.6	8.1	285
3	NW	75	54.3	50.6	8.1	285
4	SE	65	47.4	44.4	8.1	285
5	SE	65	47.4	44.4	8.1	285
6	SE	65	47.4	44.4	8.1	285
7	NW	75	54.3	50.6	8.1	285
8	NW	75	54.3	50.6	8.1	285
9	NW	75	54.3	50.6	8.1	285

Notes: 1. A geotechnical berm or a ramp after a vertical height of 126 m. OSA assumed to be equal to or less than value presented.

2. Based in 18 m (triple 6 m) benches; using Ryan and Pryor (2000), Bench width (m)=0.2 x height +4.5.

These open pit slope design recommendations are based upon the geological, structural, geomechanical, and hydrogeological data presented herein. This design assumes that controlled blasting, geotechnical monitoring, and on-going data collection will be completed throughout the life of the mine.

1.0 INTRODUCTION AND PROJECT DESCRIPTION

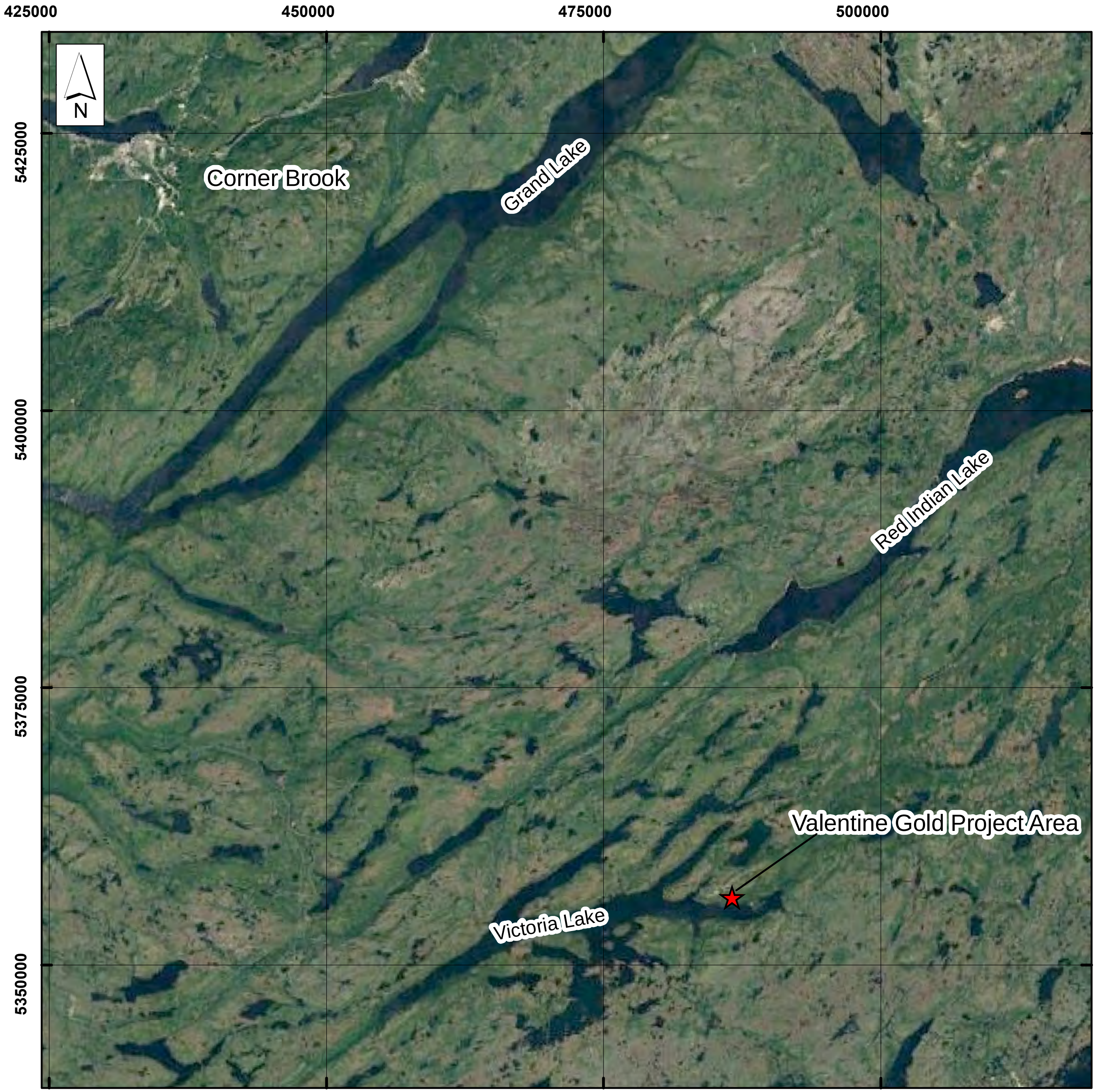
Terrane Geoscience Inc. (Terrane) was engaged by Marathon Gold Corp., (Marathon) to complete a pre-feasibility geotechnical assessment for their Valentine Gold Project located in Central Newfoundland, Newfoundland and Labrador, Canada (Figure 1). For the purposes of this assessment, the Valentine Gold Project refers to the Marathon and Leprechaun deposits.

This report includes Terrane's Scope of Work, a review of open pit slope design criteria, geologic setting, field and laboratory data collection, geotechnical data analysis, and recommendations for open pit slope design and future geotechnical work.

The Valentine Gold Project is in the west-central region of the island of Newfoundland, Newfoundland and Labrador, Canada. The project is situated approximately 90 kilometres (km) southwest of Millertown, NL and is accessible year-round by gravel road. To date, a total of four gold deposits have been identified within the project area, these being the Leprechaun, Marathon, Sprite, and Victory deposits. Marathon is currently completing a pre-feasibility study on the Marathon and Leprechaun deposits (Figure 2). Marathon has built an exploration camp on the property near Victoria Lake. Roads from the exploration camp allow for access to both the Marathon and Leprechaun deposits and other prospects on the property (Lincoln et al., 2018).

The Marathon deposit occurs approximately 6 km northeast of the Marathon Camp and is reported (Lincoln et al., 2018) to contain a total open pit resource of (measured plus indicated) 33,848,000 tonnes of ore grading 1.693 grams per tonne (g/t) gold for a total of 1,842,700 oz of gold. The current proposed pit (economic pit shell) measures approximately 1,250 metres (m) in length from northeast – southwest, approximately 650 m in width from northwest – southeast, and approximately 375 m vertical in depth (Lincoln et al., 2018).

The Leprechaun deposit occurs approximately 3.5 km west-northwest of the Marathon Camp and is reported (Lincoln et al., 2018) to contain a total open pit resource (measured plus indicated) 8,770,000 tonnes of ore grading 2.221 g/t gold for a total of 626,300 oz of gold. The current proposed pit (economic pit shell) measures approximately 950 m in length from northeast – southwest, 650 m in width from northwest – southeast, and approximately 320 m vertical in depth (Lincoln et al., 2018).



MARATHON GOLD

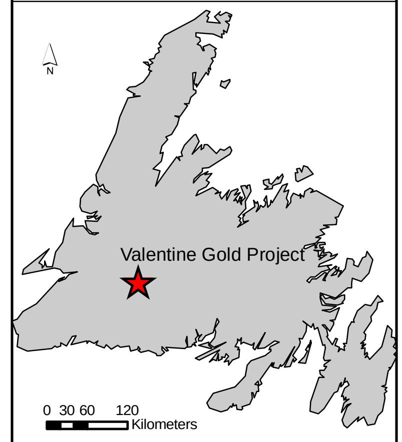


Figure 1 -
Project Location Map

Date: 01/10/2019	Drawn By: ATG
Checked By: TLG	NAD 83 Zone 21 North

Scale 1 : 500,000

0 2 4 8 12 16 Kilometers

VALENTINE GOLD PROJECT
PRE-FEASIBILITY
GEOTECHNICAL INVESTIGATION

Project # 19-0015-H

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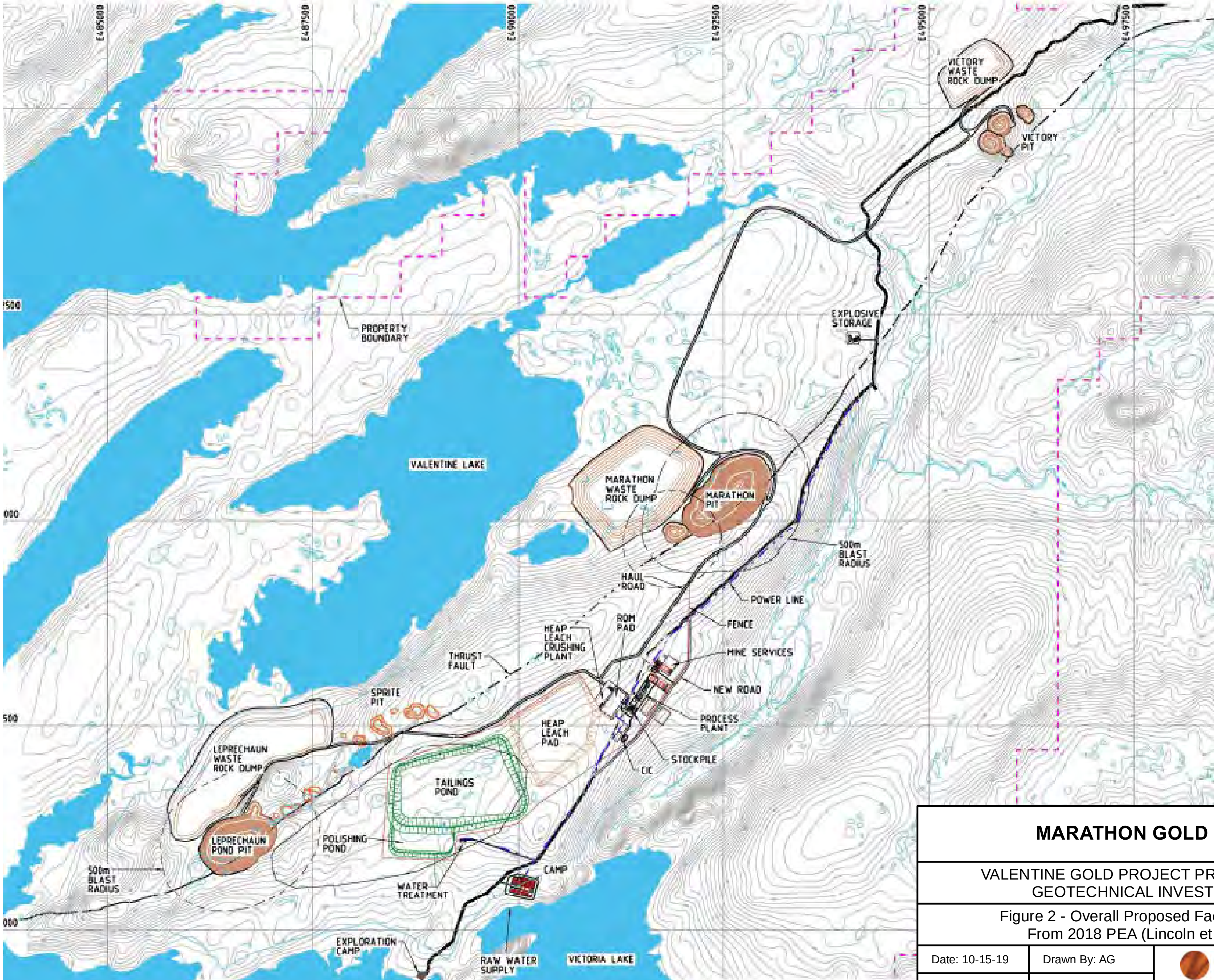


**Adapted from Lincoln et al., 2018. Technical Report Preliminary Economic Assessment of the Valentine Lake Gold Project.*

MARATHON GOLD CORP.		
VALENTINE GOLD PROJECT PRE-FEASIBILITY GEOTECHNICAL INVESTIGATION		
Figure 2 - Overall Proposed Facility Site Plan From 2018 PEA (Lincoln et al., 2018)		
Date: 10-15-19	Drawn By: AG	TERRANE GEOSCIENCE INC. www.terranegeoscience.com
REV 0.0	Checked By: TLG	
NO. 19-0015-H	Approved By: TLG	

2.0 SCOPE OF WORK

Terrane completed the following Scope of Work as part of this pre-feasibility project:

- Review of background information,
- Design of a geotechnical drill program,
- Geotechnical oriented drill program and field data collection,
- Collected samples of geotechnical drill holes and performed geomechanical laboratory testing,
- Developed a 3D geomechanical fault model,
- Completed hydrogeology field data collection, analysis and reporting,
- Rock mass characterization and divided the pit into geotechnical domains,
- Compiled the available data into a geotechnical 3D model,
- Completed kinematic and numerical stability modelling to allow for pit slope design recommendations.

3.0 OPEN PIT SLOPE DESIGN OVERVIEW

3.1 General

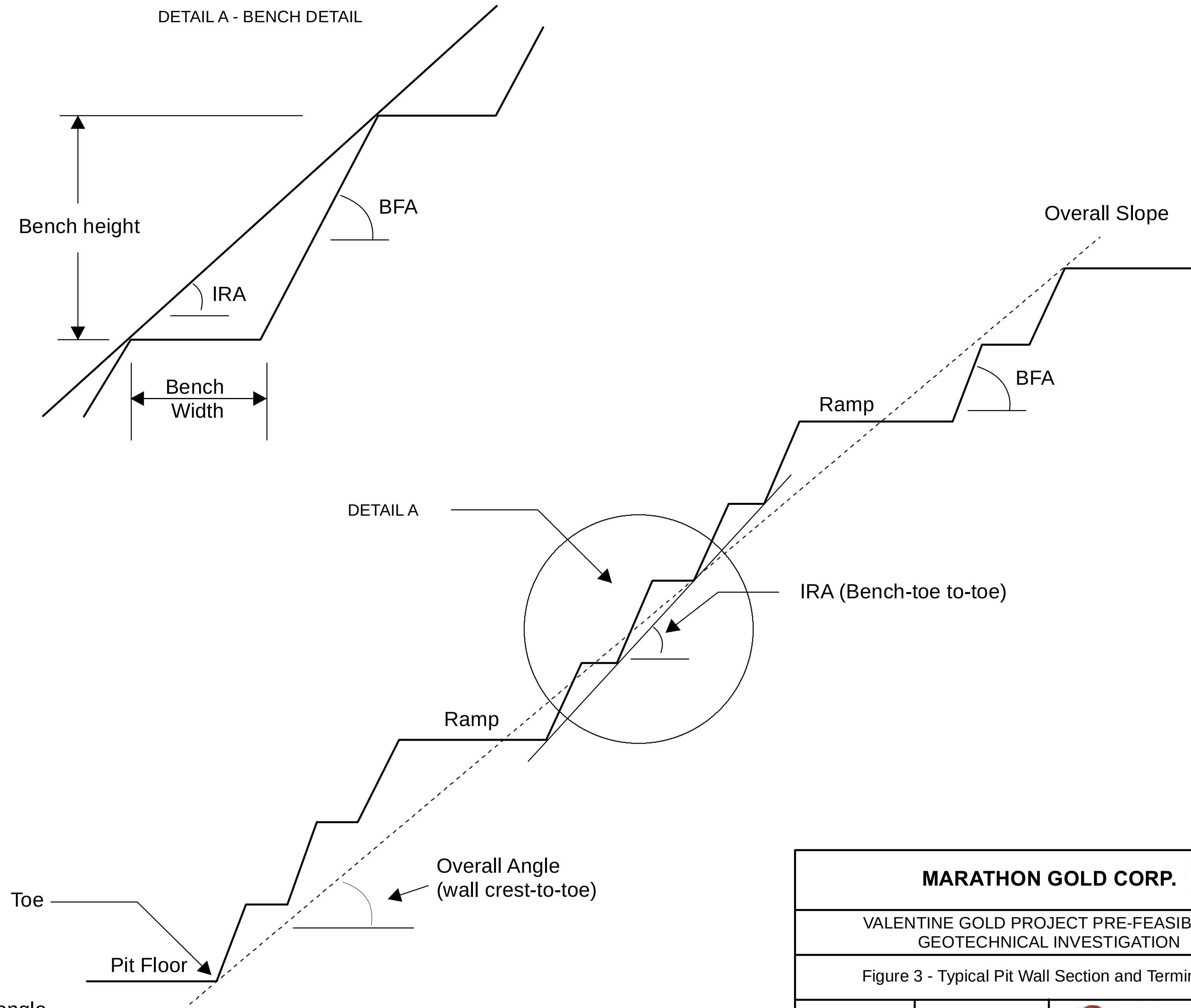
The objective of any open pit slope design is to provide an optimal excavation that leads to the steepest possible open pit slopes while ensuring that safety, ore recovery and financial return are maximized. Weighed against this objective, is the potential that steeper slopes may result in slope stability issues. Such slope stability issues could impact worker safety, ore recovery, and ultimately the financial viability of the project (Read and Stacey, 2009).

Generally, slope design takes into consideration an analysis of the overall slope stability of a pit wall (i.e. all the benches and ramps from the pit floor to the surface) and the bench design (i.e. bench width, bench face angle, and bench height). From these analyses the overall slope angle, inter-ramp angle and the bench face angles are designed based on achieving an acceptance Factor of Safety (FOS). From an operational perspective, the open pit slopes are considered too conservative if no instabilities occur. As a result, some instability is expected and is planned to be controlled during open pit development (Read and Stacey, 2009).

The following sections summarize open pit slope terminology and the key geotechnical and operational factors that affect open pit design and introduce the pit slope design methodology used for this pre-feasibility study.

3.2 Open Pit Slope Design Terminology

Figure 3 displays a typical open pit slope section that shows the relationship between bench geometry (bench face angle and bench width), inter-ramp angle (i.e. bench toe-to-bench toe), and overall pit slope angle (i.e. wall crest-to-toe).



BFA - Bench face angle
IRA - Inter-ramp slope angle
Overall - Overall slope angle

MARATHON GOLD CORP.		
VALENTINE GOLD PROJECT PRE-FEASIBILITY GEOTECHNICAL INVESTIGATION		
Figure 3 - Typical Pit Wall Section and Terminology		
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The primary elements of an open pit design are (Figure 3):

Bench Configuration – The bench is composed of two main components, the bench face which is defined by the bench height and the bench face angle (BFA) and the bench width. The bench height is generally determined by the equipment chosen for mining. Double or triple benches are common and result in steeper inter-ramp and overall slope angles. The BFA is designed based an acceptable level of instability based on the acceptance criteria discussed above. The bench width is designed to capture bench scale wedges and/or blocks to prevent them from falling down the slope. The combination of the bench face height, BFA, and bench width dictates the inter-ramp angles.

Inter-Ramp Slope – As mentioned above, the maximum inter-ramp angle is controlled by the bench height and the BFA and is measured from bench-toe toe. However, from a design perspective, it is necessary to evaluate inter-ramp instabilities in relation to large-scale (multi-bench) major structures (e.g. faults/shear zones and persistent joints). In some cases, such structures may dictate the achievable inter-ramp angle resulting in a flattened slope.

Overall Slope – The overall slope angle is measured from the wall crest-to-toe and is generally flatter than the inter-ramp angle. Additionally, the width of the ramp, any geotechnical berms and the rock mass strength may act to reduce the overall slope angle.

3.3 Open Pit Slope Design Methodology

Once an open pit has been divided into geotechnical domains (areas of similar rock mass quality and structural setting), the pit can be broken into design sectors based on pit wall geometry. Following this, stability analysis can be undertaken for each design sector at the bench, inter-ramp, and overall pit scales. Herein we have completed the following stability analysis:

Kinematic Stability Analyses – Stereographic analysis of discontinuity orientation data are conducted to identify kinematically possible failure modes. Kinematic failure modes, at the bench scale, are anticipated to potentially affect the slopes locally. Bench face angles could be designed to avoid all possible failures; however, this would result in uneconomically flat slope angles. The design herein assumes that some intermittent failures will occur, and unstable blocks can be controlled by scaling and catchment on the bench. If structures are persistent enough to cut multiple benches, then kinematic failure modes at the inter-ramp scale must also be evaluated.

Numerical Modelling (Rock Mass Stability) Analyses – The overall factor of safety against large-scale, multi-bench rock mass failure is evaluated using a limit equilibrium approach. Using the acceptance criteria outlined by Read and Stacey (2009) a minimum FoS of at least 1.3 has been used for our analyses.

4.0 GEOLOGIC SETTING

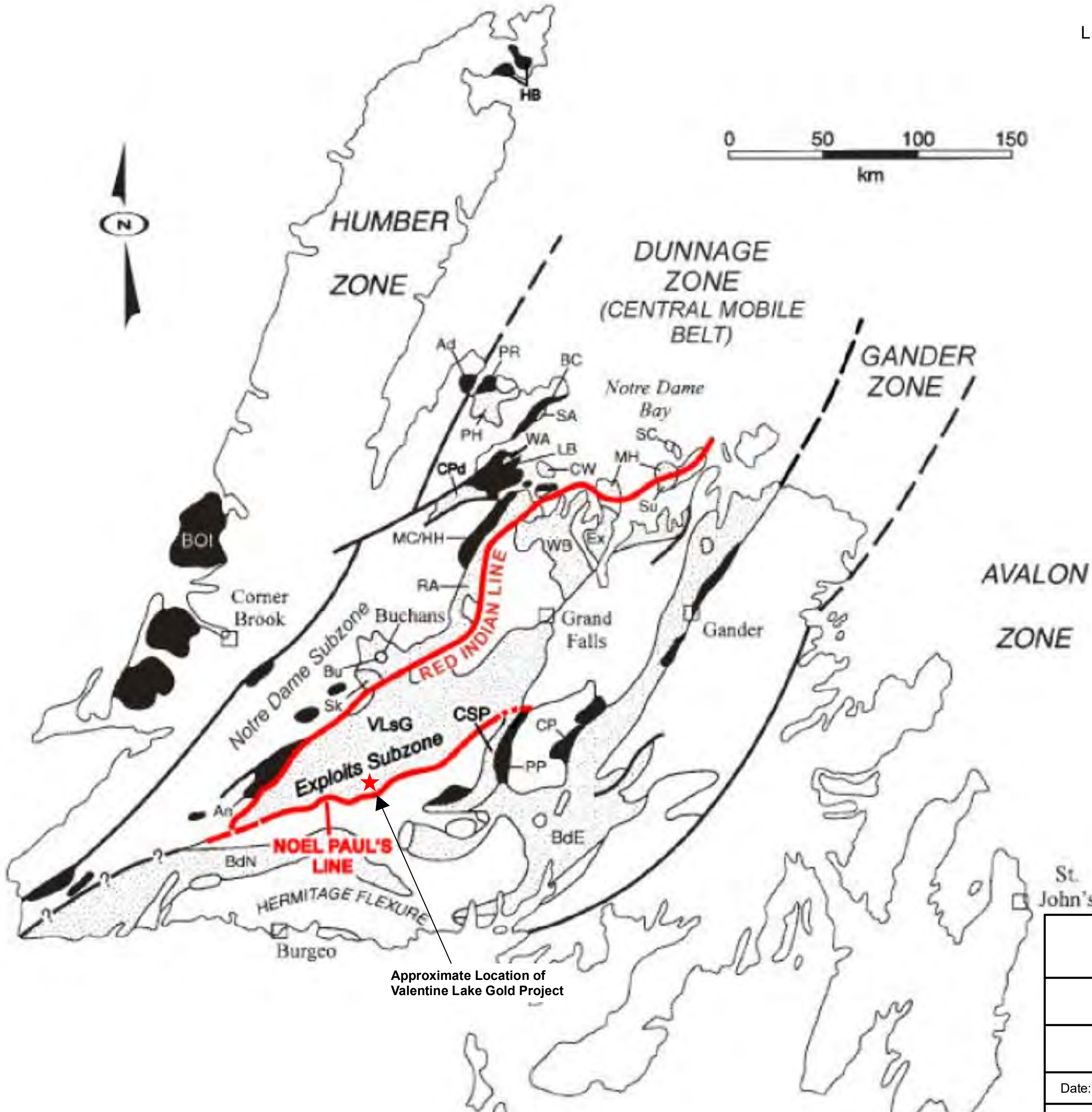
4.1 Tectonic Setting

The Valentine Gold Project is located on the island of Newfoundland within the northeastern segment of the Appalachian Orogen. Within Newfoundland, the Appalachian Orogen has been sub-divided into four tectonostratigraphic terranes, these being, from west to east, the Humber, Dunnage, Gander and Avalon (Williams, 1979; Figure 4). The Humber zone, west of the project area, is comprised of Paleozoic sedimentary rocks that lie unconformably on pre-Cambrian Grenville basement rocks at the eastern margin of the North American (Laurentian) continent. East of the study area lies the Gander terrane, which encompasses sedimentary rocks deposited in proximity to the African (Gondwanan) continental margin (Blackwood, 1982). In the middle of these two continental margin terranes, lies the Dunnage zone. The Dunnage terrane is characterized by a series of east vergent, structurally telescoped assemblages of ophiolitic and arc to back-arc volcanic rocks, with volcaniclastic to epiclastic sedimentary rocks. Collectively, these rocks of the Dunnage terrane represent remnants of early to middle Paleozoic oceanic terranes (Squires, 2005).

The Dunnage zone is broken into two fault bounded subzones, with an island scale fault, the Red Indian line (RIL) cutting the Dunnage terrane from southwest to northeast. The Dunnage zone north of the RIL is termed the Notre Dame Subzone, and the portion south and east of the RIL is known as the Exploits Subzone. These subzones are inferred to have formed on either side of the Iapetus Ocean. The Dunnage zone was subjected to later deformation in the Silurian by the Salinic orogeny. Gold mineralization within the Dunnage tectonostratigraphic terrane occurs coincident with late syn- to post-Salinic orogenic events and is typically associated with major structural features (Lincoln et al., 2018). The area of interest (Figures 4 and 5) is located within the Exploits Subzone near the contact with the Valentine Lake Thrust Fault.

4.2 Regional Geology

The project area largely consists of the Victoria Lake Group volcanic and epiclastic units that are intruded by later granitoid and gabbroic intrusions of Cambrian to Silurian age (Figure 5). The project area is located within the large multiphase trondhjemite, quartz monazite, and gabbro Victoria Lake Intrusive Complex (VLIC) which forms a structural inlier within the Victoria Lake Group volcano-sedimentary package (Lincoln et al., 2018). The VLIC is located along the contact between the Victoria Lake Group to the northwest and the Rogerson Lake Conglomerate to the southeast.



LEGEND

Black Areas Ophiolite

Stippled Areas Underlain by Iapetan Volcanic and Volcaniclastic Rocks

Ad-Advocate Cplx.;
An-Annieopsquotch Cplx.;
BC-Betts Cove Cplx.;
BdE-Baie D'Espoir Gp.;
BdN-Bay du Nord Gp.;
BOI-Bay of Islands Cplx,;
Bu-Buchans Gp.; CP-Coy Pond Cplx.;
CPd- Catcher's Pond Gp.;
CSP-Cold Spring Pond Fm.;
CW-Cutwell Gp.; Ex-Exploits Gp.;
HB-Hare Bay Ophiolite; LB-Lushs Bight Gp.;
MC/HH-Mansfield Cove/Hall Hill;
MH-Morton's Harbour Gp.;
PH-Pacquet Harbour Gp.;
PP-Pipestone Pond Cplx.;
PR-Point Rousse Cplx.;
RA-Roberts Arm Gp.;
SA-Snooks Arm Gp.;
SC-Sleepy Cove Gp.;
Sk-Skidder basalt.;
Su-Summerford Gp.;
VLsG-Victoria Lake Supergroup;
WA-Western Arm Gp.;
WB-Wild Bight Gp.

MARATHON GOLD CORP.

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Figure 4 - Tectonostratigraphic zones of
the Island of Newfoundland, NL.

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***Adapted from Evans and Kean (2002).**
The Victoria Lake Supergroup, Central Newfoundland.

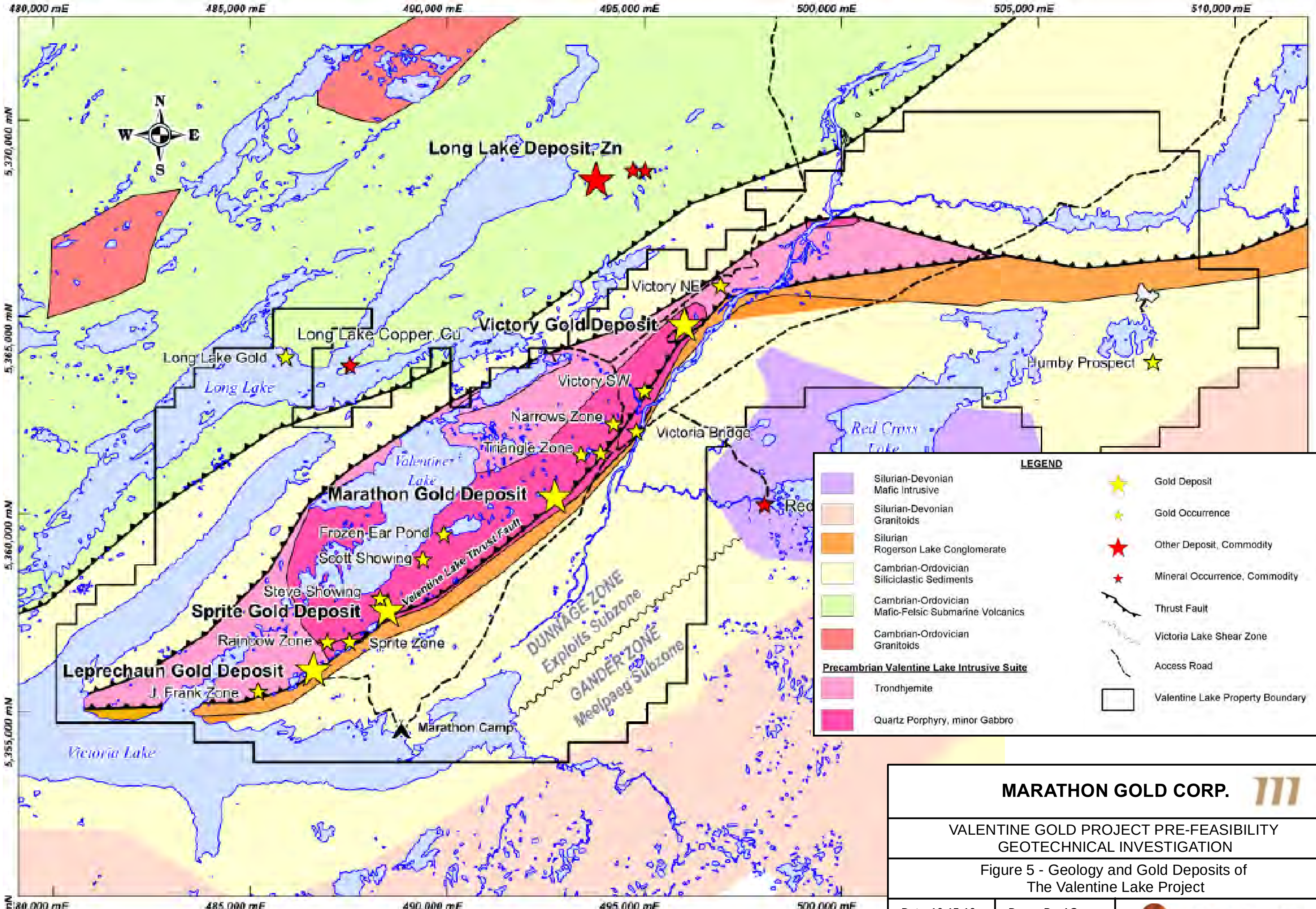

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Figure 5 - Geology and Gold Deposits of
The Valentine Lake Project

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**Adapted from Lincoln et al., 2018. Technical Report Preliminary Economic Assessment of the Valentine Lake Gold Project and Marathon Gold files.*

At the deposit scale the project is hosted by an elongated (~22 km long by 4.5 km wide) zoned intrusion of the VLIC. This zoned intrusion occurs within the Victoria Lake Group and is host to most of the mineralization on the property including the Marathon and Leprechaun deposits. Quartz porphyry monzonite and trondhjemite are the dominant lithologies present. The VLIC is unconformably overlain by younger Rogerson Lake Conglomerate. Bedrock within the project area is generally overlain by glacial till up to 4 m thick and/or boggy organic soil, locally (Lincoln et al., 2018).

Four gold deposits have been discovered on the property, from south to north these are: The Leprechaun, Marathon, Sprite, and Victory deposits. This geotechnical study is focusing only on the Marathon and Leprechaun deposits.

4.3 Structural Geology

Barbour (1990) interprets the deformation in the Valentine Lake area to be the result of a single episode (D1) of east to southeast vergent transpression which produced a strong penetrative s-fabric associated with a prominent flattening fabric and stretching lineation.

Hrabi and Siddorn (2013) interpret two phases of ductile to brittle ductile deformation:

D1 – Fabrics related to D1 include a regional, northeast – southwest trending, penetrative foliation (S_1) and associated moderately north-northeast-plunging stretching lineation (L_1). D1 is interpreted to be the result of east-vergent thrusting during Salinic orogenesis, with a subordinate component of left-lateral displacement. Deformational styles were reported to vary from mainly brittle in the intrusive rocks to ductile in the conglomerate unit.

D2 – D2 structures and fabrics include isoclinal folds of early quartz veins and a secondary generation of folding. S_2 cleavage developed axial planar to second-generation folds in ductile lithologies and is locally moderately dipping to the northeast. Mafic dykes north of the Leprechaun deposit area preserve fold patterns consistent with formation during this deformational event. D2 deformation is interpreted as a possible later, progressive stage of D1 (Hrabi and Siddorn, 2013).

Large- scale structures (10’s of km) in the area are dominated by the Valentine Lake Thrust Fault (Dunswoth and Walford, 2018). This fault strikes northeast – southwest, is subvertical to steeply northwest dipping, and marks the contact between the VLIC and the Rogerson Lake Conglomerate. Kinematic observations of the Valentine Lake Thrust Fault indicate oblique, sinistral- reverse movement along the fault (Lincoln et al., 2018).

Modelling of late brittle structures by Terrane based on topographic (LiDAR) and magnetic lineaments, geotechnical drilling, exploration structural geology logging, and drill hole RQD indicated a pattern of first and second-order faults intersecting about a steep to moderately N-plunging axis (see Sections 7.2 below). Superficially, the modelled fault pattern suggests a late strike-slip system may have been active, post-dating D1 and D2 deformation. However, further structural investigation is required to test this hypothesis.

5.0 FIELD DATA COLLECTION

The geotechnical field data collection program was designed with the aim of characterizing the rock mass, structural fabrics, and major structures associated with both the Marathon and Leprechaun proposed open pits. This data was collected with the intent of developing a geotechnical model suitable for open pit slope design. The field data collection program consisted of geotechnical logging, index strength testing (i.e. point load testing and/or rebound hammer), packer testing, scan-line mapping, geomechanical sample collection, and optical/acoustic televiewer surveying. The geotechnical field program was completed between July 7 and August 23 of 2019.

5.1 Geotechnical Oriented Core Logging

Drilling was completed by RnR Diamond Drilling, Springdale, NL in two, 12-hour shifts/day. A total of seven, HQ3 size (61.1 mm) triple tube holes were completed totaling 2,061 m and split between the Marathon and Leprechaun deposits. A total of 1,060.5 m was drilled at Marathon and 1,001 m at Leprechaun. Drillcore was oriented (where possible) using a Reflex Instruments ACT III orientation instrument combined with a Sprint IQ Gyro non-magnetic down hole survey tool. Continuous measurements for dip and azimuth were collected with the Gyro along the entire length of each hole. All core was logged at Marathon's core logging facility at their Valentine Lake camp. Geotechnical core logging was completed by an Intermediate Geological Engineer from Terrane with assistance from a Marathon geologist for the Marathon deposit holes and assistance by a Terrane Junior Geological Engineer for the Leprechaun deposit holes.

All core logging was completed in accordance with accepted geotechnical logging standards and included the collection of the required parameters that enabled the calculation of RMR_{76} (Bieniawski, 1976). The logging consisted of interval logging or detailed logging of each core run. Each core run was 3 m long using a standard core barrel. For interval logging, data was collected on core recovery, RQD, discontinuity characteristics (e.g. alteration, weathering, and infill) and fracture counts.

In addition to interval logging, Terrane completed discrete logging of all orientated core. This included measurement of the alpha and beta angles to evaluate structural fabric orientations. Logging also included measuring all brittle discontinuities and faults encountered within the geotechnical drill core.

5.1.1 *Marathon Drilling*

Geotechnical drilling at Marathon consisted of three holes totaling 1,060.5 m (Figure 6). Collar locations, azimuth, dip and total length for each hole is summarized in Table 1. Geotechnical core logs are included in Appendix A.

Table 1 – Summary of Geotechnical Drillholes – Marathon Deposit

Drill Hole ID[1.]	Easting[2.]	Northing[2.]	Elevation (m)	Collar Azimuth (°)	Dip (°)	End of Hole (m)
MA-GT-19-05	492,406	5,360,533	335.9	128.7	60.3	399.5
MA-GT-19-06	492,596	5,359,896	374.7	324.5	52.5	260.0
MA-GT-19-07	492,936	5,360,740	338.5	218.7	51.8	401.0

Notes: 1. MA – Marathon Deposit, 2. NAD83 UTM Zone 21 North

5.1.2 *Leprechaun Drilling*

Leprechaun geotechnical drilling consisted of four holes totaling 1,001 m (Figure 7). Collar locations, azimuth, dip and total length for each hole is summarized in Table 2. Geotechnical core logs are included in Appendix A.

Table 2 - Summary of Geotechnical Drillholes - Leprechaun Deposit

Drill Hole ID[1.]	Easting[2.]	Northing[2.]	Elevation (m)	Collar Azimuth (°)	Dip (°)	End of Hole (m)
VL-GT-19-01	486,294	5,356,228	397.1	115.2	54.8	374.0
VL-GT-19-02	486,902	5,355,940	403.8	299.5	45.2	401.0
VL-GT-19-03	486,274	5,355,785	386.0	150.7	64.9	101.0
VL-GT-19-04	486,886	5,356,272	394.4	59.1	69.3	125.0

Notes: 1. VL – Leprechaun Deposit, 2. NAD83 UTM Zone 21 North

5.2 Oriented Core

The core was oriented at the drill rig using the Reflex ACT III (ACT III) core orientation system. The ACT III system includes an accelerometer-based sensor that records the location of the bottom or keel of the core, such that it can be marked after removing it from the hole.

Discontinuity orientation data including alpha, beta, and depth was collected in each run for all open discontinuities as part of our discrete logging.

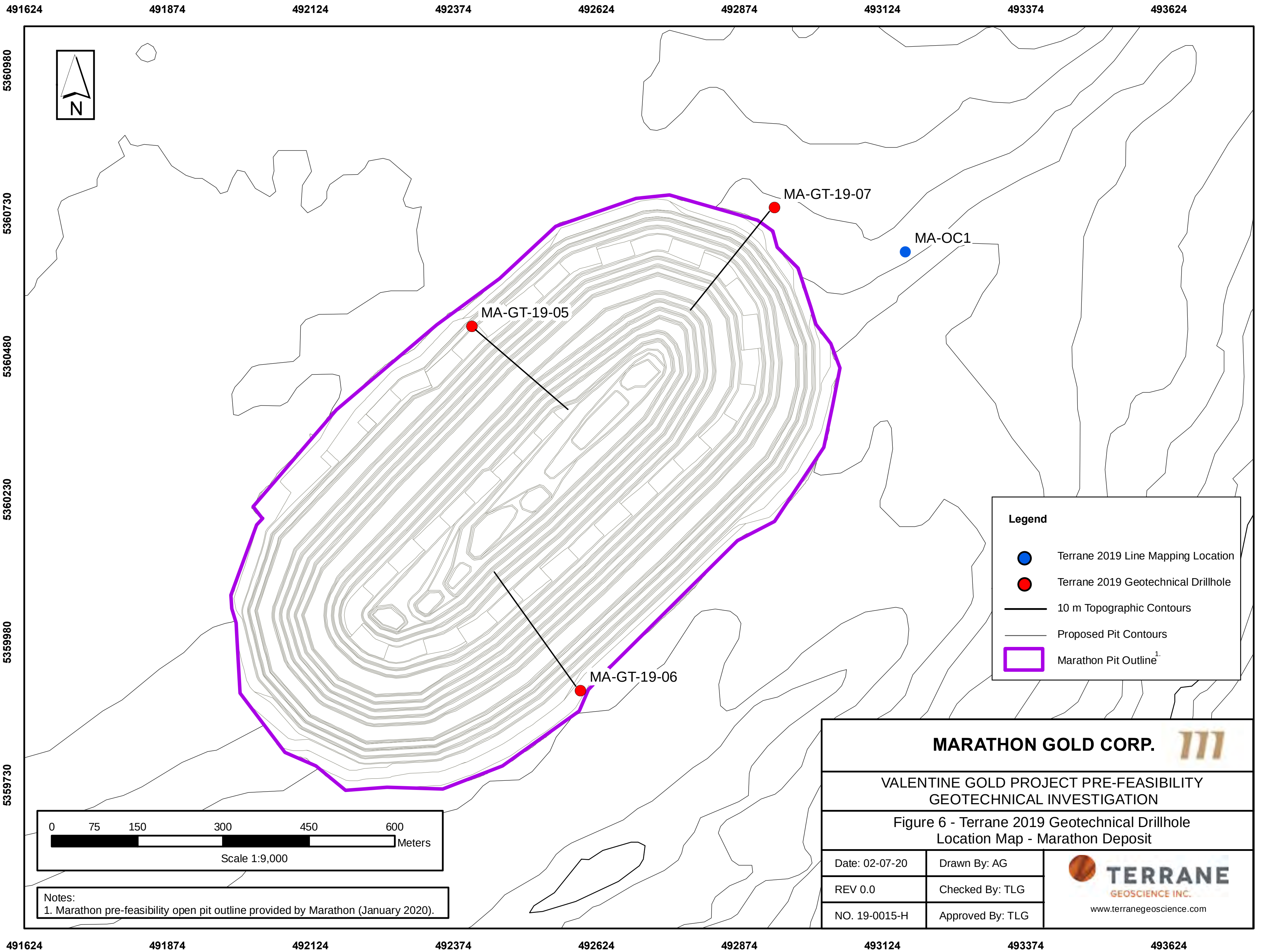

Figure 6 - Terrane 2019 Geotechnical Drillhole Location Map - Marathon Deposit

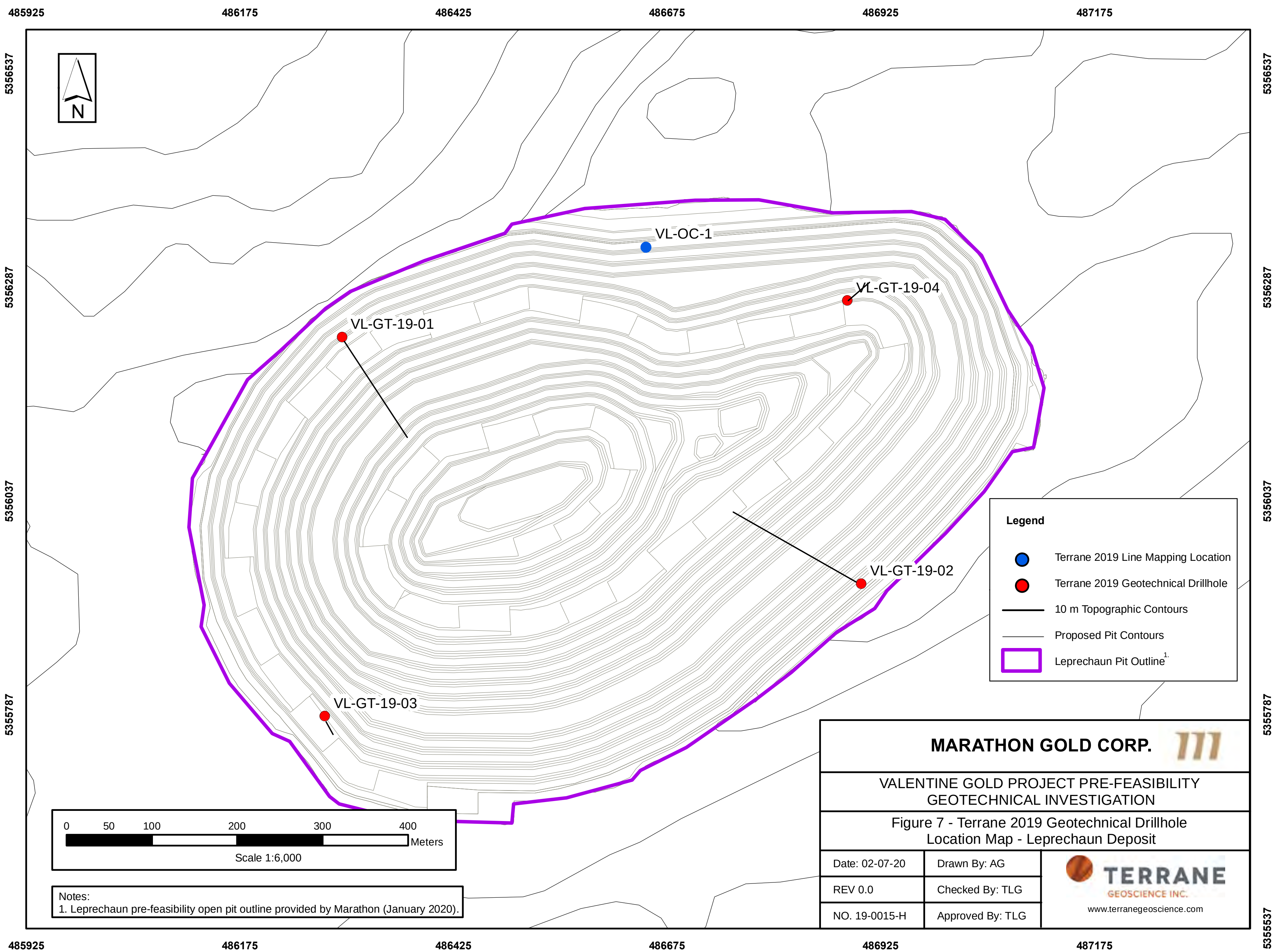

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Figure 7 - Terrane 2019 Geotechnical Drillhole Location Map - Leprechaun Deposit

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5.2.1 Oriented Core Data Analysis and QA/QC

A total of 6,600 discontinuities were measured, 3,722 at the Marathon deposit and 2,878 at the Leprechaun deposit. After measuring the alpha and beta angles and depth for each discontinuity, the data was analyzed in Rocscience DIPS (Rocscience, 2019) and converted to strike and dip. As a further QA/QC check the alpha and beta data was also imported into LeapfrogGeo as a secondary check on the alpha and beta conversion. It was found that the data interpretation from both software's agreed.

During discrete core logging, Terrane also collected data on orientation mark quality, line quality between adjacent runs. Collecting this data allowed for a QA/QC assessment of the oriented core data upon completion of logging.

Mark quality is a semi-quantitative assessment of the quality of the bottom orientation mark at the end of each core run. It varies from 0 for no mark, to 4 for an excellent quality mark with < 2° angular uncertainty. Table 3 displays the values assigned to each run based on mark quality and Table 4 displays a summary of the mark quality results for both the Marathon and Leprechaun deposits. At Marathon, 87% of the core runs have a mark quality of 2 (acceptable) or better, while at Leprechaun, 85% of runs have a mark quality of 2 or better.

Table 3 - Oriented Core Mark Quality Assessment

Mark Quality	Interpretation
0	No mark or spurious mark (e.g. loose and/or broken core at end of run)
1	Poor – Angular uncertainty > 10°
2	Acceptable – Angular uncertainty +/- 10°
3	Good - Angular uncertainty +/- 5°
4	Excellent - Angular uncertainty +/- 2°

Table 4 - Oriented Core Mark Quality Summary - Marathon and Leprechaun

Mark Quality	Marathon Deposit		Leprechaun Deposit	
	Core Runs for Each Mark Quality[1.]	% of Total for Each Mark Quality	Core Runs for Each Mark Quality[2.]	% of Total for Each Mark Quality
0	27	7.8	36	11
1	17	4.9	14	4.2
2	55	15.9	60	18.1
3	190	54.8	117	35.3
4	58	16.7	104	31.4

Notes: 1. Total number of Marathon orientated core runs, 347.

2. Total number of Leprechaun orientated core runs, 331.

Line quality is an assessment of the angular mis-match in the orientation line between adjacent core runs and how the runs align above and below the core run being logged. It varies from 0 for no line or a problematic line to 5 which represents a perfect alignment over three or more successive runs. Table 5 displays the values assigned to each run based on line quality and Table 6 displays a summary of the mark quality results for both the Marathon and Leprechaun deposits.

Table 5 - Oriented Core Line Quality Assessment

Line Quality	Interpretation
0	No line or clearly problematic line
1	Run locks with one or more runs with a lock angle > 20º
2	Run does not lock with another run, so it is not independently validated
3	Run locks with another run; lock angle between 10º – 20º
4	Run locks with another run; lock angle between 2º – 10º
5	Perfect lock with three or more runs

Table 6 - Oriented Core Line Quality Summary - Marathon and Leprechaun

Line Quality	Marathon Deposit		Leprechaun Deposit	
	Core Runs for Each Line Quality[1.]	% of Total for Each Line Quality	Core Runs for Each Mark Quality[2.]	% of Total for Each Mark Quality
0	5	1.4	22	6.6
1	38	11.0	36	10.9
2	99	28.5	77	23.2
3	36	10.4	39	11.8
4	146	42.1	143	43.2
5	23	6.6	14	4.2

Notes: 1. Total number of Marathon orientated core runs, 347.

2. Total number of Leprechaun orientated core runs, 331.

For the purposes of our geotechnical assessment Terrane has used all discontinuity orientation data that were assigned a line quality of three or greater. A total of 59% of the discontinuities measured at both the Marathon and Leprechaun deposit met this QAQC criteria.

Finally, the oriented core data for each drillhole was plotted to evaluate the orientation data for small circle artifacts. During the stereonet evaluation of orientation data for a given hole, data should be considered suspect if the poles to the strike and dip of each measurement cluster about a small circle on a stereonet with the trend and plunge of the drillhole at the centre. An evaluation of the Marathon and Leprechaun oriented core data did not display any small circle artifacts.

5.3 Optical and Acoustic Televiewer Surveying

Optical (OTV) and acoustic (ATV) downhole televiewer surveys were performed on each geotechnical drillhole completed on the Marathon and Leprechaun deposits. Televiewer surveying acts as a validation check on oriented core and provides additional orientation data for analyses. In total, Terrane completed 1,940.6 m of OTV surveying and 1,712.7 m of ATV surveying within seven drillholes. Table 7 summarizes the OTV and ATV data collection for both Leprechaun and Marathon.

Table 7 - Optical and Acoustic Televiewer Surveying Summary - Marathon and Leprechaun

Drill Hole ID	Deposit	Optical Surveyed (m)	Acoustic Surveyed (m)	No. of Discontinuities
VL-GT-19-1	Leprechaun	370.8	370.8	1,071
VL-GT-19-2	Leprechaun	395	180.5	383
VL-GT-19-3	Leprechaun	100.1	99.8	385
VL-GT-19-4	Leprechaun	122.9	122.9	309
MA-GT-19-5	Marathon	396.9	396.9	779
MA-GT-19-6	Marathon	254.9	148.3	456
MA-GT-1907	Marathon	300.0	393.5	553

The total number of meters drilled differs from the amount of televiewer surveying completed on the project. This occurred because of blockages or collapsed sections of the drillholes that did not allow the downhole survey equipment to pass; resulting in only a portion of the drillhole being surveyed.

5.4 Index Testing (Rebound Testing)

Rebound hammer testing was conducted on each 3 m run of drill core to develop a data base of indicated strengths along the entire length of the drillhole. Rebound hammer testing was developed as a tool to estimate the strength of concrete but has been adapted for use in rock mass classification.

It is also common to use point load testing (PLT) as a strength indicator in rock mass classification. Terrane began the geotechnical investigation using point load testing; however, given the relatively high rock strengths found that the correlations between PLT and uniaxial compressive strength (UCS) was not representative of lithologies being tested. Rebound hammer testing showed a better correlation between the index values returned for each test and the anticipated UCS values of the rocks.

A total of 675 rebound hammer tests were completed on geotechnical drillholes at Marathon and Leprechaun.

5.5 Scan Line Mapping

A total of four geotechnical scan lines were completed as part of the geotechnical investigation. Two scan lines near the proposed Marathon open pit and two near the proposed Leprechaun open pit. The intent of this mapping was to collect rock mass fabric orientation data, rock mass characteristics and where possible to observe major structures in the field.

Scan line mapping comprises stretching a tape along the length of the outcrop face and mapping all geological features or discontinuities which intersect the line. This type of mapping is considered a statistically representative sample of the discontinuities that make up an outcrop; however, it can be subject to orientation bias.

The results of geotechnical scan line mapping are included in Appendix A. The geotechnical line mapping exercise resulted in data collection on 204 discontinuities. Data collected included; discontinuity type, strike, dip, spacing, roughness, planarity, and persistence for each discontinuity. Additionally, descriptions of the rock types encountered were also noted.

5.6 Packer Testing

In-situ packer hydraulic conductivity testing was conducted in selected geotechnical drillholes in both the Marathon and Leprechaun proposed open pit areas during the geotechnical drilling program. The objective of the packer testing was to provide general estimates of hydraulic conductivity for the rock mass in the areas of each proposed pit that could be used for preliminary calculations of pit inflows. This packer testing was not extensive and included both intact rock and fractured rock zones.

5.6.1 Methods

The objective of the packer testing was to estimate the bulk hydraulic conductivity of the primary bedrock units associated with both Marathon and Leprechaun open pits and to determine if there was any variation associated with depth or the presence of various structural features (faults, fractures, shear zones). Five geotechnical drillholes were packer tested as part of the current program. The location of the drillholes are shown on Figures 6 and 7 (Appendix E) and included the following:

- Two (2) drillholes (MA-GT-19-05, and MA-GT-19-06) located along the proposed pit shell of the Marathon pit; and,
- Three (3) drillholes (VL-GT-19-01, VL-GT-19-02, and VL-GT-19-03) located along the proposed pit shell of the Leprechaun pit.

The packer tests were conducted using a Standard Wireline Packer System (SWiPS) manufactured by Inflatable Packers International (IPI). The tests were performed using the constant head (Lugeon) packer injection test method and utilized a single packer inserted through the HQ drill rods to test selected intervals as the hole was advanced.

A total of 10 packer tests were performed within these drillholes at selected intervals that covered the full planned depths of pit development; including six tests carried out in the Leprechaun pit at downhole depths ranging from 12 to 374 meters below ground surface (mbgs), and four tests in the Marathon pit at downhole depths ranging from 30 to 296 mbgs. The tests for the Leprechaun pit were all completed within the trondhjemite (TRJ) bedrock unit, which makes up the bulk of the pit's rock mass; while the tests for the Marathon Pit were limited to one or two tests for each of the three primary bedrock units making up the pit's rock mass (quartz-eye porphyry (QEP), conglomerate (CG) and mafic intrusive (MFI)

In addition, the Valentine Lake thrust fault was tested in packer test PT2 in drillhole VL-GT-19-03 (Leprechaun pit). An additional test location was selected to test the Valentine Lake thrust fault in drillhole MA-GT-19-06 (Marathon pit); however, a mechanical issue with the drill rig resulted in termination of this drillhole before reaching the target downhole depth of approximately 320 m.

Except for packer test PT2 in drillhole MA-GT-19-06 (Marathon pit), which had a test interval length of approximately 71 m, all other packer tests were performed over intervals ranging from 17 to 32 m long. Table 8 summarizes packer test information and results for this program. Packer test data are presented on the analysis reports in Appendix E.

Table 8 summarizes the Packer testing completed as part of this geotechnical investigation.

Table 8 - Summary of Packer Testing

Borehole ID[2.]	Packer Test (Lugeon)					Lithology	Hydraulic Conductivity (K) (m/s)		Notes
	Test ID	Tested Zone (m)[1]							
		From	To	Interval Length	Mid-point (tested zone)		Packer Test Average	Geometric mean	
VL-GT-19-01	PT1	30.3	50.0	19.7	40.2	Trondhjemite (TRJ)	2.51E-08	5.88E-08	
	PT2	150.3	170.0	19.7	160.2	Trondhjemite (TRJ)	5.79E-10		
	PT3	348.3	374.0	25.7	361.2	Trondhjemite (TRJ)	4.14E-08		Fault identified from 350 - 353 m; associated with good rock quality
VL-GT-19-02	PT1	354.3	374.0	19.7	364.2	Trondhjemite (TRJ), QTP	1.73E-07		
VL-GT-19-03	PT1	12.3	32.0	19.7	22.2	Trondhjemite (TRJ), QTP	2.36E-07		
	PT2	45.3	77.0	31.7	61.2	Trondhjemite (TRJ); Mafic Dyke (MD)	1.69E-06		Fault identified from 44 - 50 m; associated with fair - poor rock quality
MA-GT-19-05	PT1	45.3	62	16.7	53.7	Mafic Dyke (MD)	8.73E-07	4.49E-08	Flowing artesian conditions; static water level not determined
	PT2	273.3	296	22.7	284.7	Quartz Eye Porphyry (QE-POR)	3.90E-10		Flowing artesian conditions; static water level not determined
MA-GT-19-06	PT1	30.3	50	19.7	40.2	Conglomerate (CG)	1.93E-06		
	PT2	189.3	260	70.7	224.7	Conglomerate (CG); Mafic Dyke (MD)	6.18E-09		Fault identified from 233 - 236 m; associated with good rock quality

Notes:

1. Depth measurements are referenced with respect to ground surface, and are inclined drill hole depths

2. VL - Leprechaun, MA - Marathon

6.0 GEOMECHANICAL LABORTORY TESTING

Geomechanical laboratory testing was completed by Natural Resources Canada at their CanmetMINING Rock Mechanics Laboratory (Canmet), Ottawa, Ontario. The testing conducted included; unconfined compressive strength testing (UCS), Brazilian tensile strength testing, direct shear strength testing, and triaxial (confined) compressive strength testing (TCS). In total 41 geomechnical tests were completed on the 7 drillholes, 19 samples from the Marathon deposit and 22 samples from the Leprechaun deposit.

The results of the geomechanical testing were analyzed by Terrane and used in our open pit slope stability design. A report from Canmet titled, *Geomechanical Lab Testing of Rock Core, Marathon Gold Project* is included in Appendix B and a summary of the results of the laboratory testing is included below.

6.1 Unconfined Compressive Strength

The UCS testing is conducted by applying an axial load to a rock core specimen that has a length-to-diameter (L/D) ratio that ranges from 2 to 2.5. The axial load is delivered at a constant rate until the sample fails. By taking the load (kN) at failure divided by the cross-sectional area of the core specimens the uniaxial compressive strength as a stress can be determined. Additionally, each sample had longitudinal and lateral strain gauges attached to the core specimen to measure the strain produced from axial loading. The results from these strain gauges is then used to compute the elastic properties of the rock, Young's Modulus (E), and Poisson's Ratio (υ). Young's Modulus is defined as the ratio of the vertical stress to the longitudinal strain and Poisson's Ratio is used to describe the relationship between lateral strain and longitudinal strain.

The UCS testing was conducted in accordance with ASTM D7012 (*Standard Test Methods for Compressive Strength and Elastic Moduli of Intact Rock Core Specimens under Varying States of Stress and Temperatures, 2019*). The tests were completed at a constant axial load of 0.0008 mm/s.

Most of the specimens from both the Marathon and Leprechaun deposit testing failed along internal defects (i.e. S_1 foliation). A valid UCS test is defined as those tests that fail through the intact rock, however, because of the pervasive S_1 foliation at both the Marathon and Leprechaun deposits most tests failed along internal defects. Plinninger and Alber (2015) note that for anisotropic rocks the minimum compressive strengths occur when the dominant defects in the rock mass are oriented between 30 – 50^o relative to the UCS loading axis. This is the case for both the Marathon and Leprechaun deposit UCS core specimens in which the S_1 foliation is oriented on average 35^o relative to the UCS loading axis. Therefore, we interpret the UCS testing results from the geomechanical laboratory program to represent the lower UCS strength values in our subsequent analyses.

Additionally, test VL-GT-19-1 (75.71 – 75.85 m) returned a value of 301.6 MPa which we consider an outlier, so this result has not been used in our analysis in Section 7.0. The results of the UCS testing program, Elastic Moduli, and Poisson's Ratio are summarized in Table 9 and Appendix B.

Table 9 - Summary of UCS Testing and Elastic Properties

Drill Hole ID[1.]	Depth (m)	UCS (MPa)	Young's Modulus (GPa)	Poisson's Ratio	Rock Type
MA-GT-19-05	98.03	62.4	72.8	0.16	Mafic Intrusive (MFI)
MA-GT-19-05	323.35	120.1	65.0	0.09	Quartz Eye Porphyry (QEP)
MA-GT-19-06	101.64	56.2	54.7	0.12	Conglomerate (CG)
MA-GT-19-07	81.24	57.1	53.6	0.07	QEP
MA-GT-19-07	179.24	66.1	56.4	0.07	QEP
MA-GT-19-07	307.01	113.2	67.0	0.10	QEP
VL-GT-19-01	111.63	76.4	52.2	0.08	Trondhjemite (TRJ)
VL-GT-19-01	353.56	68.6	59.1	0.07	TRJ
VL-GT-19-02	233.38	41.9	46.9	0.05	CG
VL-GT-19-02	349.25	96.1	59.3	0.09	QEP
VL-GT-19-02	233.23	43.8	53.7	0.11	CG
VL-GT-19-03	54.74	78.2	41.2	0.12	CG/Dyke
VL-GT-19-03	84.54	63.4	42.5	0.11	CG
VL-GT-19-03	84.68	66.1	42.3	0.11	CG
VL-GT-19-04	75.71	301.6	83.1	0.16	TRJ
VL-GT-19-04	118.52	40.2	41.1	0.04	TRJ

Notes: 1. MA – Marathon Deposit, 2. VL – Leprechaun Deposit.

6.2 Triaxial Compressive Strength

Triaxial compressive strength (TCS) testing is conducted by encasing in a heat shrink tubing and applying the sample to a confining pressure (σ_3) while simultaneously subjecting the core specimen to an axial load (σ_1) until failure occurs. The load (kN) at failure when divided by the cross-sectional area of the core specimen results in the triaxial compressive strength (σ_1) of the rock at the applied confining pressure (σ_3). The TCS tests were completed at confining pressures of 3 or 7 MPa. The TCS testing was conducted in accordance with ASTM D7012 (*Standard Test Methods for Compressive Strength and Elastic Moduli of Intact Rock Core Specimens under Varying States of Stress and Temperatures, 2019*).

The results of the TCS testing program, Elastic Moduli, and Poisson's Ratio are summarized in Table 10 and Appendix B.

Table 10 - Summary of Triaxial Compressive Strength Testing

Drill Hole ID[1.]	Depth (m)	Confining Pressure (σ_3 - MPa)	TCS (σ_1 - MPa)	Rock Type
MA-GT-19-05	102.48	3	204.8	Mafic Intrusive (MFI)
MA-GT-19-05	227.59	3	182.5	Quartz Eye Porphyry (QEP)
MA-GT-19-06	99.75	3	56.1	Conglomerate (CG)
MA-GT-19-07	363.7	7	164.8	QEP
VL-GT-19-01	228.11	3	46.9	Trondhjemite (TRJ)
VL-GT-19-02	141.78	7	134.2	CG
VL-GT-19-02	349.61	7	198	TRJ
VL-GT-19-04	75.10	7	322.3	TRJ

Notes: 1. MA – Marathon Deposit, 2. VL – Leprechaun Deposit.

6.3 Direct Shear Testing

Direct shear testing was completed on natural discontinuities from nine core specimens. A direct shear sample is composed of two separate core specimens that fit together along a natural break or discontinuity. Direct shear testing is completed by applying a normal load (i.e. perpendicular) to the discontinuity being tested while also monitoring the shear stress (σ_t) that results in the displacement of one block relative to the other. To determine the shear strength a multi-stage testing procedure was used that applies three different normal loads (1, 2, and 3MPa) to the core specimen and monitoring the shear stress induced when movement occurs. After each normal load is applied and movement occurs the discontinuity is repositioned back to its original position. Then the relationship between the applied normal load and the shear strength can be plotted to determine the shear strength envelope. From these data points a peak and residual shear strength of the discontinuity can be determine using statistical regression analysis. All testing was completed in accordance with ASTM D5607-16 (*Standard Test Method for Performing Laboratory Direct Shear Strength Tests of Rock Specimens Under Constant Normal Force, 2019*).

The results of direct shear testing are used to estimate the shear strength of natural discontinuities within the rock mass (e.g. foliation, joints, faults, and veins). The shear strength values can be plotted against the applied average normal stress (σ_1) to determine the friction angle for Mohr-Coulomb strength criteria.

The results of the direct shear testing are summarized in Table 11 and Appendix B.

Table 11 - Summary of Direct Shear Testing

Drill Hole ID[1.]	Depth (m)	Stage	Average Normal Stress (σ_1 - MPa)	Residual Shear Stress (σ_t - MPa)	Residual Friction Angle (°)	Description of Discontinuity
MA-GT-19-06	56.39	1	0.94	0.41	23.3	Natural joint, dip = 40° (TCA), smooth waxy surface with lineation's.
		2	1.92	0.82		
		3	2.91	1.25		
MA-GT-19-06	157.59	1	0.94	0.49	25.5	Natural joint, dip angle ~ 43° (TCA), well defined slickenside lineation's.
		2	1.92	0.85		
		3	2.91	1.42		
MA-GT-19-06	76.41	1	0.47	0.35	33.8	Natural joint along foliation, dip angle ~ 40° (TCA).
		2	1.93	1.29		
		3	3.9	2.61		
MA-GT-19-06	202.12	1	0.47	0.31	34.7	Natural joint, dip angle ~ 45° (TCA), waxy coating.
		2	1.92	1.3		
		3	3.89	2.72		
VL-GT-19-02	119.40	1	0.47	0.33	36.3	Natural joint, rough, dip angle ~ 30° (TCA)
		2	1.94	1.47		
		3	3.92	2.86		
VL-GT-19-04	112.62	1	0.47	0.3	32.2	Natural joint, rough, undulating, dip angle ~ 45° (TCA)
		2	1.92	1.10		
		3	3.90	2.49		
MA-GT-19-07	77.47	1	0.94	0.69	32.6	Natural joint, dip angle ~ 58°(TCA), well defined slickenside lineation's.
		2	1.91	1.28		
		3	2.89	1.78		
VL-GT-19-01	282.26	1	0.94	0.62	32.3	Natural joint, rough surface, no infilling, waxy coating on joint.
		2	1.91	1.25		
		3	2.89	1.78		
VL-GT-19-03	54.55	1	0.94	0.49	27.8	Natural joint, undulating surface, waxy coating, no infilling, dip ~ 60° (TCA)
		2	1.91	1.06		
		3	2.88	1.48		

Notes: 1. MA – Marathon Deposit, 2. VL – Leprechaun Deposit.

6.4 Brazilian Tensile Testing

Splitting tensile strength tests, also known as Brazilian tensile testing, were conducted on eight core specimens. Brazilian tensile testing is completed by subjection a cylindrical core specimen (disc) to a diametrical load. The loading causes a tensile deformation normal to the loading direction, yielding a tensile failure. Using the resulting ultimate load and knowing the dimensions of the cylindrical core specimen allows for calculation of the indirect tensile strength of the rock. All testing was completed in accordance with ASTM D3967-16 (*Standard Test Method for Splitting Tensile Strength of Intact Rock Core Specimens, 2019*).

The results of the Brazilian tensile testing are used in conjunction with the results of both TCS and UCS testing to determine the material constant parameter m_i for the generalized Hoek-Brown criteria (Hoek et al., 2002).

The results of the Brazilian tensile testing are summarized in Table 12 and Appendix B.

Table 12 - Summary of Brazilian Tensile Testing

Drill Hole ID[1.]	Maximum Applied Load ($P_{failure}$ – kN)	Splitting Tensile Strength (σ_t – MPa)	Description
MA-GT-19-05	60.2	15.8	QEP – Diametral splitting
MA-GT-19-06	33.6	8.8	CG - Diametral splitting with some crushing at the sample ends.
MA-GT-19-07	76.5	20.1	QEP – Diametral splitting
MA-GT-19-07	42.6	11.2	QEP – Diametral splitting
VL-GT-19-01	49.8	13.0	TRJ – Diametral splitting
VL-GT-19-02	59.5	15.7	TRJ – Diametral splitting
VL-GT-19-04	68.6	18.0	TRJ – Diametral splitting
VL-GT-19-04	44.2	11.6	TRJ – Diametral splitting

Notes: 1. MA – Marathon Deposit, 2. VL – Leprechaun Deposit.

7.0 GEOTECHNICAL CHARACTERIZATION

The geotechnical model is the basics for all open pit slope design and is comprised of four sub models, these are:

- Geological Model;
- Structural Model;
- Rock mass model, and
- Hydrogeological model.

The sub-models that make up the geotechnical model and the key data included in each model is summarized below on Figure 8.

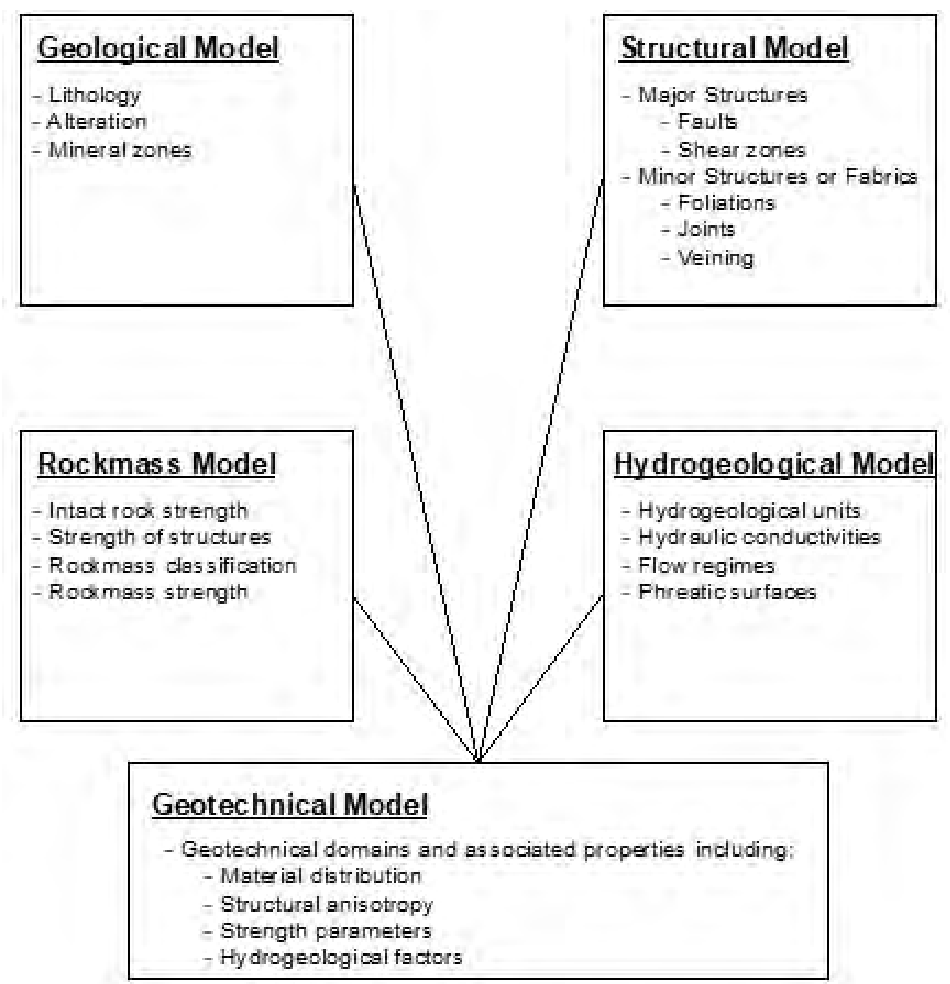


Figure 8 - Geotechnical Model, sub - models, and their key data inputs (adapted from Read and Stacey, 2009)

7.1 Geological Model

The geological model used in our analysis were provided to Terrane by Marathon and is summarized below. Figures 9 and 10 display the distribution of geological units with the open pit outlines for Marathon and Leprechaun. The geology of each deposit is summarized below.

7.1.1 Marathon Deposit

Marathon deposit gold mineralization has a strike length of approximately 1,500 m and has been intercepted at drillhole depths of up to 1,000 m. The deposit hosted in quartz porphyry monzonite (QEP) with gold concentrated in shallowly dipping (25°- 30°) quartz tourmaline porphyry (QTP) veins. To the southeast of the Marathon deposit is the Rogerson Lake Conglomerate and to the northwest limited drilling indicates that mafic intrusive rocks dominate. Figure 9 display the distribution of geological units in relation to the Marathon open pit shell (Lincoln et al., 2018).

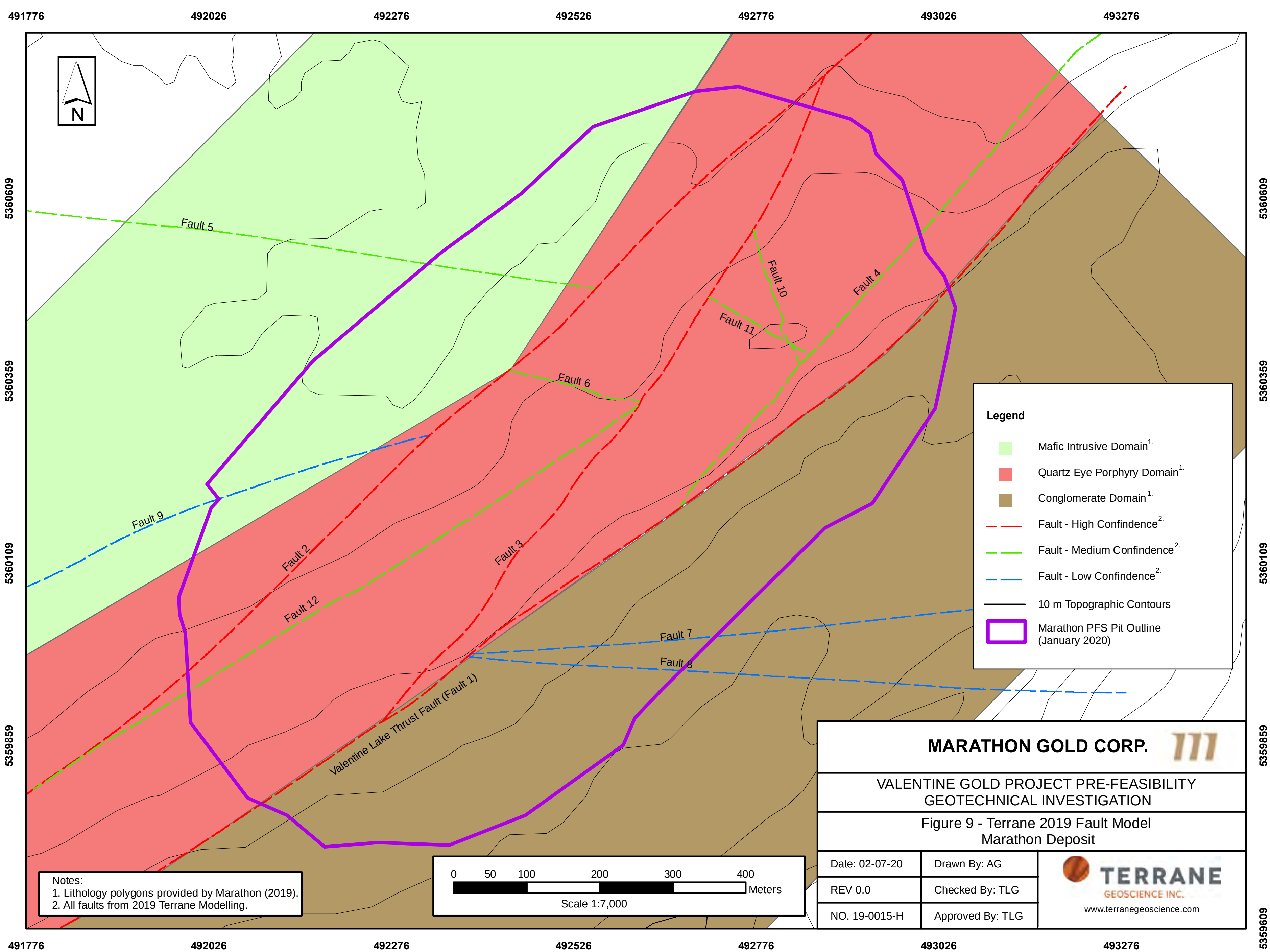
7.1.2 Leprechaun Deposit

The Leprechaun deposit is characterized by gold mineralization that has a strike length of over 950 m, width of approximately 400 m, and a vertical depth of at least 400 m. Most of the gold mineralization within the deposit is hosted by trondhjemite in a series of steeply northwest dipping QTP veins and to a lesser extent shallow southwest dipping, extensional QTP veins. To the southeast of the Leprechaun deposit is the Rogerson Lake Conglomerate which is in fault contact with the trondhjemite along the Valentine Lake Thrust Fault (Lincoln et al., 2018). Figure 10 display the distribution of geological units in relation to the Leprechaun open pit shell.

7.2 Structural Model

Tectonically, the Valentine Gold Project is dominated by east vergent thrust faulting and folding interpreted to have resulted in two main brittle-ductile deformational events (Hrabi and Siddorn, 2013). Fault modelling by Terrane, summarized herein, suggests that a late brittle strike-slip system may overprint the brittle-ductile D1 and D2 fabrics. However, further structural investigation is required to test this hypothesis.

The structural model, herein, produced by Terrane includes an analysis of major structures (e.g. faults and shear zones) and small-scale planar discontinues or fabrics (e.g. foliation, joints, veins). In the following sections the major structures and fabrics associated with each deposit are summarized.


491776
492026
492276
492526
492776
493026
493276
5360609
5360359
5360109
5359859
5359609
N
Fault 5
Fault 9
Fault 2
Fault 12
Fault 6
Fault 3
Fault 10
Fault 11
Fault 4
Fault 7
Fault 8
Valentine Lake Thrust Fault (Fault 1)
Legend
Mafic Intrusive Domain 1.
Quartz Eye Porphyry Domain 1.
Conglomerate Domain 1.
Fault - High Confindence 2.
Fault - Medium Confindence 2.
Fault - Low Confindence 2.
10 m Topographic Contours
Marathon PFS Pit Outline (January 2020)
Notes:
1. Lithology polygons provided by Marathon (2019).
2. All faults from 2019 Terrane Modelling.
0 50 100 200 300 400
Meters
Scale 1:7,000
MARATHON GOLD CORP.
VALENTINE GOLD PROJECT PRE-FEASIBILITY GEOTECHNICAL INVESTIGATION
Figure 9 - Terrane 2019 Fault Model
Marathon Deposit
Date: 02-07-20
Drawn By: AG
REV 0.0
Checked By: TLG
NO. 19-0015-H
Approved By: TLG
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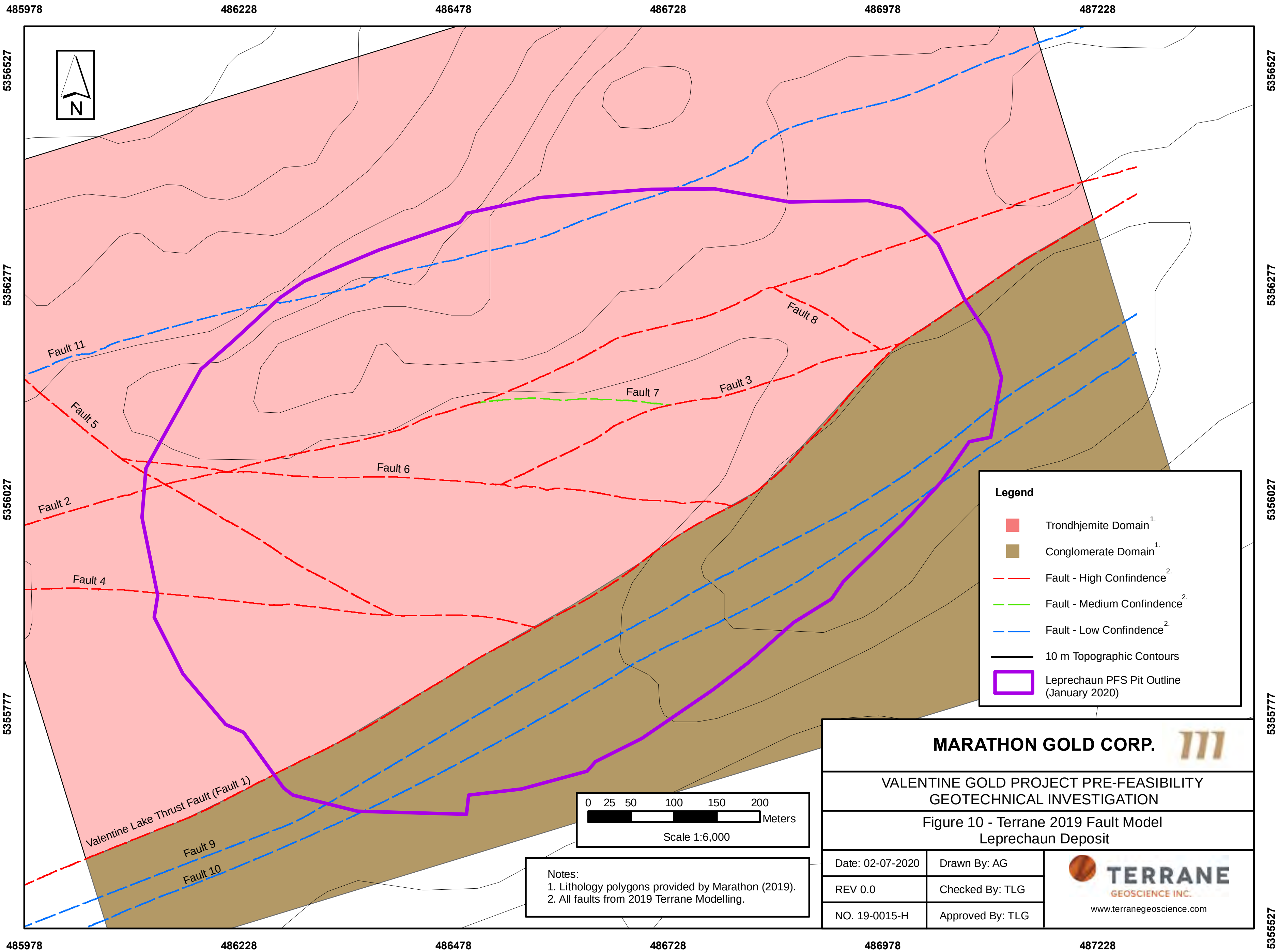
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5355777
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N
Fault 11
Fault 8
Fault 5
Fault 7
Fault 3
Fault 6
Fault 2
Fault 4
Valentine Lake Thrust Fault (Fault 1)
Fault 9
Fault 10
Legend
Trondhjemite Domain 1.
Conglomerate Domain 1.
Fault - High Confidence 2.
Fault - Medium Confindence 2.
Fault - Low Confindence 2.
10 m Topographic Contours
Leprechaun PFS Pit Outline (January 2020)
0 25 50 100 150 200
Meters
Scale 1:6,000
Notes:
1. Lithology polygons provided by Marathon (2019).
2. All faults from 2019 Terrane Modelling.
MARATHON GOLD CORP.
VALENTINE GOLD PROJECT PRE-FEASIBILITY GEOTECHNICAL INVESTIGATION
Figure 10 - Terrane 2019 Fault Model
Leprechaun Deposit
Date: 02-07-2020
Drawn By: AG
REV 0.0
Checked By: TLG
NO. 19-0015-H
Approved By: TLG
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7.2.1 Marathon Deposit – Major Structures

A 3D structural model was developed by Terrane for the Marathon deposit using exploration drill holes, geotechnical drill holes, oriented core, and optical/acoustic televiewer data. The methodology involved completing a lineament analysis to identify possible structures from the topographic and magnetic geophysical data sets which could potentially represent geologic structures. The surficial lineaments were then compared in 3D to the drill hole database to determine if their location correlated with structures logged, televiewer structures or intervals with low RQD values and/or missing core. Where correlations were identified, the attitudes measured through core orientation and/or televiewer surveying were used to extrapolate the structures across drill holes in order to create a 3D fault surface.

A total of 12 faults (Figure 11) were interpreted and modelled by Terrane for the Marathon deposit (Figure 9). Through-going first order structures (i.e. Fault 1 and Fault 2) are interpreted to represent the main fault zone boundaries. Second order faults (ie. faults 3 & 4) intersect the first-order faults on a steeply N-plunging axis of symmetry, consistent with a strike-slip system. One low-angle fault was also inferred from the data (Fault 10). Table 13 summarizes the faults modelled at the Marathon deposit and our fault confidence based on the number of coincident geologic features used to interpret each fault.

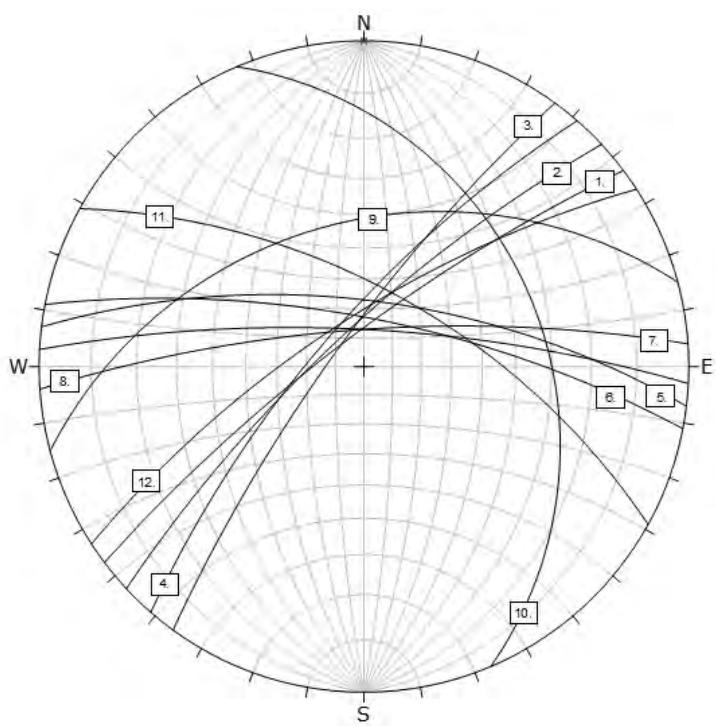

Figure 11 - Stereonet plot of modelled Marathon faults

It is important to note that, at this stage of the project, ground truthing of the modelled faults has not occurred and is considered a key recommendation.

Table 13 - Marathon Modelled Fault Summary

Fault ID	Strike(°)[1.]	Dip(°)[2.]	Topo. Lineament[3.]	Magnetic Lineament[3.]	No. DDH Logged Faults[4.]	No. DDH Intercepts RQD < 50%[5.]	Televiewer /Orientated Core[6.]	Confidence Score	Confidence[7.]
Fault 1	233	79	1	1	5	5	0	12	High
Fault 2	227	77	1	1	1	5	3	11	High
Fault 3	216	79	0	1	5	5	1	12	High
Fault 4	221	74	0	1	2	5	0	8	Medium
Fault 5	277	67	1	0	1	1	2	5	Medium
Fault 6	281	73	0	1	0	2	3	6	Medium
Fault 7	266	77	1	0	0	2	1	4	Low
Fault 8	273	77	0	0	1	1	2	4	Low
Fault 9	255	42	1	0	0	0	1	2	Low
Fault 10	337	32	0	0	0	3	2	5	Medium
Fault 11	299	63	0	1	1	2	1	5	Medium
Fault 12	237	73	0	1	0	5	2	8	Medium

Notes:
1. Strike using right-hand rule, reported strike is the mean strike from stereonet analysis of each faults modelled vertices.
2. Dip is the mean dip from stereonet analysis of each faults modelled vertices.
3. Does a topographic or magnetic geophysical lineament exist, yes (1) or no (0).
4. Number of logged structures used to model fault that are coincide with logged fault zone (>0.25 m), lost core zones, and/or conglomerate quartz eye porphyry contact (Fault 1 – Valentine Lake thrust fault). Score ranges from 0-5, score capped at 5.
5. Number of RQD runs used to model fault that are coincident with modelled fault with RQD<50%. Score ranges from 0-5, score capped at 5.
6. Number of times fault is observed in televiewer and/or oriented core. Score ranges from 0-3, score capped at 3.
7. Low (0-4), Medium (5-9), High (>10).

7.2.2 Marathon Deposit – Fabrics

As summarized in Section 5.0 discontinuity orientation data was collected from three sources; oriented core, televiewer downhole surveying, and scan line mapping. These data have been analyzed using lower hemisphere, equal-area projection stereonet plots.

Appendix C contains a summary of stereonet analysis results for the Marathon deposit orientation data. Subset stereonet plots in Appendix C include; i) oriented core data per drillhole, ii) televiewer orientation per drillhole, iii) combined oriented core and televiewer orientation per drillhole, iv) scan line mapping, and v) all data sets combined.

Analysis of the Marathon deposit orientation data from our geotechnical investigation indicates there are two main discontinuity sets present within the pit shell. Several, less penetrative discontinuity sets were noted in the data, but they are not concentrated enough to warrant classification as a discrete set. Additionally, Terrane's analysis noted slight differences in the distribution of data for each drillhole, however, these changes are not considered to be significant enough to constitute separate structural domains but rather result from orientation bias introduced by the drill hole azimuth and dip.

Table 14 and Figure 12 summarize the discontinuity sets for the Marathon deposit and their average orientation from stereonet analysis.

Table 14 - Summary of Marathon Discontinuity Sets

Discontinuity Set	Strike (°)[1.]	Dip (°)[2.]	Strike Range (°)[3.]	Dip Range (°)
S_1	234	83	223 - 268	68 - 90
JS_1	288	7	173 - 313	5 - 10

Notes:
1. Mean strike from stereonet analysis of all data sets using the right-hand rule
2. Mean strike from stereonet analysis of all data sets
3. Range selected from mean values obtained from each drillhole.

The dominant, pervasive discontinuity sets are described as follows:

> **S_1** – Approximately east-west to northeast-southwest oriented, steeply dipping regional foliation. It is interpreted that this discontinuity set is related to D1 deformation and regional east-vergent thrust faults. The majority of S1 foliation measurements dip towards the north to north-northwest, however, the steeply dipping measurements indicate a reversal in dip direction towards the south to south-southeast.
>
> **JS_1 –** The strike of JS_1 varies dramatically because of the shallow dip of the discontinuity set. The low-angle orientation of JS_1 may be related to D1 thrust faulting.

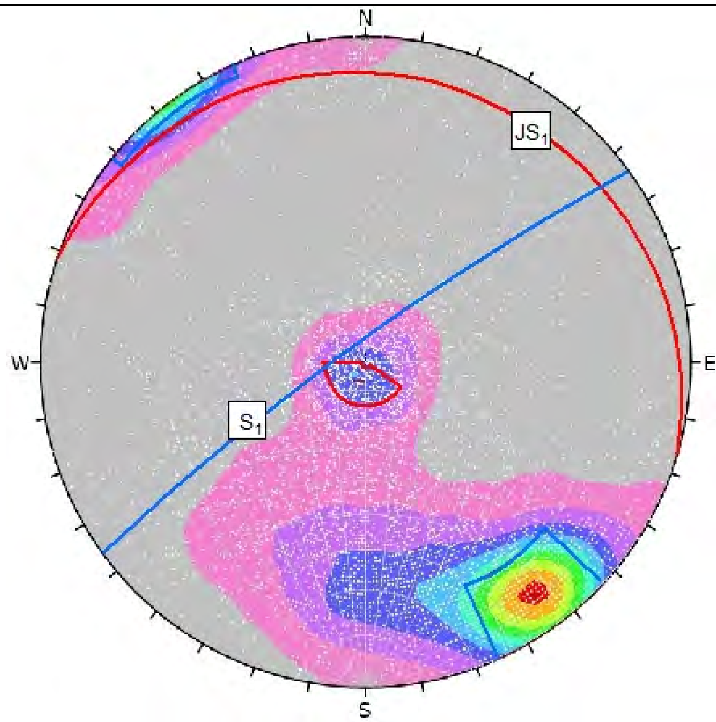

Figure 12 - All orientation data – Marathon deposit

7.2.3 Leprechaun Deposit – Major Structures

A 3D structural model was developed by Terrane for the Leprechaun deposit using exploration drill holes, geotechnical drill holes, oriented core, and optical/acoustic televiewer data. The methodology involved completing a lineament analysis to identify possible geologic structures from the topographic and magnetic data sets. Interpreted surficial lineaments were then compared in 3D to the drill hole database to determine if their location correlated with structures logged in core, televiewer interpretations, intervals with low RQD values, and/or lost core. Where correlations were identified, potential structure attitudes measured from oriented core data and/or televiewer surveying were used to extrapolate the structures across drill holes in order to create a 3D fault surface.

A total of 11 faults were interpreted and modelled for the Leprechaun deposit (Figure 10). Through-going first order structures (i.e. Fault 1, 9 and 10) are interpreted to represent major D1 reverse or oblique faults. Second order faults (ie. faults 4 & 5) intersect each other and the first-order faults on a steeply NE-plunging axis of symmetry, consistent with a strike-slip system. Table 15 summarizes the faults modelled at the Leprechaun deposit and the fault confidence based on the number of coincident geologic features used to interpret each fault. Table 15 and Figure 13 summarize the faults modelled at the Leprechaun deposit and our fault confidence based on the number of coincident geologic features used to interpret each fault.

It is important to note that at this stage of the project ground truthing of the modelled faults has not occurred. This is recommended for subsequent geotechnical investigation and will be an important step in further confirming fault confidence.

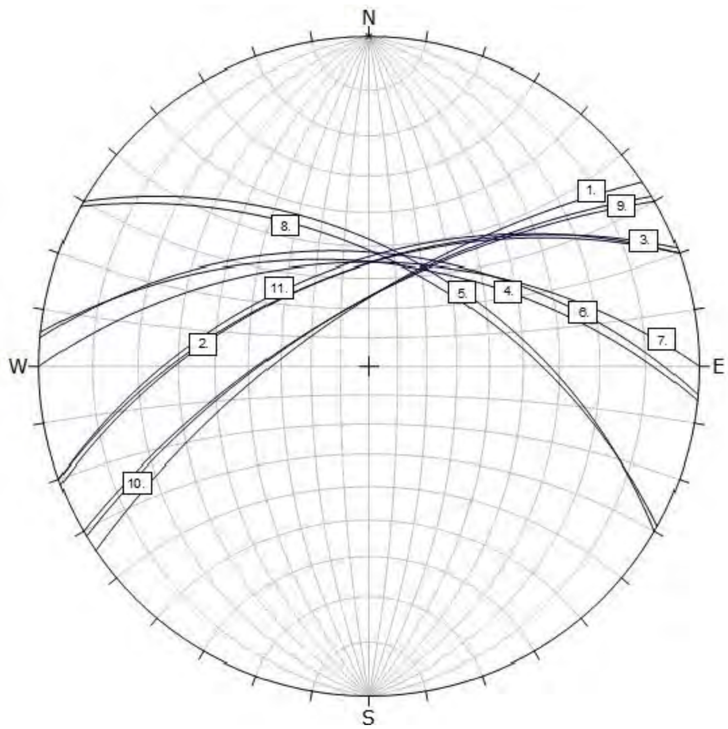

Figure 13 - Stereonet plot of modelled Leprechaun faults

Table 15 - Leprechaun Modelled Fault Summary

Fault ID	Strike(°)[1.]	Dip(°)[2.]	Weak Topo. Lineament[3.]	Strong Topo. Lineament[3.]	No. DDH Logged Faults[4.]	No. DDH Intercepts RQD < 50%[5.]	Televiewer /Orientated Core[6.]	Confidence Score	Confidence[7.]
Fault 1	236	70	1	1	5	5	0	12	High
Fault 2	250	57	1	1	5	5	3	15	High
Fault 3	249	58	1	1	3	5	0	10	High
Fault 4	276	55	1	1	5	5	0	12	High
Fault 5	300	53	1	1	5	5	0	12	High
Fault 6	275	52	1	1	3	5	0	10	High
Fault 7	270	55	1	0	2	5	0	8	Medium
Fault 8	299	56	1	0	1	5	3	10	High
Fault 9	240	69	1	1	0	0	1	3	Low
Fault 10	239	69	1	0	0	1	1	3	Low
Fault 11	250	55	1	1	0	2	0	4	Low

Notes:
1. Strike using right-hand rule, reported strike is the mean strike from stereonet analysis of each faults modelled vertices.
2. Dip is the mean dip from stereonet analysis of each faults modelled vertices.
3. Does a topographic lineament exist, if so, is it weak or very well defined, strong, yes (1) or no (0).
4. Number of logged structures used to model fault that are coincide with logged fault zone (>0.25 m), lost core zones, and/or conglomerate quartz eye porphyry contact (Fault 1 – Valentine Lake thrust fault). Score ranges from 0-5, score capped at 5.
5. Number of RQD runs used to model fault that are coincident with modelled fault with RQD<50%. Score ranges from 0-5, score capped at 5.
6. Number of times fault is observed in televiewer and/or oriented core. Score ranges from 0-3, score capped at 3.
7. Low (0-4), Medium (5-9), High (>10).

7.2.4 Leprechaun Deposit – Fabrics

As summarized in Section 5.0 discontinuity orientation data was collected from three sources; oriented core, televiewer downhole surveying, and scan line mapping. These data have been analyzed using lower hemisphere, equal-area projection stereonet plots.

Appendix C contains a summary of the stereonet analysis for the Leprechaun deposit orientation data. Subsets plots of the data including; i) oriented core data per drillhole, ii) televiewer orientation per drillhole, iii) combined both oriented core and televiewer orientation per drillhole, and iv) scan line mapping are presented.

Analysis of the Leprechaun deposit orientation data indicates there are two main discontinuity sets (for the purposes of geotechnical investigation) present within the pit shell. Several, less penetrative discontinuity sets were noted in the data; however, they are not concentrated enough to select a discrete set. Results also show slight differences in the distribution of orientation data for each drillhole; however, these changes are not considered to be significant enough to constitute separate structural domains.

Table 16 and Figure 14 summarize the discontinuity sets for the Leprechaun deposit and their average orientation.

Table 16 - Summary of Leprechaun Discontinuity Sets

Discontinuity Set	Strike (°)[1.]	Dip (°)[2.]	Strike Range (°)[3.]	Dip Range (°)
S_1	250	56	234 - 266	49 - 72
JS_1	303	24	312 - 340	19 - 21

Notes: 1. Mean strike from stereonet analysis of all data sets using the right-hand rule

2. Mean strike from stereonet analysis of all data sets

3. Range selected from mean values obtained from each drillhole.

The dominant, pervasive joint sets are described as follows:

S_1 – Approximately east-west to northeast-southwest oriented, moderately to steeply dipping regional foliation. It is interpreted that this discontinuity set is related to D1 deformation and regional east-vergent thrust faults. The majority of S_1 foliation measurements dip towards the north to north-northwest, however, steeply dipping measurements indicate a reversal in dip direction towards the south to south-southeast, locally.

JS_1 – The strike of JS_1 strikes approximately north northwest to northwest (right hand rule). The mean strike and dip (RHR) of JS_1 in the Leprechaun Pit is 317/21. However, the concentration of JS_1 is less conclusive than the Marathon data.

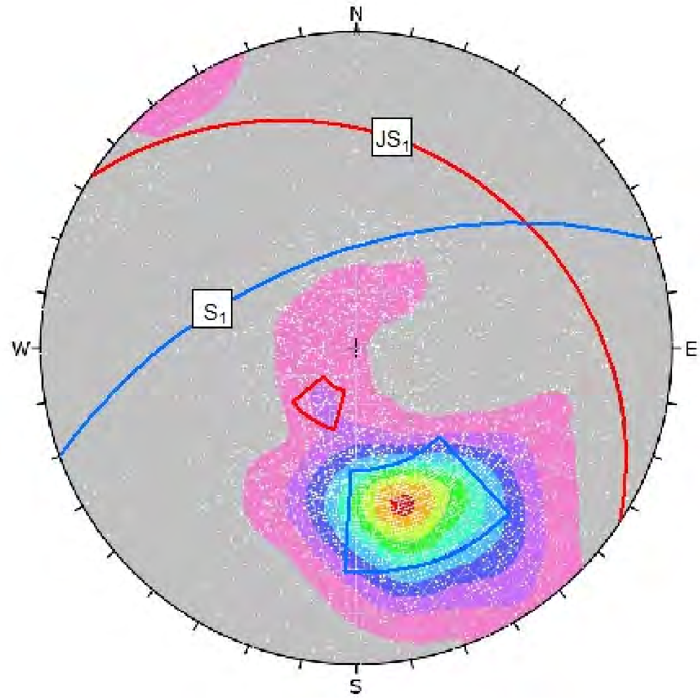

Figure 14 - All Orientation data - Leprechaun Deposit

7.3 Rock Mass Model

The nature of the rock near the proposed open pits has been characterized through a combination of oriented geotechnical drilling, scan line mapping, structural assessment, laboratory testing and rock mass classification. The rocks were classified using the RMR_{76} method (Bieniawski, 1976).

The primary geotechnical parameters for RMR_{76} were logged for each core run, thereby creating a profile of the rock mass classification with depth in each geotechnical drill hole.

Bieniawski (1976) created a rock mass classification system called the Geomechanics Classification or the Rock Mass Rating (RMR_{76}). Over time this system has been successively refined as more case histories have become available (Hoek, 2000). The following five parameters are used to classify a rock mass using the RMR system:

1. Uniaxial compressive strength (UCS) of the rock.
2. RQD – Rock Quality Designation.
3. Spacing of joints.
4. Condition of joints.
5. Groundwater conditions.

Table 17 summarizes rock mass quality using the RMR_{76} system.

Table 17 - RMR_{76} Classification

Rock Mass Quality	RMR_{76} Range
Very Good Rock	100-81
Good Rock	80-61
Fair Rock	60-41
Poor Rock	40-21
Very Poor Rock	< 20

The following section summarizes the rock mass rating, intact rock strength and generalized Hoek-Brown criteria for each geotechnical domain in each deposit.

Geotechnical domains are composed of areas or domains of an open pit rock mass that exhibit similar material properties such as intact rock strength, structural domain, rock mass classification, and rock mass strength characteristics.

7.3.1 Geotechnical Domains – Marathon Deposit

The Marathon deposit is underlain by three primary rock types; mafic intrusive, quartz eye porphyry, and conglomerate. An analysis of the RMR_{76} data for each rock type indicates that the rock mass at the Marathon deposit displays a normal distribution and ranges from 55 – 77 with a mean value of 67. This mean corresponds to rock mass quality of Good (Table 15).

Table 18 displays the average rock mass rating for each lithologic unit along with the minimum and maximum value assigned based on the average plus or minus one standard deviation. A summary of the rock mass characterization for the Marathon deposit can be found in Appendix D.

Table 18 - Summary of Rock Mass Classification for Each Rock Type - Marathon Deposit

Lithology	Mean RMR_{76}	Minimum RMR_{76} [1.]	Maximum RMR_{76} [1.]
Mafic Intrusive	66	55	77
Quartz Eye Porphyry	68	59	76
Conglomerate	67	59	74

Notes: 1. Minimum and maximum values based on mean +/-

A review of the RMR_{76} values, RQD (%), and fracture frequency vs. depth indicates that the rock mass is generally consistent with depth. The upper section (approximately 40 m depth from surface) displays slightly reduced rock mass quality and increased fracture frequency (Appendix D).

As discussed in Section 6, the minimum compressive strengths for anisotropic rocks from UCS testing occur when the dominant defects in the rock mass are oriented between 30 – 50^{o} relative to the UCS loading axis. This was determined to be the case for both the Marathon and Leprechaun deposits. Therefore, we have interpreted the UCS testing results from the laboratory program to represent the lower UCS strength values for each lithology. Furthermore, the average results obtained from rebound hammer index testing more accurately reflected the anticipated rock strengths for each lithology. As a result, the rebound hammer values have been used as the anticipated mean strength for each rock type. All rock types returned values for intact rock strength in the Strong to Very Strong range (Brown, 1981). A summary of the intact rock strength for each of the primary lithologies is included in Table 19.

Table 19 - Summary of Intact Rock Strength for Each Rock Type - Marathon Deposit

Lithology	Mean Rock Strength (MPa)[1.]	Classification[2.]	Minimum Rock Strength (MPa)	Classification[2.]
Mafic Intrusive	120	R5 – Very Strong	62	R4 - Strong
Quartz Eye Porphyry	145	R5 – Very Strong	89	R4 - Strong
Conglomerate	113	R5 – Very Strong	54	R4 - Strong

Notes: 1. Mean value from rebound hammer testing,

2. After Brown (1981),

Field logging (Appendix A) estimates of the intact rock strength (Brown, 1981) generally agreed with the rebound hammer results. The average for all three rock types was estimated as R5 – Very Strong. Based on Terrane's analysis, the Marathon deposit has been subdivided into three geotechnical domains correspond to the primary lithological units encountered within the deposit.

7.3.2 Geotechnical Domains – Leprechaun Deposit

Bedrock in the Leprechaun deposit area primarily consists of trondhjemite and conglomerate lithologies. An analysis of the RMR_{76} data for each rock type indicates that the rock mass at the Leprechaun deposit displays a normal distribution and ranges from 60 – 82 with a mean value of 71. This mean corresponds to rock mass quality of Good (Table 15).

Table 20 has the average rock mass rating for each lithologic unit along with a minimum and maximum value assigned based on the average plus or minus one standard deviation. A summary of the rock mass characterization for the Leprechaun deposit is included in Appendix D.

Table 20 - Summary of Rock Mass Classification for Each Rock Type - Leprechaun Deposit

Lithology	Mean RMR_{76}	Minimum RMR_{76}[1.]	Maximum RMR_{76}[1.]
Trondhjemite	71	60	82
Conglomerate	71	60	83

Notes: 1. Minimum and maximum values based on mean +/-

A review of the RMR_{76} values, RQD (%), and fracture frequency results vs. depth indicates that the rock mass is generally consistent with depth. The upper section (vertically to approximately 50 m below surface) displays slightly reduced rock mass quality and increased fracture frequency (Appendix D).

As discussed in Section 6, the UCS testing results from the laboratory program are interpreted to represent the lower UCS strength values for each lithology. Furthermore, the average results obtained from rebound hammer index testing more accurately reflect the anticipated rock strengths for each lithology. As a result, the rebound hammer values have been used as our anticipated mean strength for each rock type. All rock types returned values for intact rock strength in the Strong to Very Strong range (Brown, 1981).

A summary of the intact rock strength for each of the primary lithologies is included in Table 21.

Table 21 - Summary of Intact Rock Strength for Each Rock Type - Leprechaun Deposit

Lithology	Mean Rock Strength (MPa)[1.]	Classification[2.]	Minimum Rock Strength (MPa)	Classification[2.]
Trondhjemite	112	R5 – Very Strong	72	R4 - Strong
Conglomerate	126	R5 – Very Strong	54	R4 - Strong

Notes: 1. Mean value from rebound hammer testing

2. After Brown (1981)

Field logging (Appendix A) estimates of the intact rock strength (Brown, 1981) generally agreed with the rebound hammer results. The average for all three rock types is estimated as R5 – Very Strong.

Based on Terrane's analyses, the Leprechaun deposit has been subdivided into two geotechnical domains that correspond to the primary lithological units encountered within the deposit.

7.4 Hydrogeology Model

The hydraulic conductivity for each test interval was determined based on the analysis of the packer test data using the software AquiferTest® Version 8 (Waterloo Hydrogeologic, Waterloo, ON). Results for each test interval and details of the hydrogeology investigation are presented in Appendix E.

The calculated hydraulic conductivity values for each test interval in the two pit areas are provided in Table 8, and a plot of hydraulic conductivity versus depth separated by rock type for each pit is presented in Figure 15.

For the Leprechaun pit, where all tests were completed within the trondhjemite bedrock unit, the hydraulic conductivity values ranged through four orders of magnitude from 5.79 $x10^{-10}$ to 1.69 $x10^{-6}$ m/s, with a geometric mean of 5.88 x 10^{-8} m/s.

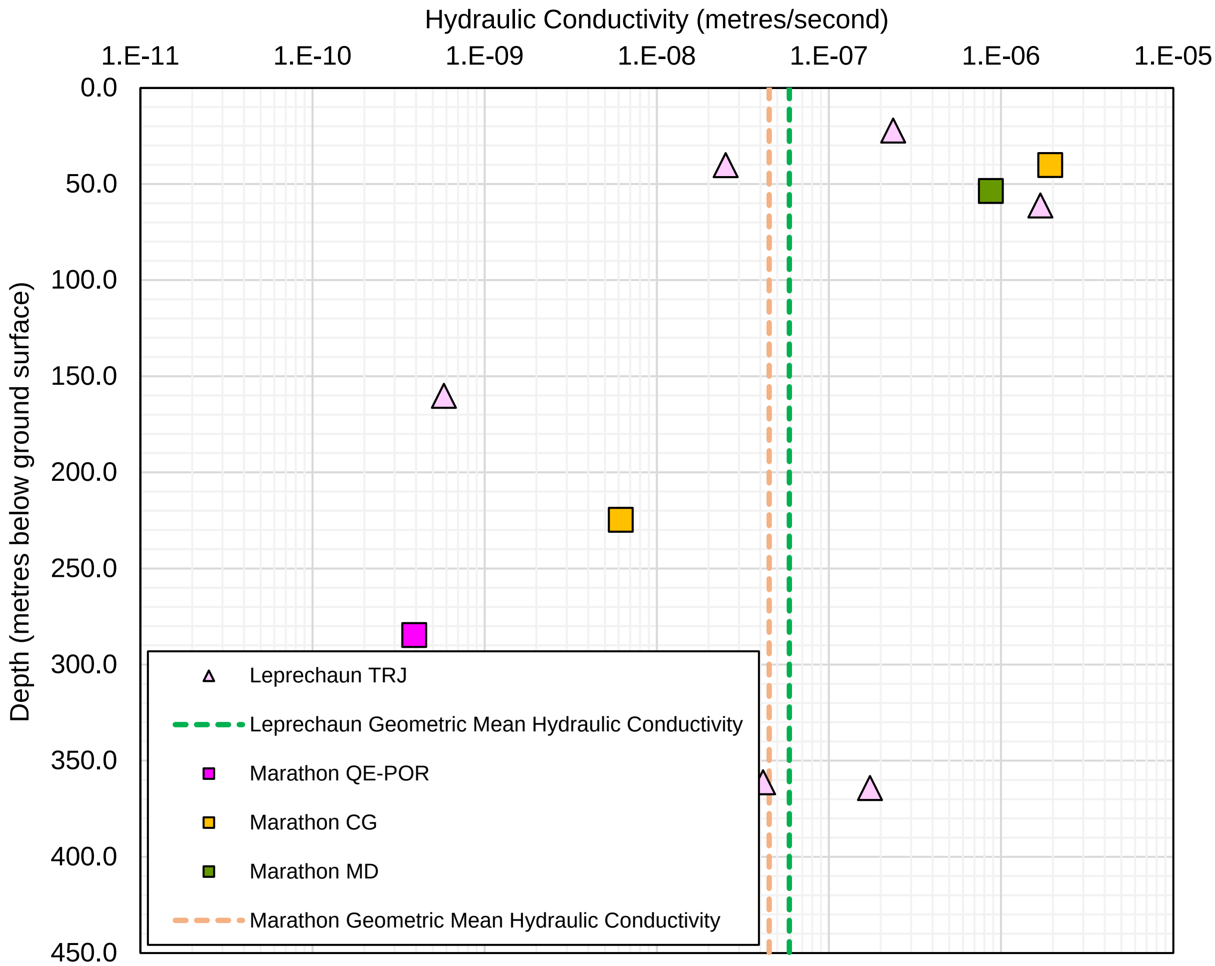

Figure 15 - Summary of Marathon and Leprechaun Hydraulic Conductivity with Depth

The highest hydraulic conductivity value (1.69 x 10-6 m/s) was measured in packer test PT2 in drillhole VL-GT-19-03, which included a faulted interval from a downhole depth of 44 to 50 m (possibly representing the Valentine Lake Fault). The relatively high hydraulic conductivity measured in this test interval is attributed to increased permeability due to fracturing associated with this structure. There was no apparent variation related to depth evident in the Leprechaun pit test results.

For the Marathon pit, a similar broad range in hydraulic conductivity values through four orders of magnitude was measured from 3.9 $x10^{-10}$ to 6.18 x 10^{-6} m/s. The variability for these results may be a function of bedrock type, as well as structure. A single hydraulic conductivity value was determined for the mafic dyke (8.73 x 10^{-7} m/s) and the quartz-eye porphyry (3.9 $x10^{-10}$ m/s). Two tests were completed in the conglomerate, both within drillhole MA-GT-19-6, including a shallow test at downhole depth 32 m, and a deeper test at downhole depth 180 m. The shallow test returned a hydraulic conductivity of 1.93 x 10^{-6} m/s, and the deeper test returned a value of

$6.18x10^{-9}$ m/s. All four test results for the Marathon pit show an apparent linear trend in decreasing hydraulic conductivity with depth, with an approximately one order of magnitude decrease in hydraulic conductivity for each 100 m increase in depth. While it is expected that hydraulic conductivity will decrease with depth as fracture apertures collapse due to lithostatic load; the sparsity of data and variable rock types does not allow this relationship to be clearly demonstrated in the Marathon pit as part of this program. In general, the hydraulic conductivity values determined during this program were within the typical range of values in the literature for similar rock types (Freeze & Cherry, 1979).

At the pre-feasibility stage the level of data collection is not enough (particularly for Marathon) to determine separate hydraulic conductivity estimates based on bedrock type and depth. Because of this, the geometric mean of the packer test results for each pit were used to represent the average hydraulic conductivity of the bulk rock mass for estimation of groundwater inflow rates.

The geometric mean hydraulic conductivity values used for estimation of groundwater pit inflow rates for the Project pits are provided in Table 8 (5.88×10^{-8} m/s for the Leprechaun pit, and 4.49 x 10-8 m/s for the Marathon pit). These average hydraulic conductivity values are similar to the average hydraulic conductivity estimates determined for the bulk rock mass of the two pits provided by Stantec (3.4×10^{-8} m/s for the Leprechaun pit, and 7.8×10^{-8} m/s for the Marathon pit) (Stantec, 2017b), and are considered to be reasonable for the purposes of preliminary estimates of pit inflows presented here. It should be noted however that various faults, fractures, and shear zones were identified in the two pits as part of the geotechnical program that weren't tested and are not yet well characterized. Such structures may have substantially higher localized permeability than the surrounding rock mass that could lead to higher pit inflows.

The full results and calculations associated with the hydrogeology analysis are included in Appendix E.

7.4.1 Estimation of Pit Inflows

Combining the estimated inflow from direct precipitation of 2,236 m^3/day (Appendix E) for the Leprechaun pit, and 2,839 m^3/day for the Marathon Pit, to the groundwater inflow estimates, a total average daily inflow rate of 4,568 m^3/day (3,172 L/min) is calculated for the final configuration of the Leprechaun pit, and 5,454 m^3/day (3,788 L/min) is calculated for the final pit configuration of the Marathon pit. These estimated of total daily inflow rates are considered manageable with conventional dewatering equipment.

Table 22 - Total Pit Inflow Estimates

Pit	Pit Area (m^2)	Direct Precipitation Inflow (m^3/day)	Groundwater Inflow (m^3/day)	Total Pit Inflow (m^3/day)
Leprechaun	6.6×10^5	2,236	2,331 (Zone 1 – 992; Zone 2 – 1,339)	4,568
Marathon	8.4×10^5	2,839	2,615 (Zone 1 – 1,139; Zone 2 – 1,476)	5,454

The total pit inflow estimates presented above suggest that inflows from direct precipitation and groundwater will provide near equivalent contributions to the total inflow amounts estimated for each pit. Further, a slightly higher total inflow rate is estimated for the Marathon pit. This higher estimated inflow rate for the Marathon pit is solely attributed to its larger geometry, since its bulk rock mass hydraulic conductivity was slightly less than that determined for the Leprechaun pit.

The calculated inflow estimates presented above represent long-term, average rates under a steady-state, full pit development scenario. It is expected that initial inflow rates into the pits will be higher than that estimated as the rock mass dewaters under higher horizontal hydraulic gradients, and overtime as the pit is developed the flow rates will relax and end–up at the steady-state inflow rates presented here. Additionally, this phase of the hydrogeology investigation did not focus on any potential inflow from nearby lakes. This should be a part of subsequent hydrogeology investigations, in particular, if there are any connections to adjacent lakes via major structures should be evaluated. Appendix E contains a full hydrogeologic analysis report with key assumptions made in the analyses.

7.4.2 Groundwater and Open Pit Slopes

Groundwater pressure (i.e. porewater pressure) is an important part of open pit slope stability. Groundwater pressures act as buoyant forces directly opposing stabilizing forces in a pit slope. As a result, porewater pressure must be considered for any slope stability modelling.

Blasting at the bench scale, generally, results in a relatively free draining slope from damage/disruption to the rock mass during development. This in turn results in a groundwater surface that is sufficiently deep within the slope so that porewater pressures and their effect on stability of the slope, at the bench scale, is limited. As a result, porewater pressures are not usually included in bench scale design.

However, since major structures, such as faults, shear zones, and/or clay filled discontinuities are common within a rock mass they can act to isolate groundwater flow to these structures. This may result in elevated porewater pressure on major structures which can affect the stability at the inter-ramp to overall pit scale. This scenario was found to be the case in VL-GT-19-03 where packer testing of the fault zone (possibly the Valentine Lake fault) yielded a relatively high hydraulic conductivity relative to other tests.

Given the relatively low hydraulic conductivities described above (except for VL-GT-19-03), we have assumed that depressurization of the slope will occur naturally (i.e. without horizontal drains) as a result of blasting induced fracturing of the rock mass. Based on experience it is assumed that blasting will result in depressurization of the slope to between 20 – 60 m behind the face. Further, this assumption is supported by Hoek and Diederichs (2006), who concluded that production blast damage can extend up to 100 m behind the slope face.

7.5 Design Sectors

Since pit slope angles within an open pit are influenced by pit wall orientation relative to discontinuities, major structures, rock mass strength, and geotechnical domains it is necessary to divide the open pit into design sectors. Design sectors represent areas of the open pit with similar wall orientations and geotechnical characteristics. The design sectors for the proposed Marathon and Leprechaun open pits are included on Figure 16 and 17 respectively. Additionally, the mean slope orientation for each design sector is included on the Figures 16 and 17.

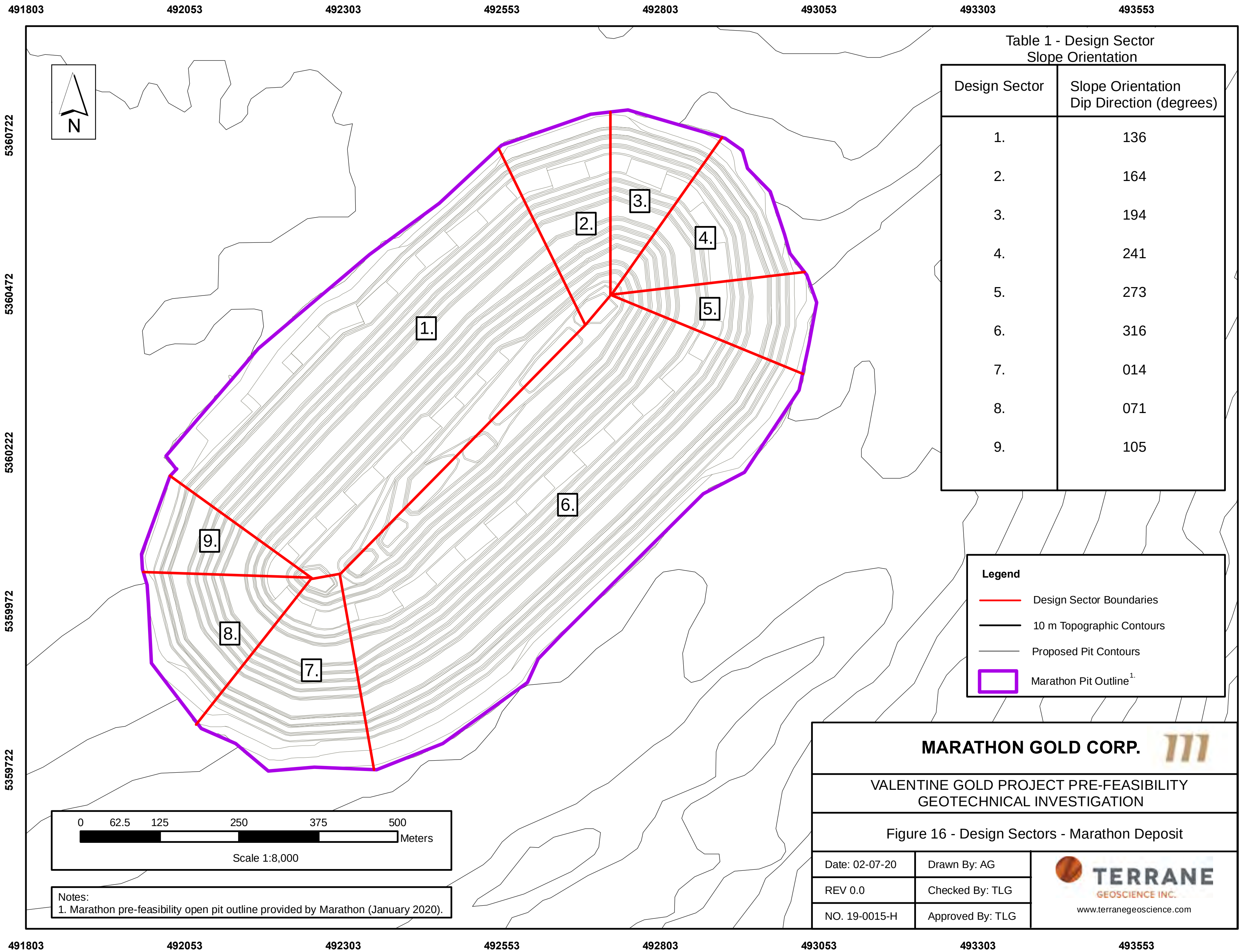

Table 1 - Design Sector Slope Orientation

Design Sector	Slope Orientation Dip Direction (degrees)
1.	136
2.	164
3.	194
4.	241
5.	273
6.	316
7.	014
8.	071
9.	105

Notes:
1. Marathon pre-feasibility open pit outline provided by Marathon (January 2020).

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Figure 16 - Design Sectors - Marathon Deposit

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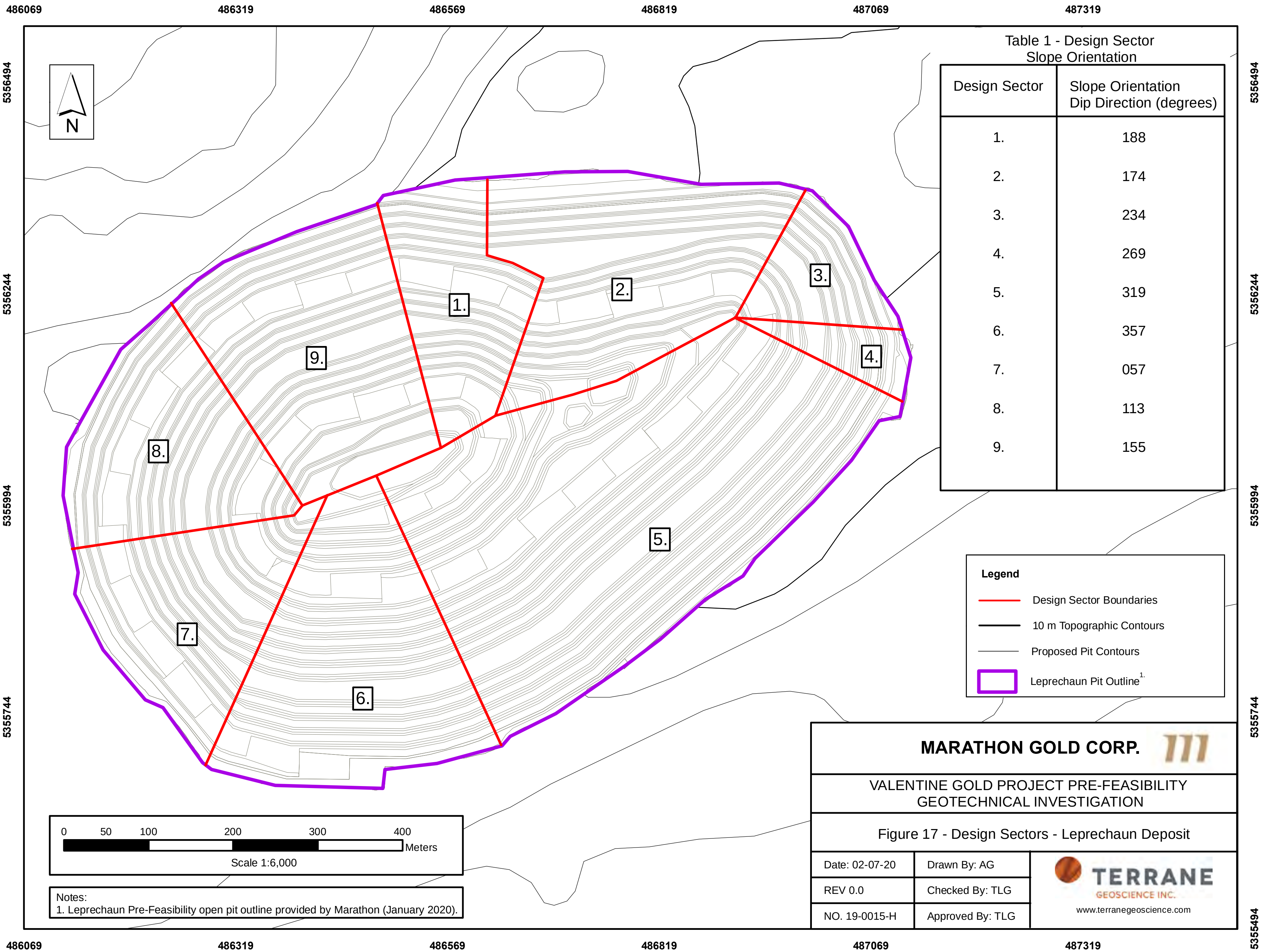

Table 1 - Design Sector Slope Orientation

Design Sector	Slope Orientation Dip Direction (degrees)
1.	188
2.	174
3.	234
4.	269
5.	319
6.	357
7.	057
8.	113
9.	155

Legend

- Design Sector Boundaries
- 10 m Topographic Contours
- Proposed Pit Contours
- Leprechaun Pit Outline[1]

Notes:
1. Leprechaun Pre-Feasibility open pit outline provided by Marathon (January 2020).

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Figure 17 - Design Sectors - Leprechaun Deposit

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8.0 OPEN PIT STABILITY ANALYSIS

Slope design takes into consideration an analysis of the overall slope stability of a pit wall (i.e. all the benches and ramps from the pit floor to the surface) and the bench design (i.e. bench width, bench face angle, and bench height). The overall slope angle, inter-ramp angle, and the bench face angles are then designed based on an acceptable criteria for probability of failure (PoF).

8.1 Acceptance Criteria

Herein we use the acceptance criterion for this design as defined by Read and Stacey (2009) who recommend design values for FOS and PoF based on the criteria summarized in Table 23. Using the probability of failure equal to the percentage of discontinuities that fall within the critical zone for kinematic instability (i.e. PoF=P[FOS<1]) the maximum bench face angles are recommended for each design sector according to the acceptable PoF (Table 8). The minimum required FOS and PoF used for our design are highlighted below (bold).

Table 23 - Acceptance Criteria, FOS, PoF and Category of Slope (Adapted, Read and Stacey, 2009)

Slope Scale	Consequences of Failure	Acceptance Criteria	
		FOS (min) (static)	PoF (max) PoF[FOS<1]
Bench	**Low - High**	**1.1**	**<25%**
Inter-ramp	Low	1.15 - 1.2	25%
	Medium	**1.2**	**20%**
	High	1.2 – 1.3	10%
Overall	Low	1.2 – 1.3	15-20%
	Medium	1.3	5-10%
	High	**1.3**	**≤5%**

8.2 Overburden Slopes

A detailed soils geotechnical program was not part of the scope of work for this project. However, Terrane has reviewed test pit logging results from the site wide hydrogeological program (Gemtec, 2019) and the overburden geological solid provided by Marathon.

Within the footprint of the Marathon deposit, overburden thicknesses generally range from 0 to 13 m with an average depth of approximately 6 m. Test pit results near the Marathon deposit show the overburden to dominantly consist of silty sand and gravel (SM); this is interpreted to be glacial Till. Tills are commonly overlain by a thin organic layer.

At the Leprechaun deposit, overburden thicknesses range from 0 to 8 m with an average depth of approximately 5 m. Test pit results near the Leprechaun deposit describe the overburden to dominantly consist of silty sand and gravel (SM) and lesser clayey sand with gravel (SC); these are interpreted to be glacial till. Tills are commonly overlain by a thin organic layer.

A full assessment of the overburden slopes was outside our scope of work; however, we have assumed overburden slopes in till to be constructed at angles that do not exceed a gradient of 2H:1V. Additionally, where overburden thickness exceeds 4 m a bench of 2.0 m should be included. Further overburden data collection is required to confirm our assumptions.

8.3 Bench Face – Kinematic Analysis

At the bench scale, local, rock structure or combinations of structures are the primary failure mechanisms affecting stability. These failures involve the movement of intact rock masses along one or more discontinuity sets. These failures are commonly broken into three categories or three kinematic failure modes; 1.) planar sliding, 2.) toppling, and 3.) wedge sliding failure. Stability of an individual rock slope for each of the above failure types is a function of discontinuity shear strength, discontinuity orientation relative to the slope orientation/dip, and groundwater conditions.

Bench face angles could be designed to avoid all possible failures, but this would result in uneconomically flat slope angles. The design herein assumes that some intermittent failures will occur and can be contained by the catch benches. These kinematic failures are not anticipated to involve failure of the entire bench but rather small blocks on the bench face.

Below is a summary of each of the three kinematic failure modes:

> Planar Sliding – A plane failure may occur when a discontinuity daylight out of a rock slope at an angle shallower than the angle of the slope but steeper than the friction of the discontinuity. Planar failures, generally, only occur when the strike of the discontinuity is sub-parallel to the strike of the rock slope (+/-20^{o}).
>
> Wedge Instabilities – A wedge failure may occur when two or more discontinuities combine to form a wedge dipping out of a rock slope. For wedge instabilities to occur the line of intersection of the two planes must dip out of the rock slope at an angle shallower than

the angle of the slope but steeper than the friction of the discontinuity. Wedge failures, generally, only occur when the azimuth of the line of intersection is within +/-45° of the dip direction of the slope.

Toppling Instabilities – Toppling failures may form when a rock mass contains numerous, near parallel to parallel, steeply dipping (away from the rock slope), persistent discontinuities. Toppling failures, generally, only occur when the strike of the discontinuities are sub-parallel to the strike of the rock slope (+/-20°).

For the Leprechaun and Marathon kinematic analysis, Terrane used a friction angle (ϕ) of 32 and no cohesion (i.e. c=0 MPa). As it was determined that only one structural domain exists for each of the Leprechaun and Marathon open pits, our kinematic analysis for each open pit was carried out by using all the discontinuities logged from the pit areas. Using the full suite of discontinuities for each open pit is interpreted to constitute a statistically representative population of discontinuities.

For each of the three kinematic failure modes an analysis was completed at varying bench face angles until the design criteria PoF$\leq$25% was reached. The kinematic failure mode that reached a PoF$\leq$25% is considered the controlling kinematic failure mode. If the PoF>25% was reached before a bench face angle of 75° was returned further limit-equilibrium analysis was completed in Rocscience SWedge, RocPlane, or RocTopple to further evaluate the effect of the dominant kinematic failure mode. While kinematic and limit-equilibrium analysis suggests that in some design sectors bench face angles steeper than 75° are possible, we do not recommend this. Operational experience at mine sites indicates that if the bench face angle is too steep and systematic discontinuities are present in the rock mass bench faces do not perform well. However, there are examples of mines with bench face angles steeper than 75°.

The kinematic analysis included herein have been performed for the proposed open pit shells as provided to Terrane by Marathon (2019).

8.3.1 Bench Face Kinematic Analysis – Marathon

As shown on Figure 16 the Marathon deposit has been divided into 9 design sectors. We have completed kinematic analysis on each design sector for each of the kinematic failure modes: 1.) planar sliding, 2.) toppling, and 3.) wedge sliding failure.

A full summary of our kinematic analysis in included in Appendix F and the results are summarized below in Table 24.

Table 24 - Summary of Kinematic Analysis Results - Marathon

Design Sector	Slope Orientation[1.] (°)	Controlling Kinematic Failure Mode	Probability of Failure (%)	Bench Face Angle (°)	Bench Width[3.] (m)	Bench Height (m)[4.]
1	136	Toppling	22.6	71	8.1	18
2	164	Toppling	24.8	71	8.1	18
3	194	Toppling	13.4	71	8.1	18
4	241	Toppling	16.3	71	8.1	18
5	273	Wedge	10.8	71	8.1	18
6	316	Wedge	14.4	71	8.1	18
7	014	Wedge	20.2[2.]	75	8.1	18
8	071	Wedge	23.9	75	8.1	18
9	105	Wedge	15.6	75	8.1	18

Notes:

1. Slope orientation reported as dip direction of the slope face.
2. Probability of failure from limit equilibrium analysis. Where kinematic analysis returned a PoF>25% for a bench face angle<75°, limit-equilibrium analysis was completed for that design sector.
3. Bench width determine using Ryan and Pryor (2000) where: Bench width (m) = 0.2 x bench height +4.5 m.
4. Assume triple, 6 m benches or 18 m total (Marathon, 2019)

Figure 18 displays a schematic representation of the proposed Marathon open pit face angles. The proposed open pit at Marathon can be divided into three areas of similar bench geometry. A northeast area composed of design sectors 1,2,3, a southeast area composed of design sectors 4,5,6 and a southwest area composed of design sectors 7,8,9.

Kinematic analysis suggests that bench face angles of 75° throughout the Marathon pit are possible, however, the final bench geometry is constrained by the overall slope angle. That is, if a bench face angle of 75° was recommended in each design sector the overall slope angle would exceed our maximum recommend gradient. As a result, the bench face angles summarized in Table 24 in combination with geotechnical berms at intervals of 90 m limit the overall slope angle to our recommended maximum. If using 18 m high benches (triple 6 m benches) a bench width of 8.1 m is recommended (Ryan and Pryor, 2000).

It should be noted that in the southeast and southwest area the S_1 foliation is interpreted to be the controlling kinematic structure. Our geotechnical investigation indicates that S_1 has an average dip of 83° and ranges from 68 – 90°. Locally, a bench face angle of 71 - 75° may not be achievable due to the S_1 foliation daylighting out of the slope. Additionally, when using triple benching it is common that an offset between each 6 m bench of up to 1 m occurs due to constraints from drilling equipment. This can result in shallowing of the bench face angle (and ultimately the entire open pit).

8.3.2 Bench Face Kinematic Analysis - Leprechaun

As shown on Figure 17 the Leprechaun deposit has been divided into 9 design sectors. We have completed kinematic analysis on each design sector for each of the kinematic failure modes: 1.) planar sliding, 2.) toppling, and 3.) wedge sliding failure.

A full summary of our kinematic analysis is included in Appendix F and the results are summarized below in Table 25.

Table 25 - Summary of Kinematic Analysis Results - Leprechaun

Design Sector	Slope Orientation[1.] (°)	Controlling Kinematic Failure Mode	Probability of Failure (%)	Bench Face Angle (°)	Bench Width[3.] (m)	Bench Height (m)[4.]
1	188	Toppling	17.7	75	8.1	18
2	174	Toppling	24.9	75	8.1	18
3	234	Toppling	17.9	75	8.1	18
4	269	Wedge	19.2	75	8.1	18
5	319	Wedge	19.1[2.]	65	8.1	18
6	357	Wedge	24.7[2.]	65	8.1	18
7	057	Wedge	24.0	75	8.1	18
8	113	Toppling	11.5	75	8.1	18
9	155	Toppling	10.3[2.]	75	8.1	18

Notes:

1. Slope orientation reported as dip direction of the slope face.
2. Probability of failure from limit equilibrium analysis. Where kinematic analysis returned a PoF>25% for a bench face angle<75°, limit-equilibrium analysis was completed for that design sector.
3. Bench width determine using Ryan and Pryor (2000) where: Bench width (m) = 0.2 x bench height +4.5 m.
4. Assume triple, 6 m benches or 18 m total (Marathon, 2019)

Figure 19 displays a schematic representation of the proposed Leprechaun open pit bench face angles. The proposed open pit at Leprechaun can be divided into two areas of similar bench geometry design. A northwest area composed of design sectors 1, 2, 3, 4, 7, 8, 9 and a southeast area composed of design sectors 5, 6.

In the northwest area kinematic analysis supplemented with limit-equilibrium analysis returned a maximum bench face angle of 75°. The southeast area analysis resulted in a maximum bench face angle of 65°. If using 18 m high benches (triple 6 m benches) a bench width of 8.1 m is recommended (Ryan and Pryor, 2000).

It should be noted that in the southeast area the S_1 foliation is interpreted to be the controlling structure. Our geotechnical investigation indicates that S_1 has an average dip of 57° and ranges from 49 – 72°. Locally, a bench face angle of 65° may not be achievable in the southeast design sectors due to the S_1 foliation daylighting out of the slope. Additionally, when using triple benching it is common that an offset between each 6 m bench of up to 1 m occurs due to constraints from drilling equipment. This can result in shallowing of the bench face angle (and ultimately the entire open pit).

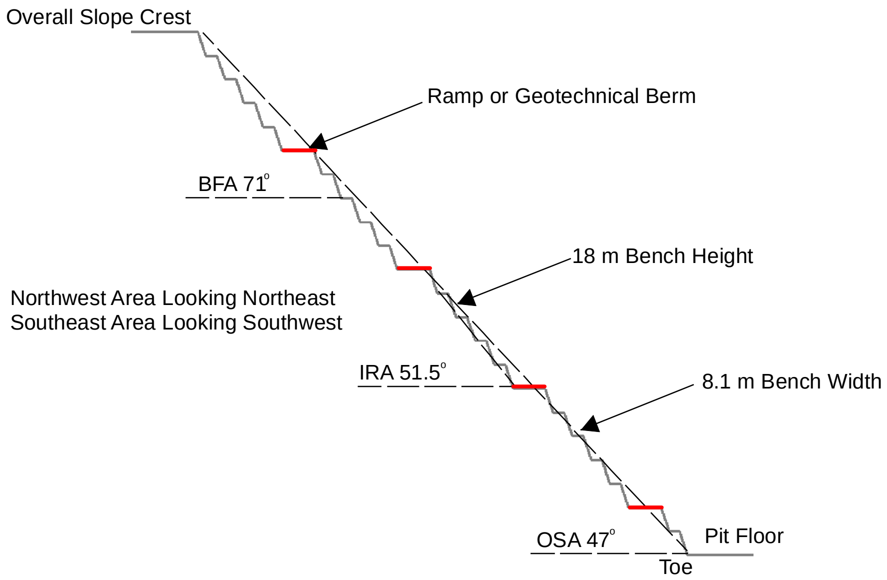
8.4 Inter-Ramp Slope Stability

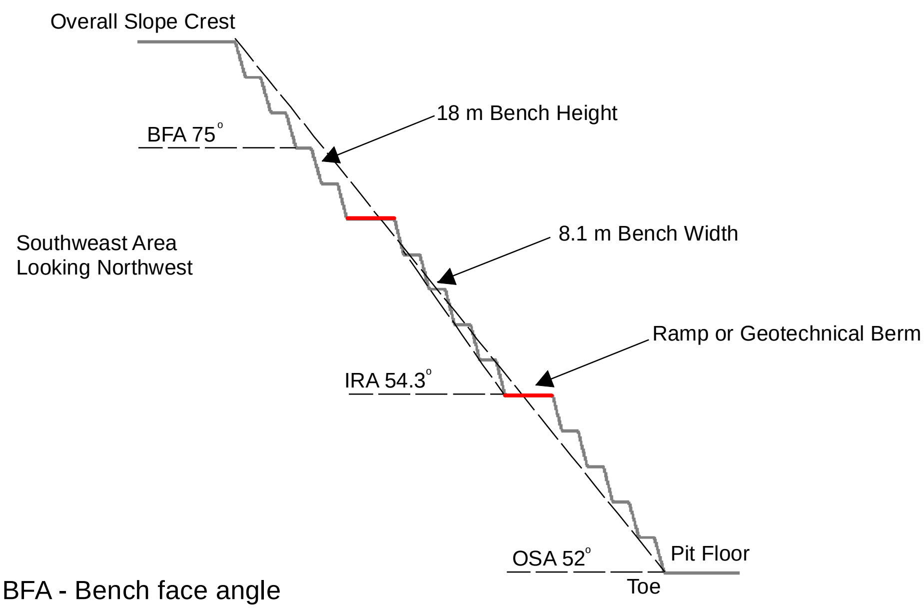
The inter-ramp angle is defined as the angle between the toe of a slope where a ramp section passes, and the toe of a bench located above (Figure 3). With the exception of modelled faults and the dominant discontinuity sets (S_1 and JS_1), it is assumed that the persistence of all the other discontinuities are less than the bench height (18 m). Therefore, at the inter-ramp scale, most often, the structures of concern are major multi-bench scale faults and persistent structural fabrics. Using the orientation of the faults developed as part of our structural geology model (Figures 9 and 10) a kinematic analysis was carried out for the inter-ramp angles. The acceptance criteria (Table 23) for inter-ramp design was used (i.e. PoF$\leq$20%) to confirm the maximum recommended inter-ramp angle.

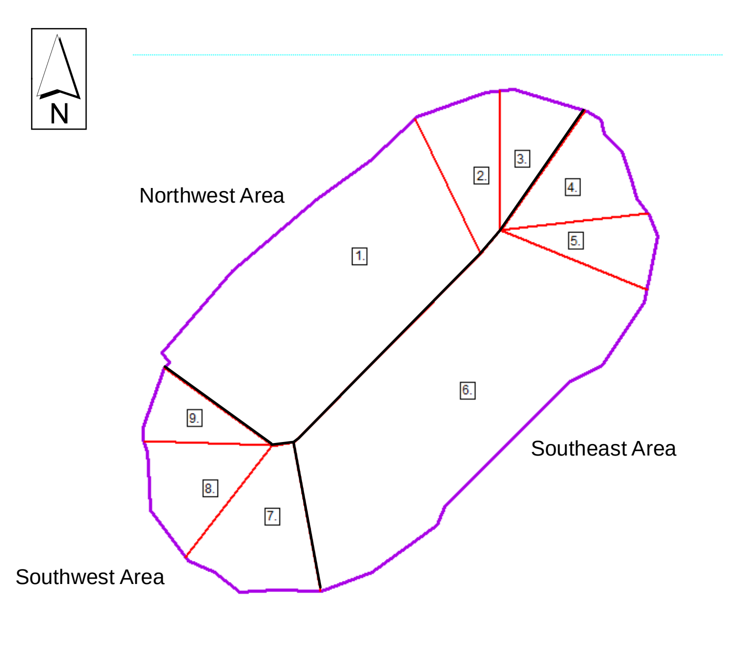
8.4.1 Inter-Ramp Analysis – Marathon

A combination of kinematic analysis and limit-equilibrium modelling in Rocscience Slide2 (Rocscience, 2018) was completed to evaluate the inter-ramp slope stability. Using the major structures (i.e. faults and dominant discontinuity sets) that occur in each design sector for the Marathon deposit kinematic analysis was performed for the proposed inter-ramp angles. It was found that the inter-ramp angle is controlled by the maximum recommended overall slope angle in each design sector for the whole pit. The inter-ramp angles were calculated to be 51.5^{o} for the northwest and southeast design sectors and 54.3^{o} for the southwest design sectors (Figure 18).

Using these inter-ramp angles and the structural data summarized above, kinematic analysis for each design sector resulted in no kinematically admissible failure mechanism at the inter-ramp scale. To further evaluate the inter-ramp slope stability, modelling was completed using limit-equilibrium software Slide2 (Rocscience, 2018). All analysis was conducted assuming a saturated slope using the Janbu method and Generalized Hoek Brown Criteria for the rock mass strength.







Marathon pre-feasibility open pit outline provided by Marathon (January 2020)

BFA - Bench face angle
IRA - Interramp slope angle
OSA - Overall slope angle

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Figure 18 - Typical Pit Wall Section - Marathon Deposit		
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Table 26 summarizes the Hoek-Brown values used to evaluate the factor of safety (FOS) of the inter-ramp slopes. This modelling indicated a minimum FOS ranging from 3.1 (sectors 1 and 2) to a minimum FOS of 4.9 (Sectors 8 and 9) for the anticipated average rock mass conditions. The design areas results obtained from limit-equilibrium modelling exceeded our design criteria FOS of 1.2.

Table 26 - Summary of Generalized Hoek-Brown Criteria, Limit-Equilibrium Modelling- Marathon

Geotechnical Domain	mi	RMR_{76} [1.] Mean	RMR_{76} [1.] Min.	UCS_{mean} [2.] (MPa)	UCS_{min} [2.] (MPa)	D Factor [3.]
Quartz Eye Porphyry	11.2	68	59	145	89	0.85
Mafic Intrusive	25	66	55	120	62	0.85
Conglomerate	11.1	67	59	113	54	0.85

Notes: 1. See Appendix D; Note GSI~=RMR_{76} for RMR_{76}>18 (Hoek et al., 1995).

2. See Appendix B

3. Damage factor of 0.85 used within 20 m of slope face.

A full summary of our kinematic analysis and limit equilibrium modelling results for the inter-ramp slope stability are included in Appendix F and Appendix G. Additionally, the results are summarized in Table 27.

Table 27 - Inter-ramp Slope Stability Summary

Design Sector	Slope Orientation (°)	Probability of Failure (%) [1.]	Limit Equilibrium FOS [2.]	Inter Ramp Angle (°)
1	136	0.0	3.1	51.5
2	164	0.0	3.1	51.5
3	194	0.0	4.0	51.5
4	241	0.0	4.0	51.5
5	273	0.0	4.0	51.5
6	316	0.0	4.0	51.5
7	014	0.0	3.7	54.3
8	071	0.0	4.9	54.3
9	105	0.0	4.9	54.3

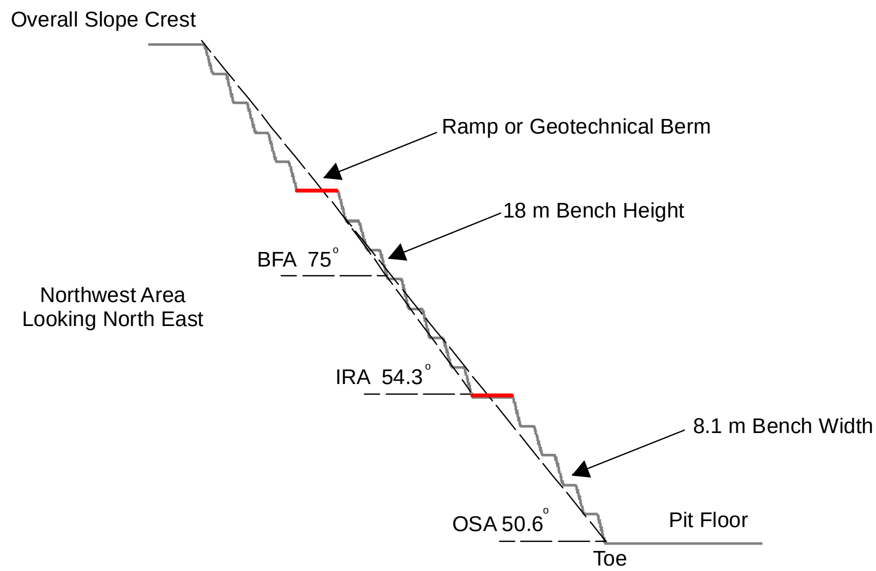
Notes: 1. From kinematic analysis

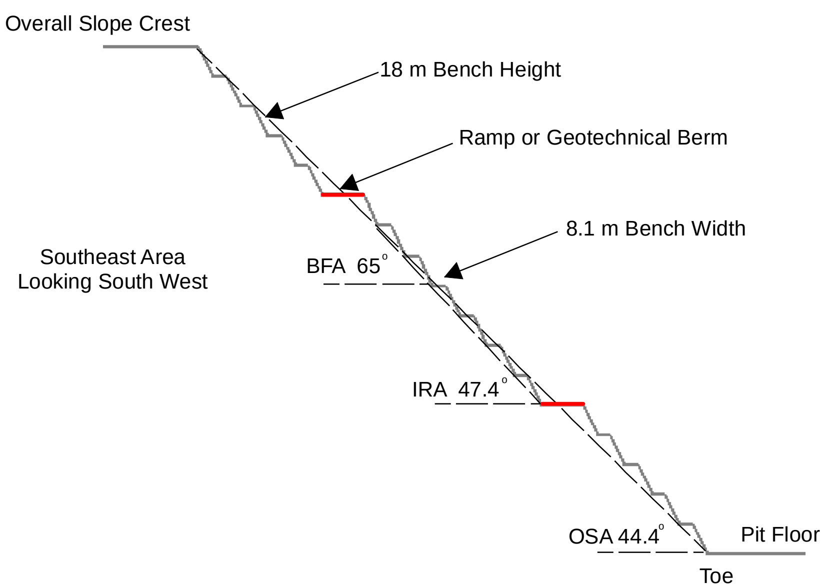
2. From limit-equilibrium modelling Slide (Rocscience, 2019)

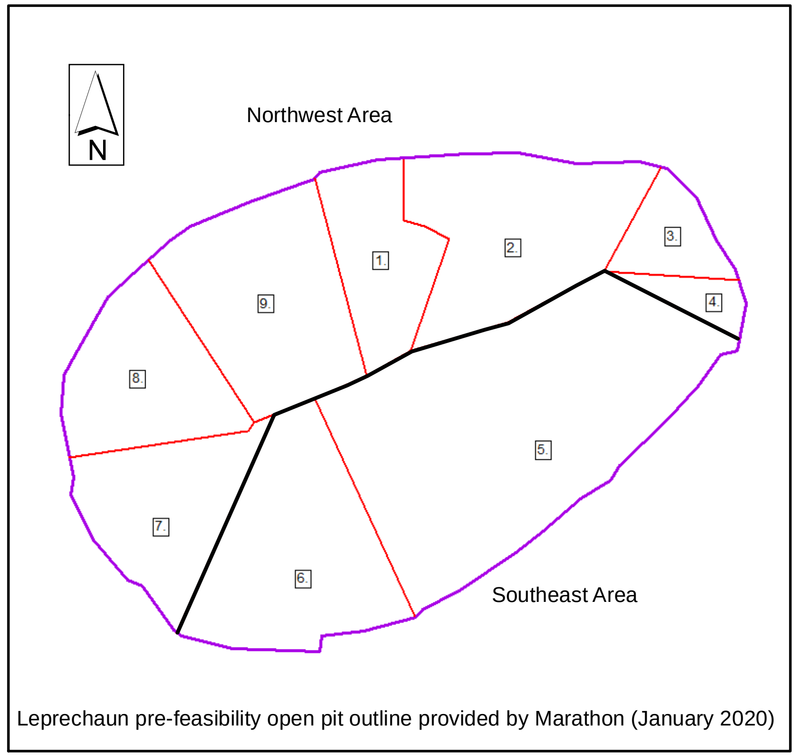
It should be noted that the FOS reported is for the anticipated average rock mass conditions. Limit – equilibrium modelling of the anticipated worse rock mass conditions was also performed and did not meet the design criteria in some domains (See Appendix F and Appendix G). In the event that the rock mass conditions are closer to the worse conditions, at the inter ramp scale (i.e. 90 m high slope with average $RMR_{76} \leq RMR_{76\ Min}$), slope depressurization may be required.

8.4.2 Inter-ramp Analysis – Leprechaun

A combination of kinematic analysis and limit-equilibrium modelling in Rocscience Slide2 (Rocscience, 2018) was completed to evaluate the inter-ramp slope stability. Using the major structures (i.e. faults and dominant discontinuity sets) that occur in each design sector for the Leprechaun deposit kinematic analysis was performed for the proposed inter-ramp angles. It was found that the inter-ramp angle is controlled by the maximum achievable bench face angle in each design sector for the whole pit. The inter-ramp angles were calculated to be 54.3^{o} for the northwest design sectors and 47.4^{o} for the southeast design sectors (Figure 19).







BFA - Bench face angle
IRA - Interramp slope angle
OSA - Overall slope angle

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Figure 19 - Typical Pit Wall Section - Leprechaun Deposit

Date: 02-07-20	Drawn By: AG	TERRANE GEOSCIENCE INC.
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NO. 19-0015-H	Approved By: TLG	www.terranegeoscience.com

Using these inter-ramp angles, kinematic analysis for each design sector resulted in no kinematically admissible failure mechanism at the inter-ramp scale. To further evaluate the inter-ramp slope stability, modelling was completed using the limit-equilibrium software Slide2 (Rocscience, 2018). The analysis was conducted using the Janbu method and Generalized Hoek Brown Criteria for the rock mass strength. Table 28 summarizes the Hoek-Brown values used to evaluate the factor of safety (FOS) of the inter-ramp slopes. This modelling indicated a minimum FOS of 2.5 in design sectors 4, 5, and 6 and a minimum FOS of 4.2 in the remaining design sectors. For both design areas the resulted obtained from limit-equilibrium modelling exceeded our design criteria FOS of 1.2.

Table 28 - Summary of Generalized Hoek-Brown Criteria, Limit-Equilibrium Modelling - Leprechaun

Geotechnical Domain	mi	RMR_{76}[1.] Mean	RMR_{76}[1.] Min.	UCS_{mean}[2.] (MPa)	UCS_{min}[2.] (MPa)	D Factor[3.]
Trondhjemite	14.7	71	60	112	72	0.85
Conglomerate	11.1	71	60	126	54	0.85

Notes:
1. See Appendix D; Note GSI~=RMR_{76} for RMR_{76}>18 (Hoek et al., 1995).
2. See Appendix B
3. Damage factor of 0.85 used within 20 m of slope face.

A full summary of our kinematic analysis and limit equilibrium modelling results for inter-ramp slope stability are included in Appendix F and Appendix G. Additionally, the results are summarized below in Table 29.

Table 29 - Inter-Ramp Slope Stability Summary

Design Sector	Slope Orientation (°)	Probability of Failure (%)[1.]	Limit Equilibrium FOS[2.]	Inter Ramp Angle (°)
1	188	0.0	4.2	54.3
2	174	0.0	4.2	54.3
3	234	0.0	4.2	54.3
4	269	0.0	2.5	47.4
5	319	0.0	2.5	47.4
6	357	0.0	2.5	47.4
7	57	0.0	4.2	54.3
8	113	0.0	4.2	54.3
9	155	0.0	4.2	54.3

Notes:
1. From kinematic analysis
2. From limit-equilibrium modelling Slide (Rocscience, 2019)

8.5 Overall Slope Stability

The overall slope angle is defined as the angle between the toe of a slope at the base of the pit, and the crest of a pit at surface (Figure 3). With the exception of modelled faults and the dominant discontinuity sets (S_1 and JS_1), it is assumed that the persistence of all the other discontinuities are less than the bench height (18 m). Therefore, at the overall slope scale, most often, the structures of concern are major multi-ramp scale faults and persistent structural fabrics. Using the orientation of the faults developed as part of our structural geology model (Figures 9 and 10) and the rockmass characteristics of the geotechnical domains, a limit-equilibrium analysis was carried out for the overall slope stability. The acceptance criteria (Table 23) for overall slope design was used (i.e. $FOS \geq 1.3$) to confirm the maximum recommended overall slope.

The overall slope was compared to benchmarking studies at similar sized open pit mines.

8.5.1 Overall Slope Stability Analysis – Marathon

The bench geometry as described above, with geotechnical berms and/or ramps, spaced vertically at a minimum of 90 m results in overall slopes with angles of 47° and 52° for the northwest/southeast design areas and the southwest design area respectively (Figure 18).

Analysis of the overall slope stability at the geometry reported above was performed using limit-equilibrium software Slide (Rocscience, 2019). Analysis was completed using the Janbu method and Generalized Hoek Brown Criteria for the rock mass strength as summarized in Table 26.

Table 30 below summarizes the results of the limit-equilibrium analysis and Appendix G contains the detailed model results. Additionally, Table 26 contains the Generalized Hoek-Brown criteria inputs used in our modelling of the overall slope. Overall slope stability modelling using the mean design values (Table 26), assuming a saturated slope returned FOS values of 2.5 for the NW design sectors, 2.4 for the southeast design sectors, and 3.3 for the southwest design sectors. Further, this modelling, included known faults as discrete failure planes with a friction angle of 28° and a cohesion (ϕ)=0 kPa. However, it was found they did not influence the overall slope FOS.

Table 30 - Summary of Limit-Equilibrium Modelling - Marathon

Design Area	Overall Slope Angle[4.]	Design Values	FOS
Northwest Area[1.]	47.0°	Mean (RMR_{76}, UCS)	2.5
		Minimum (RMR_{76}, UCS)	1.5
Southeast Area[2.]	47.0°	Mean (RMR_{76}, UCS)	2.4
		Minimum (RMR_{76}, UCS)	1.3
Southwest Area[3.]	52.0°	Mean (RMR_{76}, UCS)	3.3
		Minimum (RMR_{76}, UCS)	1.7

Notes: 1. Northwest area includes design sectors 1,2, and 3.

2. Southeast area includes design sectors 4,5, and 6.

3. Southwest area includes design sectors 7,8, and 9.

4. OSA assumed to be equal to or less than value presented.

Figure 20 displays the factor of safety curves for overall slope height vs. slope angle (Lutton, 1970; Hoek and Bray, 1981; Sjoberg, 1996) with data points representing various stable and unstable open pit developments. As shown on Figure 20, an overall slope height of 295 m at an overall angle of 47.0^{o} for the northwest and southeast design area has a FOS between 1.0 and 1.3. Additionally, an overall slope height of 270 m at an angle of 52.0^{o} give the southwest design area an FOS~1. These curves are empirical summaries of open pit mine stability and act as a benchmark comparison between the Marathon open pit and other large open pits.

8.5.2 Overall Slope Stability Analysis – Leprechaun

The bench geometry as described above, with two geotechnical berms, two ramp sections, or a geotechnical berm and a ramp, result in overall slopes with angles of 50.6^{o} and 44.4^{o} for the northwest and southwest design areas respectively (Figure 19).

Analysis of the overall slope stability at the geometry reported above was performed using limit-equilibrium software Slide (Rocscience, 2019). We have used the Janbu method and Generalized Hoek Brown Criteria for the rock mass strength as summarized in Table 26.

Table 31 below summarizes the results of the limit-equilibrium analysis and Appendix G contains the detailed model results. Additionally, Table 28 contains the Generalized Hoek-Brown criteria inputs used in our modelling of the overall slope. Overall slope stability modelling using the mean design values (Table 28), assuming a saturated slope returned FOS values of 2.7 and 2.1 respectively. Modelling for the southeast design sector included Fault 10 and the Valentine Lake Thrust Fault as discrete failure planes with a friction angle of 28^{o} and a cohesion (ϕ)=0 kPa.

Table 31 - Summary of Limit-Equilibrium Modelling - Leprechaun

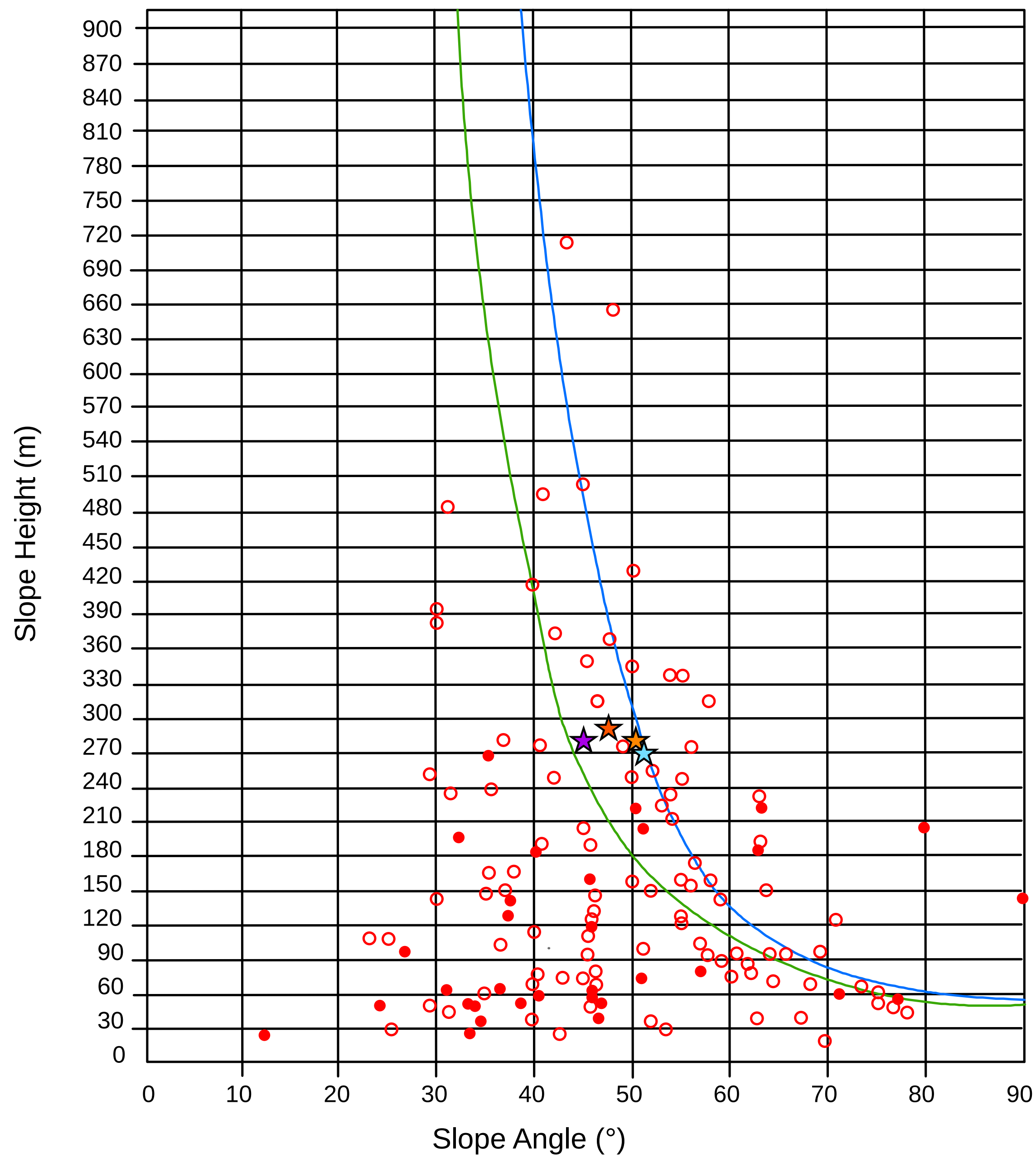
Design Area	Overall Slope Angle[3.]	Design Values	FOS
Northwest Area[1.]	50.6^{o}	Mean (RMR_{76}, UCS)	2.7
		Minimum (RMR_{76}, UCS)	1.3
Southeast Area[2.]	44.4^{o}	Mean (RMR_{76}, UCS)	2.1
		Minimum (RMR_{76}, UCS)	1.3

Notes: 1. Northwest area includes design sectors 1,2,3,7,8, and 9.

2. Southeast area includes design sectors 4,5, and 6.

3. OSA assumed to be equal to or less than value presented.

Figure 20 displays the factor of safety curves for overall slope height vs. slope angle (Lutton, 1970; Hoek and Bray, 1981; Sjoberg, 1996) with data points representing various stable and unstable open pit developments. As shown on Figure 20, an overall slope height of 285 m at an overall angle of 50.6^{o} for the northwest design area has a FOS~1. Additionally, for an overall slope height of 285 m at an angle of 44.4^{o} for the southeast design area has a FOS between 1.0 and 1.3. These curves are an empirical summary of a series of open pit mine stability worldwide and are intended to act as a benchmark comparison between the Leprechaun open pit development and other large open pits.



Notes: Original data points after Lutten 1970, Hoek and Bray 1981, and Sjoberg 1996

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FIGURE 20 -
ROCK SLOPE DESIGN
SLOPE HEIGHT
VS. SLOPE ANGLE

Legend

- Unstable Slope
- Stable Slope
- 1.0 Factor of Safety
- 1.3 Factor of Safety
- Leprechaun Pit - NW Area
- Leprechaun Pit - SE Area
- Marathon Pit - NW/SE Area
- Marathon Pit - SW Area

Drawn By: ATG	Date: 02/07/2020
Checked By: TLG	Rev 0

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9.0 OPEN PIT DESIGN SUMMARY

9.1 Design Summary – Marathon

Figure 18 displays the recommended bench configuration, inter-ramp angle, and overall angle for the proposed Marathon open pit. A summary of the recommended pit design for each design sector is included in Table 28 and in Appendix F and G.

Table 32 - Summary Open Pit Mine Recommendations - Marathon

Design Sector	Design Area	Bench Face Angle (BFA°)	Inter-Ramp Angle (IRA°)	Overall Slope Angle (OSA°)1.	Catch-Bench Width (m)2.	Overall Height (m)
1	NW	71	51.5	47.0	8.1	295
2	NW	71	51.5	47.0	8.1	295
3	NW	71	51.5	47.0	8.1	295
4	SE	71	51.5	47.0	8.1	295
5	SE	71	51.5	47.0	8.1	295
6	SE	71	51.5	47.0	8.1	295
7	SW	75	54.3	52.0	8.1	270
8	SW	75	54.3	52.0	8.1	270
9	SW	75	54.3	52.0	8.1	270

Notes: 1. A geotechnical berm or a ramp after a vertical height of 90 m. OSA assumed to be equal to or less than value presented.

2. Based in 18 m (triple 6 m) benches; using Ryan and Pryor (2000), Bench width (m)=0.2 x height +4.5.

It should be noted that the overall slope angle recommendations were guided by benchmarking studies and our experience with similar sized operating mines. There is an opportunity to increase the slope angles when more data becomes available (i.e. feasibility stage). Our geotechnical investigation indicates that S_1 has an average dip of 83° and ranges from 68 – 90°. Locally, a bench face angle of 71° and 75° may not be possible in the southeast and southwest design sectors due to the S_1 foliation. When using triple benching it is common that an offset between each 6 m bench of up to 1 m occurs due to constraints from drilling equipment. This can result in shallowing of the bench face angle (and ultimately the entire open pit).

It is considered best practice to include an extra wide catch bench at regular intervals in large open pits, for the purpose of providing additional safety to operators and equipment. The inclusion of a geotechnical berm is recommended for the Marathon open pit where a ramp does not cross the slope after a vertical height of 90 m. These geotechnical berms are recommended to be approximately 24 m wide. Our analysis of the overall slope includes these berms at intervals not exceeding 90 m.

9.2 Design Summary – Leprechaun

Figure 19 displays the recommended bench configuration, inter-ramp angle and overall angle for the proposed Leprechaun open pit. A summary of the recommended pit design for each design sector is included in Table 29 and in Appendix F and Appendix G.

Catch benches are designed to add protection against rock fall at the bench scale in open pit mines. Based on Ryan and Pryor (2000) and bench heights of 18 m.

Table 33 - Summary Open Pit Mine Recommendations - Leprechaun

Design Sector	Design Area	Bench Face Angle (BFA°)	Inter-Ramp Angle (IRA°)	Overall Slope Angle (OSA°)1.	Catch-Bench Width (m)2.	Overall Height (m)
1	NW	75	54.3	50.6	8.1	285
2	NW	75	54.3	50.6	8.1	285
3	NW	75	54.3	50.6	8.1	285
4	SE	65	47.4	44.4	8.1	285
5	SE	65	47.4	44.4	8.1	285
6	SE	65	47.4	44.4	8.1	285
7	NW	75	54.3	50.6	8.1	285
8	NW	75	54.3	50.6	8.1	285
9	NW	75	54.3	50.6	8.1	285

Notes: 1. A geotechnical berm or a ramp after a vertical height of 126 m. OSA assumed to be equal to or less than value presented.

2. Based in 18 m (triple 6 m) benches; using Ryan and Pryor (2000), Bench width (m)=0.2 x height +4.5.

It should be noted that in the southeast area (design sectors 1,2,3,7,8, and 9) the S_1 foliation is interpreted to be the controlling kinematic structure. Our geotechnical investigation indicates that S_1 has an average dip of 57° and ranges from 49 – 72°. Locally, a bench face angle of 65° may not be possible in the southeast design sectors due to the S_1 foliation. When using triple benching it is common that an offset between each 6 m bench of up to 1 m occurs due to constraints from drilling equipment. This can result in shallowing of the bench face angle (and ultimately the entire open pit).

It is considered best practice to include an extra wide catch bench at regular intervals in large open pits, for the purpose of providing additional safety to operators and equipment. The inclusion of a geotechnical berm is recommended for the Leprechaun open pit where a ramp does not cross the slope after a vertical height of 126 m. These geotechnical berms are recommended to be approximately 25 m wide. Our analysis of the overall slope includes berms at intervals not exceeding 126 m.

10.0 OPERATIONAL RECOMMENDATIONS – SLOPE MANAGEMENT

Terrane considers the slope design recommendations within this report to represent a robust design based on the data available at the time of writing. However, this design is not considered conservative and represents the inter-play between pit safety and economics based on commonly accepted practices. It is considered vital that the condition of the pit walls be maintained over time through an on-site program of monitoring and active management practices. Below we highlight some common operational considerations.

10.1 Controlled Blasting

Rock strength can be greatly affected by blast disturbance which may influence the bench to overall pit scale stability. At the bench to inter-ramp scale, slope stabilities are often driven by on-going deterioration of the wall face. This deterioration or raveling is generally initiated by small, discontinuity bounded, rock blocks known as key blocks. To achieve the steepest possible bench faces it is imperative that such on-going deterioration be limited during blasting activities.

Controlled blasting techniques should be implemented to facilitate steeper final bench face slopes by minimizing face damage from blast disturbance. Typically, controlled blasting involves completing a series of small diameter blast holes known as a pre-shear line of holes. This works best in massive to hard rock. Blast hole lengths are generally staggered to avoid intercepting the crest of the bench below.

10.2 Bench Maintenance

Bench faces should be regularly maintained during mining operations and where possible kept clear of debris to insure they function as designed. Scaling is an important component of a bench face maintenance program and is generally conducted after blasting has occurred in areas where safe access is possible.

10.3 Groundwater and Slope Depressurization

Control measures should be implemented so that any surface water is prevented from flowing into the pit and saturating the pit slopes. Further, ponding of water within the pit and on catch benches should be avoided, where possible. A slope of 1-2% on the catch benches is recommended to facilitate drainage of the benches.

Additionally, vibrating wire piezometers should be installed at regular intervals around each pit perimeter to allow for detailed monitoring of groundwater levels and pore-water pressures. If high pore-water pressures are encountered during mining, slope depressurization measures may need to be implemented. These typically consist of horizontal drains drilled into the slope allow for drainage of the slope.

10.4 Geotechnical Monitoring

Geotechnical monitoring and field data collection of the open pit walls is recommended throughout the life of the open pits. The following is recommended as part of a geotechnical monitoring program:

- Geotechnical mapping and regular inspection of benches. This should include tension crack mapping along the crest of benches.
- Geological and major structures mapping
- Maintain an up to date lithological and structural model
- Implement a geomechanical testing program to confirm all design values
- Develop a program to monitor any potential large-scale movements on the open pit slopes. This may include surface prism displacement monitoring and/or radar monitoring.
- Regular 3rd part inspections and slope stability audits.
- Comparison and adjustment of the recommended design based on performance monitoring of the slope.

11.0 RECOMMENDATIONS FOR FUTURE WORK

Open pit slope design recommendations for bench geometry (bench face angle and bench width), inter-ramp, and over all slope angles have been provided for the proposed Marathon and Leprechaun open pits. These designs are based on the data available at the time of writing and Terrane believes they represent a Level 2 or pre-feasibility level design (Read and Stacey, 2009).

To increase data confidence and bring the geotechnical database to a Level 3 or Feasibility Level, Terrane recommends the following:

Geology Model – As additional exploration and resource drilling is completed the updated geological model should be incorporated into future geotechnical analysis.

Structural Model - Structural fabrics – Additional, targeted geotechnical drilling should be completed on both the Leprechaun and Marathon pits. The aim of this drilling will be to increase the data confidence related to structural fabrics. Particular attention should be given to design sectors 5 and 6 within the Leprechaun deposit as currently they are the shallowest recommended bench face angles. Additional scan-line mapping should be completed near the proposed open pits to further characterize the persistence, spacing and planarity of the structural fabrics.

Structural Model – Major Structures – Major structures interpreted from the pre-feasibility study, should be ground-truthed in the field. Targeted geotechnical drilling should be completed to characterize major structures that may affect open pit stability. An assessment of the major modelled structures and their potential connectivity to large water bodies should be completed.

Rock Mass Model – Additional geomechanical testing should be completed to further characterize the material properties of the geotechnical domains and increase the data confidence of the rock mass model.

Hydrogeological Model – Additional hydrogeological field work should include, further Packer testing, piezometer installation, and development of a 3D hydrogeological model for each pit. Perform an initial assessment of slope depressurization and dewatering requirements.

Geotechnical Model – Following completion of further field work the geotechnical model should be updated to incorporate any changes and design optimization. This additional slope design work should include:

- Optimize the bench face and inter-ramp angle designs,
- Perform more advanced numerical modeling including, 3D limit equilibrium modelling and/or finite element modelling,
- Complete a break-out analysis of acoustic televiewer data to allow for an estimate of in situ stresses to evaluate the influence of in situ stresses on open pit design.
- Complete a geotechnical assessment of the overburden near each of the proposed open pits to characterize the overburden and design safe and economical slopes.

12.0 CLOSURE

We trust that this report meets the needs of Marathon related to open pit mine design for the proposed Marathon and Leprechaun open pits.

This report was prepared and reviewed by:

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LICENSED PROFESSIONAL ENGINEER
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Tony Gilman
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April 22, 2020
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13.0 REFERENCES

Barbor, D., 1990. Valentine Lake gold prospect; in Metallogenic Framework of Base and Precious Metals Deposits, Central and Western Newfoundland, (ed.) H.S. Swinden, D.T.W. Evans and B.F. Kean; 8th IAGOD Symposium, Field Trip Guidebook 1, Geological Survey of Canada, Open File 2156, p. 73-76

Bieniawski, Z., 1976. Rock Mass Classification in Rock Engineering. *Exploration for Rock Engineering: Proceedings of the Synposium on Exploration for Rock Engineering*, (pp. 97-106). Johannesburg.

Blackwood, R.F., 1982. Gander (Mount Peyton E), Newfoundland. Map 80-198. Scale 1:50,000. In Geology of the Gander Lake and Gander River area. Government of Newfoundland and Labrador, Department of Mines and Energy, Report 82-04, 63 pp. NFLD/13/1312b.

Brown, E.T. (ed.), 1981. Suggested methods for determining intact rock strength. Rock characterization, testing and monitoring – ISRM suggested methods. Pp 135-137.

Dunsworth, S. and Walford, P., 2018. Marathon's Valentine Lake Property – A developing multi-millon ounce gold camp in central Newfoundland, Canada. Atlantic Geology, Volume 54, Abstracts, pp 133-145.

Freeze, R.A. & Cherry, J.A., 1979. Groundwater. Prentice Hall: Englewood, Cliffs, N.J.

Hrabi B., and Siddorn, J., P., 2013. Structural Geology Investigation of the Valentine Lake Gold Project, Newfoundland. Internal Maraton Gold Corp. report, pp 27.

Hoek, E., & Bray, J., 1981. Rock Slope Engineering, 3rd ed. *Institute of Mining and Metallurgy*, 358.

Hoek, E. (2000). Rock-Mass Properties for Surface Mines. In M. a. Society for Mining, *Slope Stability in Surface Mining*, pp. 59-69.

Hoek, E., Carranza-Torres, C., Corkum, B., 2002. Hoek-Brown Failure Criterion – 2002 Edition. Proceedings of the Fifth North American Rock Mechanics Symposium (NARMS-TAC), University of Toronto Press, Toronto, pp 267-273.

Hoek, E., Deiderichs, M.S., 2006. Emprical estimation of rock mass modulus. International Journal of Rock Mechanics and Mining Sciences, pp 203-215.

Lincoln, N., Farmer, R., Eccles, R. and Deering, P., 2018. Technical Report Preliminary Economic Assessment of the Valentine Lake Gold Project, Newfoundland, NL, Canada. National Instrument 43-101.

Plinninger, R.J., and Alber, M., 2015. Assessment of Intact Rock Strength in Anisotropic Rock – Theory, Experiences and Implications on Site Investigations. EUROROCK 2015 & 64th Geomechanics Colloquium, Salzburg.

Read, J., & Stacey, P., 2009. *Guidelines for Open Pit Slope Design.* Australia: CSIRO Publishing.

Rocscience, 2019. Dips 7.0. *Version 7.0*. Toronto. Retrieved from www.rocscience.com

Rocscience, 2019. RocPlane Version 3.007 Toronto. Retrieved from www.rocsience.com

Rocscience, 2019. Swedge Version 6.018 Toronto. Retrieved from www.rocscience.com

Rocscience, 2019. Slide Version 8.022 Toronto. Retrieved from www.rocscience.com

Rocscience, 2019. RocData Version 5.009 Toronto. Retrieved from www.rocscience.com

Ryan, T., & Pryor, P. (2000). Designing Catch Benches and Inter-ramp Slopes. In M. a. Society for Mining, *Slope Stability in Sureface Mining* (pp. 27-38). Littleton.

Sjoberg, J., 1996. Large Scale Slope Stability in Open Pits - A Review. In L. U. Technology, *Division of Rock Mechanics* (pp. 97-187). Lulea.

Squires, G., C., 2005. Gold and Antimony Occurrences of the Exploits Subzone and Gander Zone: Review of Recent Discoveries and Their Interpretation. Current Research, Newfoundland and Labrador, Department of Natural Resources Geological Survey, pp 223-237.

Williams, H., 1979. Appalachian Orogen in Canada. Canadian Journal of Earth Science, volume 16, pp 792-807.

TERRANE GEOSCIENCE INC.

MARATHON GOLD

GEOTECHNICAL CORE LOG

Terranegeoscience.com

100 - 5435 Portland Place, B3K 2Y7

Halifax, Nova Scotia, Canada

PROJECT: Marathon Gold Corporation - Valentine Gold Project
Pre-Feasibility Geotechnical Investigation

DEPOSIT: Leprechaun

BOREHOLE ID: VL-GT-19-01

AZIMUTH: 115.2° **DIP:** -54.8°

COORDINATES: 486294.2 E 5356228.5 N

ELEVATION: 397.1 masl

DATUM: UTM NAD83 z21

START: 0.0 m

END: 374.0 m

CORE SIZE: HQ3

DEFINITIONS

RQD - Rock Quality Designation (%)
IRS - Intact Rock Strength (Brown, 1981)
STR - Major Structure (ie. Fault)
RMR76 - Rock Mass Rating (Bieniawski,1976)
FF/m - Number of fractures per meter
TCA - Angle relative to core axis

Depth | Geotech Domain | Lithology | Description | RQD% | Rock Strength | Fracture Frequency | RMR76 | Major Structure

Depth scale: 0 to 124.76

Geotech Domain: TRJ

Lithology / Description:
- OVBN: Soil and Boulders
- TNJ: fg-mg grey to green trondhjemite with feldspar phenocrysts; rich in sercite.
- TNJ/MQTP: Pale peach, mg, w/ pervasive trml wisps up to 1 cm wide +/- QC lenses & trace py.
- TNJ: fg-mg grey to green tronj w/ patchy chl in groundmass & sporadic QC wisps <1cm wide. Slivers of fg dark green mafic dike. One QT vein w/ 10cm pink alt halo from 231.02 to 231.07 m

RQD (%) 0–100; IRS (MPa) 0–200; FF/m 0–15; RMR'76 0–100; Structure (TCA) 0–90

RQD (%)	IRS	FF/m	RMR'76
46	R4	4.47	47
96	R5	3.33	64
84	R5	4.67	61
92	R5	4.00	64
88	R5	4.67	61
87	R5	5.00	61
95	R5	3.00	74
87	R5	4.00	61
83	R5	5.33	61
97	R5	3.00	74
100	R5	3.33	74
97	R5	2.00	74
100	R5	2.33	74
100	R5	1.67	74
100	R5	1.00	79
100	R5	2.00	74
97	R5	2.00	74
99	R5	3.00	74
100	R5	2.33	74
100	R5	1.67	74
97	R5	2.00	74
100	R5	1.00	74
98	R5	1.33	74
100	R5	1.33	79
100	R5	1.67	74
99	R5	2.00	74
98	R5	2.67	74
100	R5	1.00	79
100	R5	0.33	79
94	R5	2.00	74
100	R5	0.33	79
99	R5	1.00	74
100	R5	1.00	74
98	R5	1.67	74
100	R5	2.67	74
87	R5	4.00	71
92	R5	3.00	74
100	R5	2.33	74
97	R5	2.67	74
95	R5	3.33	64
98	R5	1.67	74

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GEOTECHNICAL CORE LOG

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Halifax, Nova Scotia, Canada

MARATHON GOLD

PROJECT: Marathon Gold Corporation - Valentine Gold Project
Pre-Feasibility Geotechnical Investigation
DEPOSIT: Leprechaun
BOREHOLE ID: VL-GT-19-01
AZIMUTH: 115.2° **DIP:** -54.8°
COORDINATES: 486294.2 E 5356228.5 N
ELEVATION: 397.1 masl

DATUM: UTM NAD83 z21
START: 0.0 m
END: 374.0 m
CORE SIZE: HQ3 **PAGE:** 2 of 4

DEFINITIONS

RQD - Rock Quality Designation (%)
IRS - Intact Rock Strength (Brown, 1981)
STR - Major Structure (ie. Fault)
RMR76 - Rock Mass Rating (Bieniawski,1976)
FF/m - Number of fractures per meter
TCA - Angle relative to core axis

Depth: 124.76 – 244.04 m

Geotech Domain: TRJ

Lithology: TNJ

Description: fg-mg grey to green tronj w/ patchy chl in groundmass & sporadic QC wisps <1cm wide. Slivers of fg dark green mafic dike. One QT vein w/ 10cm pink alt halo from 231.02 to 231.07 m

RQD (%)	Rock Strength IRS (MPa)	Fracture Frequency FF/m	RMR'76
97	R5	3.00	74
88	R5	5.00	61
99	R5	2.33	74
98	R5	2.33	74
100	R5	3.00	74
97	R5	3.00	74
95	R5	4.00	64
67	R4	10.00	52
56	R4	8.67	52
63	R4	7.67	52
55	R4	11.67	52
81	R4	8.33	56
95	R4	4.33	59
91	R4	5.33	59
88	R4	5.00	56
78	R4	7.00	56
88	R4	6.67	56
89	R4	5.00	56
98	R4	3.00	69
96	R4	4.33	59
91	R4	4.00	59
91	R4	4.33	59
38	R4	18.67	47
95	R5	4.00	74
98	R5	3.00	74
100	R5	1.67	74
97	R5	3.00	74
99	R5	3.33	74
98	R5	3.67	64
89	R5	4.00	61
99	R5	2.67	64
95	R4	3.33	59
100	R4	2.67	59
95	R4	3.67	69
96	R4	3.33	59
90	R5	4.00	61
99	R5	2.67	74
100	R5	1.67	74
93	R5	3.33	74
94	R5	4.00	74

Major Structure: Structure (TCA)

TERRANE GEOSCIENCE INC.

MARATHON GOLD

GEOTECHNICAL CORE LOG

Terranegeoscience.com
100 - 5435 Portland Place, B3K 2Y7
Halifax, Nova Scotia, Canada

PROJECT: Marathon Gold Corporation - Valentine Gold Project
Pre-Feasibility Geotechnical Investigation
DEPOSIT: Leprechaun
BOREHOLE ID: VL-GT-19-01
AZIMUTH: 115.2° **DIP:** -54.8°
COORDINATES: 486294.2 E 5356228.5 N
ELEVATION: 397.1 masl

DATUM: UTM NAD83 z21
START: 0.0 m
END: 374.0 m
CORE SIZE: HQ3 **PAGE:** 3 of 4

DEFINITIONS

RQD - Rock Quality Designation (%)
IRS - Intact Rock Strength (Brown, 1981)
STR - Major Structure (ie. Fault)
RMR76 - Rock Mass Rating (Bieniawski,1976)
FF/m - Number of fractures per meter
TCA - Angle relative to core axis

Depth: 244.04 – 363.32 (scale ticks 250, 260, 270, 280, 290, 300, 310, 320, 330, 340, 350, 360)

Geotech Domain: TRJ

Lithology	Description
TNJ	fg-mg grey to green tronj w/ patchy chl in groundmass & sporadic QC wisps <1cm wide. Slivers of fg dark green mafic dike. One QT vein w/ 10cm pink alt halo from 231.02 to 231.07 m
TNJ/MQTP	Beige to peach, mg, w/ QC veins & trml breccias up to 3cm with minor py. Moderate peach alt halos up to 20 cm wide. Pervasive trml slip faults.
TNJ	fg-mg grey-green trond w/localized beige-peach colored sections, sporadic qtz-calc±trml±py veining, pervasive trml bands trending high angle to CA
QTP	One large qtz dominated vein w/large patchy trml masses, fragments of sub angular host rock, minor cubic CP
TNJ	Mg peach trond w/pale blue sub rounded qtz eyes. Pervasive spotty background chl. Sporadic veining.

RQD% (RQD (%))	Rock Strength (IRS (MPa))	Fracture Frequency (FF/m)	RMR76 (RMR'76)
94	R5	4.00	74
94	R5	4.67	74
100	R5	3.33	74
95	R4	4.00	59
95	R5	4.00	64
99	R5	3.00	64
95	R5	3.33	64
98	R5	2.33	74
96	R5	3.00	74
98	R5	2.00	74
100	R5	2.00	74
98	R5	2.33	74
100	R5	3.00	64
93	R5	4.33	64
76	R5	2.67	71
100	R5	2.00	74
94	R4	4.67	69
100	R5	2.67	74
94	R5	1.67	74
97	R5	5.33	64
99	R5	3.67	74
96	R5	3.00	74
95	R5	2.67	74
99	R5	2.67	74
93	R5	4.00	74
100	R5	2.33	74
92	R5	3.00	64
92	R5	3.33	74
93	R5	3.33	64
95	R6	2.67	77
86	R5	6.00	61
94	R5	2.00	74
94	R5	3.33	74
79	R5	6.67	61
84	R5	7.00	61
85	R5	7.33	61
81	R5	5.67	61
100	R5	2.00	74
100	R5	1.33	74
83	R5	6.67	61
94	R5	4.33	74

Major Structure: Structure (TCA) (scale 30, 60, 90)

TERRANE GEOSCIENCE INC.

GEOTECHNICAL CORE LOG

Terranegeoscience.com
100 - 5435 Portland Place, B3K 2Y7
Halifax, Nova Scotia, Canada

PROJECT: Marathon Gold Corporation - Valentine Gold Project
Pre-Feasibility Geotechnical Investigation
DEPOSIT: Leprechaun
BOREHOLE ID: VL-GT-19-01
AZIMUTH: 115.2° **DIP:** -54.8°
COORDINATES: 486294.2 E 5356228.5 N
ELEVATION: 397.1 masl

MARATHON GOLD

DATUM: UTM NAD83 z21
START: 0.0 m
END: 374.0 m
CORE SIZE: HQ3 **PAGE:** 4 of 4

DEFINITIONS

RQD - Rock Quality Designation (%)
IRS - Intact Rock Strength (Brown, 1981)
STR - Major Structure (ie. Fault)
RMR76 - Rock Mass Rating (Bieniawski,1976)
FF/m - Number of fractures per meter
TCA - Angle relative to core axis

Depth	Geotech Domain	Lithology	Description	RQD% (RQD (%))	Rock Strength (IRS (MPa))	Fracture Frequency (FF/m)	RMR76 (RMR'76)	Major Structure (Structure (TCA))
363.32	TRJ	TNJ	Mg peach trond w/pale blue sub rounded qtz eyes. Pervasive spotty background chl. Sporadic veining.	94	R5	4.33	74	
				100	R5	1.67	74	
370				86	R5	3.67	61	
374				65	R5	6.67	57	

Axis scales: RQD (%) 0, 25, 50, 75, 100; IRS (MPa) 0, 50, 100, 150, 200; FF/m 0, 5, 10, 15; RMR'76 0, 25, 50, 75, 100; Structure (TCA) 0, 30, 60, 90

PROJECT: Marathon Gold Corporation - Valentine Gold Project
Pre-Feasibility Geotechnical Investigation
DEPOSIT: Leprechaun
BOREHOLE ID: VL-GT-19-02
AZIMUTH: 299.5° **DIP:** -45.2°
COORDINATES: 486902.1 E 5355940.0 N
ELEVATION: 403.8 masl

DATUM: UTM NAD83 z21
START: 0.0 m
END: 401.0 m
CORE SIZE: HQ3 **PAGE:** 1 of 4

TERRANE GEOSCIENCE INC.

MARATHON GOLD

GEOTECHNICAL CORE LOG

Terranegeoscience.com

100 - 5435 Portland Place, B3K 2Y7

Halifax, Nova Scotia, Canada

DEFINITIONS

RQD - Rock Quality Designation (%)
IRS - Intact Rock Strength (Brown, 1981)
STR - Major Structure (ie. Fault)
RMR76 - Rock Mass Rating (Bieniawski,1976)
FF/m - Number of fractures per meter
TCA - Angle relative to core axis

Depth (0 – 124.76) | Geotech Domain: CG | Lithology: CNGL

Description: Green to green-grey fg chorlitic, polymictic conglomerate. The dominant elongated deformed clasts consists of blood-red jasper in upper 30m. Occasional distortion in gm can be seen surrounding clasts. of Sporadic QC±P rare trml veining seen throughout.

RQD (%)	Rock Strength IRS (MPa)	Fracture Frequency FF/m	RMR76	Major Structure (TCA)
8	R3	26.32	28	
20	R3	20.00	28	
35	R3	12.67	38	
18	R3	22.67	28	
47	R3	10.00	44	
17	R4	11.67	42	
81	R5	4.00	61	
100	R5	1.33	74	
94	R5	2.67	64	
94	R5	2.00	74	
100	R5	0.67	79	
93	R5	1.67	74	
100	R5	1.00	79	
100	R6	1.00	85	
100	R5	1.33	74	
89	R5	2.00	71	
87	R5	4.00	71	
94	R5	3.33	74	
96	R5	4.33	74	
89	R5	2.67	71	
83	R5	4.67	71	
99	R5	2.67	74	
94	R5	3.33	74	
99	R5	1.33	74	
96	R5	3.00	74	
94	R5	2.33	74	
91	R5	1.67	74	
100	R5	1.00	74	
79	R5	5.67	61	
100	R5	0.67	79	
89	R5	1.67	71	
87	R5	5.00	61	
81	R5	3.67	71	
92	R5	1.33	74	
99	R5	1.33	74	
100	R5	0.33	79	
100	R5	0.33	79	
100	R5	1.00	79	
100	R5	1.33	74	
100	R5	1.00	74	
97	R5	1.00	74	

TERRANE GEOSCIENCE INC.

MARATHON GOLD

GEOTECHNICAL CORE LOG

Terranegeoscience.com
100 - 5435 Portland Place, B3K 2Y7
Halifax, Nova Scotia, Canada

PROJECT: Marathon Gold Corporation - Valentine Gold Project
Pre-Feasibility Geotechnical Investigation
DEPOSIT: Leprechaun
BOREHOLE ID: VL-GT-19-02
AZIMUTH: 299.5° **DIP:** -45.2°
COORDINATES: 486902.1 E 5355940.0 N
ELEVATION: 403.8 masl
DATUM: UTM NAD83 z21
START: 0.0 m
END: 401.0 m
CORE SIZE: HQ3 **PAGE:** 2 of 4

DEFINITIONS
RQD - Rock Quality Designation (%)
IRS - Intact Rock Strength (Brown, 1981)
STR - Major Structure (ie. Fault)
RMR76 - Rock Mass Rating (Bieniawski,1976)
FF/m - Number of fractures per meter
TCA - Angle relative to core axis

Depth: 124.76 – 244.04 (ticks at 130, 140, 150, 160, 170, 180, 190, 200, 210, 220, 230, 240)

Geotech Domain: CG
Lithology: CNGL
Description: Green to green-grey fg chorlitic, polymictic conglomerate. The dominant elongated deformed clasts consists of blood-red jasper in upper 30m. Occasional distortion in gm can be seen surrounding clasts. of Sporadic QC±P rare trml veining seen throughout.

RQD% — RQD (%) (0, 25, 50, 75, 100); Rock Strength — IRS (MPa) (0, 50, 100, 150, 200); Fracture Frequency — FF/m (0, 5, 10, 15); RMR76 — RMR'76 (0, 25, 50, 75, 100); Major Structure — Structure (TCA) (30, 60, 90)

RQD%	Rock Strength	Fracture Frequency	RMR76	Major Structure
97	R5	0.67	79	
100	R5	2.00	74	
72	R5	1.00	72	
100	R5	0.00	79	
98	R5	1.00	79	
97	R5	1.33	74	
100	R5	1.33	74	
96	R5	1.67	74	
96	R5	1.33	74	
93	R5	0.67	74	
100	R5	0.33	87	
100	R5	1.33	79	
100	R5	1.00	87	
100	R5	0.33	87	
93	R5	1.67	74	
100	R5	0.33	87	
100	R5	0.67	79	
97	R5	0.67	79	
94	R5	2.00	74	
100	R5	1.00	79	
100	R5	0.67	87	
97	R5	1.00	79	
100	R5	0.67	87	
98	R5	0.67	79	
100	R5	1.67	74	
98	R5	1.00	79	
98	R5	1.33	79	
95	R5	1.33	79	
100	R5	2.33	74	
98	R5	1.00	74	
99	R4	1.00	69	
100	R5	0.67	79	
99	R5	1.00	74	
98	R5	0.67	79	
99	R4	2.33	69	
97	R4	1.33	69	
90	R4	2.33	66	
94	R4	1.33	69	
100	R5	1.00	74	
65	R4	2.67	62	

TERRANE GEOSCIENCE INC.

GEOTECHNICAL CORE LOG

Terranegeoscience.com

100 - 5435 Portland Place, B3K 2Y7

Halifax, Nova Scotia, Canada

PROJECT: Marathon Gold Corporation - Valentine Gold Project
Pre-Feasibility Geotechnical Investigation
DEPOSIT: Leprechaun
BOREHOLE ID: VL-GT-19-02
AZIMUTH: 299.5° **DIP:** -45.2°
COORDINATES: 486902.1 E 5355940.0 N
ELEVATION: 403.8 masl

MARATHON GOLD

DATUM: UTM NAD83 z21
START: 0.0 m
END: 401.0 m
CORE SIZE: HQ3 **PAGE:** 3 of 4

DEFINITIONS

RQD - Rock Quality Designation (%)
IRS - Intact Rock Strength (Brown, 1981)
STR - Major Structure (ie. Fault)
RMR76 - Rock Mass Rating (Bieniawski,1976)
FF/m - Number of fractures per meter
TCA - Angle relative to core axis

Depth: 244.04 – 363.32 (axis marks 250, 260, 270, 280, 290, 300, 310, 320, 330, 340, 350, 360)

Geotech Domain: CG (244.04 to about 254); TRJ (below)

Lithology	Description
CNGL	Green to green-grey fg chorlitic, polymictic conglomerate. The dominant elongated deformed clasts consists of blood-red jasper in upper 30m. Occasional distortion in gm can be seen surrounding clasts. of Sporadic QC±P rare trml veining seen throughout.
CNGL; QTP	dak grey-green; matrix-dominant w/ polymitic, polymodal clasts locally elongted along foliation; foliation @ 10-30 deg to CA; strong chl & serc alt
TNJ	grey-green; mg; moderately sheared near cong contact
TNJ; QTP	light to dark grey-green; mg; moderate chl alt QTP veins up to 10cm thick w/ bleached alt halos; patchy trml; local CP; veins @ low angle to CA (10-20)
TNJ	light to dark grey-green; mg; moderate chl alt
TNJ; QTP	QTP veins up to 3cm w/ CP up to 2cm & pink alt halos in light to dark grey-green; mg trondj
TNJ	light to dark grey-green; mg trondj
TNJ; QTP	QTP veins up to 4cm & trml+Bx veins up to 11cm w/ peach to beige alt halos. CP up to 3cm in veins. light to dark grey-green; mg trondj
TNJ; DYKE	light to dark grey-green; mg trondj w/ sporatic QTP veins & few MD < 1.5m
TNJ; QTP	grey-green, mg with mod chl alt generally irregular QTP veining up to 5cm thick with local weak stockwork zones
TNJ	grey-green, mg with mod chl alt
TNJ; QTP	FG-mg grey groundmass , peach alt halos associated w/ qtp veins <=19cm +/- CP

Columns: RQD% – RQD (%) scale 0, 25, 50, 75, 100; Rock Strength – IRS (MPa) scale 0, 50, 100, 150, 200; Fracture Frequency – FF/m scale 0, 5, 10, 15; RMR76 – RMR'76 scale 0, 25, 50, 75, 100; Major Structure – Structure (TCA) scale 0, 30, 60, 90

RQD%	Rock Strength	FF/m	RMR76	Major Structure
65	R4	2.67	62	
95	R4	2.33	69	
99	R5	2.33	74	
92	R4	2.67	69	
99	R5	1.33	74	
100	R5	1.67	74	
100	R5	1.00	74	
100	R5	1.33	74	
97	R4	2.00	59	
98	R5	0.67	79	
100	R5	0.00	87	
100	R5	1.00	74	
99	R5	0.67	74	
100	R5	0.67	79	
91	R5	3.33	74	
99	R5	0.67	79	
100	R5	1.33	74	
95	R5	1.67	74	
100	R5	1.33	74	
97	R5	1.33	74	
100	R5	0.67	79	
94	R5	2.00	74	
98	R5	0.67	79	
100	R5	0.67	79	
97	R5	0.67	79	
100	R5	0.67	79	
100	R5	1.33	74	
100	R5	1.67	74	
98	R5	0.67	74	
94	R5	2.33	74	
100	R5	2.00	74	
98	R5	0.33	79	
94	R5	2.67	64	
99	R5	0.33	79	
99	R5	0.67	79	
100	R5	0.00	92	
94	R5	1.33	79	
100	R5	1.00	74	
98	R5	0.67	79	
95	R5	2.00	74	
100	R5	1.33	74	

TERRANE GEOSCIENCE INC.

GEOTECHNICAL CORE LOG

Terranegeoscience.com

100 - 5435 Portland Place, B3K 2Y7

Halifax, Nova Scotia, Canada

MARATHON GOLD

PROJECT: Marathon Gold Corporation - Valentine Gold Project
Pre-Feasibility Geotechnical Investigation

DEPOSIT: Leprechaun

BOREHOLE ID: VL-GT-19-02

AZIMUTH: 299.5° **DIP:** -45.2°

COORDINATES: 486902.1 E 5355940.0 N

ELEVATION: 403.8 masl

DATUM: UTM NAD83 z21

START: 0.0 m

END: 401.0 m

CORE SIZE: HQ3 **PAGE:** 4 of 4

DEFINITIONS

RQD - Rock Quality Designation (%)
IRS - Intact Rock Strength (Brown, 1981)
STR - Major Structure (ie. Fault)
RMR76 - Rock Mass Rating (Bieniawski,1976)
FF/m - Number of fractures per meter
TCA - Angle relative to core axis

Depth	Geotech Domain	Lithology	Description
363.32	TRJ	TNJ; QTP	FG-mg grey groundmass , peach alt halos associated w/ qtp veins <=19cm +/- CP
370	TRJ	TNJ	Fg-mg grey-beige groundmass , eldspar phenocrysts & qtz eyes <=3mm, chl blebs
	TRJ	TNJ; QTP	Fg-mg grey green groundmass, little to nill alt halos w/ qtp veining <=6cm
380 – 390 – 400 – 401	TRJ	DYKE	Fg-mg green-grey w/ moderate-strong shearing at 30 deg to CA

RQD% (RQD (%))	Rock Strength (IRS (MPa))	Fracture Frequency (FF/m)	RMR76 (RMR'76)
100	R5	1.33	74
97	R5	2.00	74
100	R5	2.00	74
96	R5	2.00	74
100	R5	1.00	79
100	R5	1.67	82
98	R5	1.00	79
96	R5	2.00	74
100	R5	1.00	82
99	R5	1.67	74
100	R5	1.67	79
93	R5	2.00	74
89	R5	4.33	61

Major Structure: Structure (TCA) (scale 0–30–60–90)

TERRANE GEOSCIENCE INC.

GEOTECHNICAL CORE LOG

Terranegeoscience.com

100 - 5435 Portland Place, B3K 2Y7

Halifax, Nova Scotia, Canada

MARATHON GOLD

PROJECT: Marathon Gold Corporation - Valentine Gold Project
Pre-Feasibility Geotechnical Investigation
DEPOSIT: Leprechaun
BOREHOLE ID: VL-GT-19-03
AZIMUTH: 150.7° **DIP:** -64.9°
COORDINATES: 486274.2 E 5355785.1 N
ELEVATION: 386.0 masl

DATUM: UTM NAD83 z21
START: 0.0 m
END: 101.0 m
CORE SIZE: HQ3 **PAGE:** 1 of 1

DEFINITIONS

RQD - Rock Quality Designation (%)
IRS - Intact Rock Strength (Brown, 1981)
STR - Major Structure (ie. Fault)
RMR76 - Rock Mass Rating (Bieniawski,1976)
FF/m - Number of fractures per meter
TCA - Angle relative to core axis

Geotech Domain	Lithology	Description
	OVBN	soil
TRJ	TNJ	Fg-mg peach trondj w/ feldspar phenocrysts, rich in sericite, trml bands/slip faults w/ minor qtz infilling, three QTP veins <=4cm
TRJ	TNJ; QTP	Fg-mg peach trondj w/ feldspar phenocrysts, rich in sericite, trml bands/slip faults w/ minor qtz infilling, QTP veining <=13cm w/ peach alt halos
TRJ	TNJ	Fg-mg peach trondj w/ feldspar phenocrysts, rich in sericite, trml bands/slip faults w/ minor qtz infilling
TRJ	TNJ; QTP	Fg-mg peach trondj w/ feldspar phenocrysts, rich in sericite, trml bands/slip faults w/ minor qtz infilling; up to 4cm QTP vns w/ strong to moderate peach alt halos
TRJ	DYKE	Green-grey; fg; wispy qtz-carb veins throughout w/ some parallel to sub-parallel to fabric.
CG	CNGL	Green-grey fg matrix w/ up to 3cm wide, typically white, peach or pale green clasts that are elongated parallel to strong S1/L1; minor mafic dikes near contact

RQD (%)	IRS (MPa)	FF/m	RMR'76	Structure (TCA)
21	R4	10.33	42	
37	R4	9.67	47	
77	R4	3.67	56	
82	R4	3.33	66	
94	R4	3.33	69	
88	R4	4.33	56	
90	R5	4.33	61	
63	R4	8.00	49	
92	R5	5.00	64	
95	R5	3.67	74	
85	R5	4.67	61	
74	R5	7.00	57	
93	R5	4.33	64	
93	R5	2.33	61	
58	R4	9.67	49	
26	R4	9.00	44	
93	R5	3.67	64	
84	R5	6.67	61	
100	R5	1.67	74	
100	R5	1.00	74	
96	R5	2.33	74	
94	R5	3.00	74	
95	R5	3.67	74	
99	R5	1.00	74	
92	R5	3.33	64	
63	R5	7.67	57	
84	R5	2.67	71	
95	R5	3.00	74	
90	R5	2.33	71	
93	R5	4.33	74	
90	R5	4.00	61	
100	R5	2.67	74	
92	R5	2.33	74	

PROJECT: Marathon Gold Corporation - Valentine Gold Project
Pre-Feasibility Geotechnical Investigation
DEPOSIT: Leprechaun
BOREHOLE ID: VL-GT-19-04
AZIMUTH: 59.1° **DIP:** -69.3°
COORDINATES: 486885.7 E 5356271.6 N
ELEVATION: 394.4 masl

MARATHON GOLD

DATUM: UTM NAD83 z21
START: 0.0 m
END: 125.0 m
CORE SIZE: HQ3 **PAGE:** 1 of 2

TERRANE GEOSCIENCE INC.

GEOTECHNICAL CORE LOG

Terranegeoscience.com
100 - 5435 Portland Place, B3K 2Y7
Halifax, Nova Scotia, Canada

DEFINITIONS

RQD - Rock Quality Designation (%)
IRS - Intact Rock Strength (Brown, 1981)
STR - Major Structure (ie. Fault)
RMR76 - Rock Mass Rating (Bieniawski,1976)
FF/m - Number of fractures per meter
TCA - Angle relative to core axis

Depth: 0 – 124.76 m; Geotech Domain: TRJ

Lithology	Description
DYKE	Green to grey; mg to fg w/ 30-60% white pl laths, some w/ pale green ep alteration; local sections of weak to moderate L1 & S1 deformation masks original textures; below 27.20 m green-grey fg mafic dike
TNJ	Light grey; mg; rare up to 2mm rounded pale blue qtz phenocrysts
DYKE	Green-grey; fg w/ local mg sections
TNJ	Light grey; mg; rare up to 2mm rounded pale blue qtz phenocrysts
TNJ; QTP	Beige; mg; rare<=2mm rounded grey qtz qtz phenocrysts; up to 6cm QTP veins w/ weak to strong pale beige to pale peach alt halos
TNJ	Beige; mg; rare up to 2mm rounded pale blue qtz phenocrysts
DYKE	Green-grey; fg
TNJ	Beige; mg; rare up to 2mm rounded pale blue qtz phenocrysts; sporadic QT veins up to 10cm w/ lesser QTP veins up to 2cm

RQD% (RQD (%))	Rock Strength (IRS (MPa))	Fracture Frequency (FF/m)	RMR76 (RMR'76)
46	R4	8.00	47
76	R4	8.00	56
66	R4	8.67	52
38	R4	11.33	47
63	R4	10.33	49
87	R5	6.67	61
74	R5	6.67	57
88	R5	2.67	68
92	R5	6.00	64
92	R5	4.00	74
79	R5	5.67	61
69	R5	7.00	57
89	R5	4.67	61
72	R5	4.00	57
78	R5	5.33	61
95	R5	5.33	64
93	R5	4.33	74
95	R5	3.00	74
40	R3	15.33	32
70	R4	7.33	46
88	R5	5.67	61
97	R5	3.00	74
100	R5	0.67	82
97	R5	1.67	74
100	R5	0.33	87
99	R5	1.00	79
98	R5	1.33	74
100	R5	0.00	87
103	R5	1.00	79
96	R5	0.67	79
99	R5	2.33	74
100	R5	1.67	74
100	R5	2.67	74
93	R5	5.00	74
100	R5	3.00	74
100	R5	1.67	79
93	R5	2.67	74
100	R5	1.00	79
94	R5	2.00	74
99	R5	3.00	74
99	R5	1.00	74

Major Structure: Structure (TCA)

TERRANE GEOSCIENCE INC.

GEOTECHNICAL CORE LOG

Terranegeoscience.com
100 - 5435 Portland Place, B3K 2Y7
Halifax, Nova Scotia, Canada

MARATHON GOLD

PROJECT: Marathon Gold Corporation - Valentine Gold Project
Pre-Feasibility Geotechnical Investigation
DEPOSIT: Leprechaun
BOREHOLE ID: VL-GT-19-04
AZIMUTH: 59.1° **DIP:** -69.3°
COORDINATES: 486885.7 E 5356271.6 N
ELEVATION: 394.4 masl
DATUM: UTM NAD83 z21
START: 0.0 m
END: 125.0 m
CORE SIZE: HQ3 **PAGE:** 2 of 2

DEFINITIONS
RQD - Rock Quality Designation (%)
IRS - Intact Rock Strength (Brown, 1981)
STR - Major Structure (ie. Fault)
RMR76 - Rock Mass Rating (Bieniawski,1976)
FF/m - Number of fractures per meter
TCA - Angle relative to core axis

Depth	Geotech Domain	Lithology	Description	RQD%	Rock Strength	Fracture Frequency	RMR76	Major Structure
				RQD (%) 0 25 50 75 100	IRS (MPa) 0 50 100 150 200	FF/m 0 5 10 15	RMR'76 0 25 50 75 100	Structure (TCA) 0 30 60 90
125 124.76								

TERRANE GEOSCIENCE INC.

MARATHON GOLD

GEOTECHNICAL CORE LOG

Terranegeoscience.com

100 - 5435 Portland Place, B3K 2Y7

Halifax, Nova Scotia, Canada

PROJECT: Marathon Gold Corporation - Valentine Gold Project
Pre-Feasibility Geotechnical Investigation
DEPOSIT: Marathon
BOREHOLE ID: MA-GT-19-05
AZIMUTH: 128.7° **DIP:** -60.3°
COORDINATES: 492406.4 E 5360532.6 N
ELEVATION: 335.9 masl

DATUM: UTM NAD83 z21
START: 0.0 m
END: 399.5 m
CORE SIZE: HQ3

DEFINITIONS

RQD - Rock Quality Designation (%)
IRS - Intact Rock Strength (Brown, 1981)
STR - Major Structure (ie. Fault)
RMR76 - Rock Mass Rating (Bieniawski,1976)
FF/m - Number of fractures per meter
TCA - Angle relative to core axis

Depth	Geotech Domain	Lithology	Description	RQD%	Rock Strength	Fracture Frequency	RMR76	Major Structure
				RQD (%) ▭ 0 25 50 75 100	IRS (MPa) ▨ 0 50 100 150 200	FF/m 🟧 0 5 10 15	RMR'76 ▭ 0 25 50 75 100	Structure (TCA) ◆ 0 30 60 90
0		OVBN	Soil					
	MFI	DYKE	Mostly med grained with some fine grained sections Dark green, some metre scale sections that are paler green due to silicification, carb alteration or epidotization. Very localized ~ 10 cm scale rafts of QEP, often alterted to pale pink. Leading up to contact with QEP, mafic is heavily sheared and broken up. Somewhat of a gradational contact with some hints of QEP mixed with the mafic.	73	R5	7.69	57	
10				95	R5	3.00	74	
				71	R5	7.00	57	
				83	R5	6.33	61	
				91	R4	4.67	59	
20				91	R4	5.33	59	
				70	R4	8.33	52	
				71	R4	8.33	52	
30				89	R4	4.67	56	
				75	R4	5.67	52	
				32	R4	5.33	41	◆
40				45	R4	13.33	47	◆ ◆ ◆
				90	R4	3.33	66	
				85	R4	5.67	56	
				70	R4	8.33	52	
50				80	R4	7.67	56	
				81	R4	5.67	56	
				89	R4	3.67	66	
60				100	R5	3.67	74	
				96	R5	3.33	82	
				89	R5	5.67	69	
70				91	R5	5.00	72	
				99	R5	3.00	74	
				95	R5	1.67	74	
				100	R5	1.67	74	
80				95	R5	1.67	79	
				76	R5	6.33	61	
				95	R5	4.67	64	
90				91	R5	1.67	74	
				97	R5	2.67	74	
				100	R5	3.33	74	
100				94	R5	2.00	74	
				97	R5	4.00	74	
				90	R5	4.67	55	◆
110				92	R5	5.00	68	
				88	R5	3.67	65	
				93	R5	4.67	68	◆
				79	R5	6.00	55	
120				55	R4	2.67	46	◆ ◆
124.76				62	R5	6.00	57	

TERRANE GEOSCIENCE INC.

PROJECT: Marathon Gold Corporation - Valentine Gold Project
Pre-Feasibility Geotechnical Investigation
DEPOSIT: Marathon
BOREHOLE ID: MA-GT-19-05
AZIMUTH: 128.7° **DIP:** -60.3°
COORDINATES: 492406.4 E 5360532.6 N
ELEVATION: 335.9 masl

MARATHON GOLD

DATUM: UTM NAD83 z21
START: 0.0 m
END: 399.5 m
CORE SIZE: HQ3 **PAGE:** 2 of 4

GEOTECHNICAL CORE LOG

Terranegeoscience.com
100 - 5435 Portland Place, B3K 2Y7
Halifax, Nova Scotia, Canada

DEFINITIONS

RQD - Rock Quality Designation (%)
IRS - Intact Rock Strength (Brown, 1981)
STR - Major Structure (ie. Fault)
RMR76 - Rock Mass Rating (Bieniawski,1976)
FF/m - Number of fractures per meter
TCA - Angle relative to core axis

Depth: 124.76 – 244.04

Geotech Domain	Lithology	Description
MFI	DYKE	Mostly med grained with some fine grained sections Dark green, some metre scale sections that are paler green due to silicification, carb alteration or epidotization. Very localized ~ 10 cm scale rafts of QEP, often alterted to pale pink. Leading up to contact with QEP, mafic is heavily sheared and broken up. Somewhat of a gradational contact with some hints of QEP mixed with the mafic.
QEP	QEP	Med grained, green-grey, mottled chl throughout, localized decimetre scale sections of intense epidote alteration. Very pale blue qtz eyes.
QEP	DYKE	Fine grained, dark green, calc wisps and bands throughout. Minor epidote alteration throughout with localized sections exhibiting more intense alteration.
QEP	QEP	Med grained, mottled with chl and also interstitial chl veining. Pale white qtz eyes.
QEP	DYKE	Fine grained, dark green, calc wisps and bands throughout. Minor epidote alteration throughout with localized sections exhibiting more intense alteration.
QEP	QEP	Dark green-grey. Med-coarse grained qtz -- unit is equigranular in places. Mottled chl. Faint blue qtz eyes.
QEP	QAP	Olive to dark green aphanitic groundmass. Groundmass is also very siliceous with no feldspars. Coarse grained blue qtz eyes peppered throughout making up about 5%. Moderately fractured throughout

RQD%	Rock Strength	Fracture Frequency	RMR76
81	R5	5.67	61
94	R5	1.67	82
100	R5	2.33	82
100	R5	2.67	82
100	R5	3.00	82
97	R5	4.00	74
97	R5	2.67	82
81	R5	5.33	71
91	R5	5.00	74
91	R5	3.67	74
90	R5	5.00	71
94	R5	4.00	74
90	R5	3.67	71
91	R5	3.00	74
99	R5	2.67	74
99	R5	3.00	74
93	R5	3.33	74
95	R5	3.67	74
100	R5	3.00	74
90	R5	3.33	71
26	R4	12.33	41
23	R4	16.00	36
82	R5	6.00	61
83	R5	5.00	71
99	R5	2.00	74
90	R5	3.33	71
97	R5	5.67	74
92	R5	4.67	74
91	R5	4.00	74
98	R5	4.67	74
83	R5	5.67	61
100	R5	2.00	74
95	R5	3.67	74
96	R5	3.00	74
98	R5	2.67	74
98	R5	2.00	74
97	R5	1.67	74
98	R5	4.00	74
85	R5	6.00	61
94	R5	5.67	64

Major Structure — Structure (TCA)

TERRANE GEOSCIENCE INC.

GEOTECHNICAL CORE LOG

Terranegeoscience.com
100 - 5435 Portland Place, B3K 2Y7
Halifax, Nova Scotia, Canada

PROJECT: Marathon Gold Corporation - Valentine Gold Project
Pre-Feasibility Geotechnical Investigation
DEPOSIT: Marathon
BOREHOLE ID: MA-GT-19-05
AZIMUTH: 128.7° **DIP:** -60.3°
COORDINATES: 492406.4 E 5360532.6 N
ELEVATION: 335.9 masl

MARATHON GOLD

DATUM: UTM NAD83 z21
START: 0.0 m
END: 399.5 m
CORE SIZE: HQ3 **PAGE:** 3 of 4

DEFINITIONS

RQD - Rock Quality Designation (%)
IRS - Intact Rock Strength (Brown, 1981)
STR - Major Structure (ie. Fault)
RMR76 - Rock Mass Rating (Bieniawski,1976)
FF/m - Number of fractures per meter
TCA - Angle relative to core axis

Depth (m): 244.04 – 363.32 (scale marks 250, 260, 270, 280, 290, 300, 310, 320, 330, 340, 350, 360)

Geotech Domain: QEP

Lithology	Description
	Olive to dark green aphanitic groundmass. Groundmass is also very siliceous with no feldspars. Coarse grained blue qtz eyes peppered throughout making up about 5%. Moderately fractured throughout
QEP	Coarse grained pale blue qtz eyes in a med grained groundmass. Heavily chloritized with interstitial chl vein-lets and mottled chl throughout. Localized sections of epidote alteration varying in intensity and minor brecciation.
DYKE	Fine grained, dark green, calc wisps and barren qtz veins throughout. Localized (sub 1 m) sections of QEP.
QEP	Med and coarse graiend qtz eyes in a med grained ground-mass, mottled chl and interstitial chl throughout, weakly fractured. Lower margin near mafic is heavily foliated with some minor faulting.
DYKE	Fine grained, dark green, calc wisps and minor qtz veins throughout. Some diss mg cubic py throughout unit as well.
QEP	Med - coarse grained pale blue qtz eyes in a med grained groundmass. Localized sections are borderline aphanitic. Dark grey - green in color. Mottled chl and interstitial chl throughout. Serc content increasing down hole. Some burgundy alteration along fractures.
DYKE	Fine grained, dark green, calc wisps and minor qtz veins throughout. Some diss mg cubic py throughout unit as well.

RQD (%)	IRS (MPa)	FF/m	RMR'76
94	R5	5.67	64
95	R5	3.33	74
93	R5	3.33	74
97	R5	2.67	74
93	R5	4.67	74
97	R5	2.67	74
91	R5	2.67	74
99	R5	2.00	74
100	R5	1.33	82
98	R5	3.00	74
97	R5	2.00	74
88	R5	4.33	71
100	R5	3.33	74
82	R5	4.00	61
100	R5	2.00	74
97	R5	3.67	74
98	R5	2.67	82
87	R4	5.33	56
94	R4	4.67	59
92	R5	3.00	74
96	R5	3.00	74
100	R5	2.67	74
93	R5	3.67	74
78	R5	4.00	65
95	R5	2.33	74
100	R5	1.33	79
100	R5	1.00	87
100	R5	1.00	87
100	R5	1.33	79
99	R5	2.33	82
98	R5	1.00	87
98	R5	1.67	74
97	R5	2.00	79
94	R5	2.33	74
89	R5	4.67	61
95	R5	2.67	74
100	R5	1.33	79
99	R5	2.00	82
93	R5	4.67	64
92	R5	2.33	74
96	R5	2.67	74

Major Structure — Structure (TCA) (scale 30, 60, 90)

TERRANE GEOSCIENCE INC.

GEOTECHNICAL CORE LOG

Terranegeoscience.com
100 - 5435 Portland Place, B3K 2Y7
Halifax, Nova Scotia, Canada

MARATHON GOLD

PROJECT: Marathon Gold Corporation - Valentine Gold Project
Pre-Feasibility Geotechnical Investigation
DEPOSIT: Marathon
BOREHOLE ID: MA-GT-19-05
AZIMUTH: 128.7° **DIP:** -60.3°
COORDINATES: 492406.4 E 5360532.6 N
ELEVATION: 335.9 masl
DATUM: UTM NAD83 z21
START: 0.0 m
END: 399.5 m
CORE SIZE: HQ3 **PAGE:** 4 of 4

DEFINITIONS
RQD - Rock Quality Designation (%)
IRS - Intact Rock Strength (Brown, 1981)
STR - Major Structure (ie. Fault)
RMR76 - Rock Mass Rating (Bieniawski,1976)
FF/m - Number of fractures per meter
TCA - Angle relative to core axis

Depth: 363.32 – 399.5 (ticks 370, 380, 390)
Geotech Domain: QEP
Lithology: QEP
Description: Med and coarse grained qtz eyes in a med grained ground-mass. Unit is becoming significantly more silica and serc altered. Some metre scaled sections are heavily fractured - almost exhibiting qualities of the QEP-Breccia unit.

RQD (%)	IRS (MPa)	FF/m	RMR'76
96	R5	2.67	74
100	R5	2.33	74
95	R5	2.00	74
100	R5	1.67	74
87	R5	4.67	61
97	R5	3.00	74
96	R5	3.00	74
100	R5	2.67	74
100	R5	1.67	74
99	R5	1.67	74
87	R5	2.67	71
88	R5	4.00	61
95	R5	2.14	74

Major Structure: Structure (TCA)

TERRANE GEOSCIENCE INC.

MARATHON GOLD

GEOTECHNICAL CORE LOG

Terranegeoscience.com

100 - 5435 Portland Place, B3K 2Y7

Halifax, Nova Scotia, Canada

PROJECT: Marathon Gold Corporation - Valentine Gold Project
Pre-Feasibility Geotechnical Investigation

DEPOSIT: Marathon

BOREHOLE ID: MA-GT-19-06

AZIMUTH: 324.5° **DIP:** -52.5°

COORDINATES: 492596.4 E 5359895.9 N

ELEVATION: 374.7 masl

DATUM: UTM NAD83 z21

START: 0.0 m

END: 260.0 m

CORE SIZE: HQ3 **PAGE:** 1 of 3

DEFINITIONS

RQD - Rock Quality Designation (%)
IRS - Intact Rock Strength (Brown, 1981)
STR - Major Structure (ie. Fault)
RMR76 - Rock Mass Rating (Bieniawski,1976)
FF/m - Number of fractures per meter
TCA - Angle relative to core axis

Depth	Geotech Domain	Lithology	Description
0 – ~8.5		OVBN	Soil and Boulders
~8.5 – 124.76	CG	CNGL	Dark green, fine grained chloritic matrix with polymodal inclusions exhibiitng a wide range of formation. Strongly foliated throughout with some sectioned of moderate foliation. Sporadic 1-2 cm wide QC veins from 86.20 to 87.60 m. Clast size increasing down hole and are very attenuated. Chlorite rich and lesser serc. 8 cm Patchy QC vein from 210.20 to 210.28 m.

RQD (%)	Rock Strength IRS (MPa)	Fracture Frequency FF/m	RMR'76
44	R5	4.00	52
97	R5	3.33	74
87	R5	5.00	61
87	R5	3.67	61
83	R5	6.00	61
90	R5	4.33	71
69	R5	5.00	57
96	R5	3.67	74
80	R5	5.00	61
94	R5	4.00	64
96	R5	2.67	74
100	R5	2.33	74
92	R5	5.00	64
88	R5	3.33	58
90	R5	3.67	61
80	R5	6.67	61
90	R5	3.33	61
92	R5	3.33	64
93	R5	5.33	64
93	R5	4.33	74
84	R5	5.67	61
89	R5	4.67	61
98	R5	1.33	74
92	R5	4.67	64
92	R5	4.33	64
95	R5	4.00	64
59	R5	6.00	51
90	R5	4.67	61
90	R5	5.00	61
100	R5	2.33	74
100	R5	3.00	74
100	R5	2.67	74
97	R5	3.00	74
82	R5	5.33	61
92	R5	5.67	64
97	R5	2.33	74
98	R5	3.67	74
100	R5	2.00	74
100	R5	3.33	74

Major Structure: Structure (TCA)

TERRANE GEOSCIENCE INC.

Terranegeoscience.com

100 - 5435 Portland Place, B3K 2Y7

Halifax, Nova Scotia, Canada

GEOTECHNICAL CORE LOG

MARATHON GOLD

PROJECT: Marathon Gold Corporation - Valentine Gold Project
Pre-Feasibility Geotechnical Investigation

DEPOSIT: Marathon

BOREHOLE ID: MA-GT-19-06

AZIMUTH: 324.5° **DIP:** -52.5°

COORDINATES: 492596.4 E 5359895.9 N

ELEVATION: 374.7 masl

DATUM: UTM NAD83 z21

START: 0.0 m

END: 260.0 m

CORE SIZE: HQ3 **PAGE:** 2 of 3

DEFINITIONS

RQD - Rock Quality Designation (%)
IRS - Intact Rock Strength (Brown, 1981)
STR - Major Structure (ie. Fault)
RMR76 - Rock Mass Rating (Bieniawski,1976)
FF/m - Number of fractures per meter
TCA - Angle relative to core axis

Depth: 124.76 – 244.04 m (scale ticks 130, 140, 150, 160, 170, 180, 190, 200, 210, 220, 230, 240)

Geotech Domain: CG

Lithology: CNGL

Description: Dark green, fine grained chloritic matrix with polymodal inclusions exhibiitng a wide range of formation. Strongly foliated throughout with some sectioned of moderate foliation. Sporadic 1-2 cm wide QC veins from 86.20 to 87.60 m. Clast size increasing down hole and are very attenuated. Chlorite rich and lesser serc. 8 cm Patchy QC vein from 210.20 to 210.28 m.

Major Structure: Structure (TCA) (scale 30, 60, 90)

RQD% (RQD (%))	Rock Strength (IRS (MPa))	Fracture Frequency (FF/m)	RMR76 (RMR'76)
95	R5	3.33	74
93	R5	4.00	74
97	R5	2.67	74
100	R5	2.67	74
99	R5	4.00	64
97	R5	3.00	74
99	R5	3.67	74
78	R4	5.67	50
86	R5	5.00	61
95	R5	4.00	64
81	R5	7.33	55
89	R5	3.67	61
62	R4	7.00	52
91	R5	5.00	64
95	R5	4.33	64
64	R5	8.67	57
76	R5	6.00	61
99	R5	1.33	74
97	R5	3.67	64
99	R5	3.00	64
81	R5	5.67	61
80	R5	2.67	71
90	R5	3.67	61
100	R5	1.67	74
86	R5	4.67	61
99	R5	2.67	74
82	R5	4.33	61
98	R5	1.67	74
72	R5	7.00	57
99	R5	2.00	74
100	R5	1.67	74
100	R5	3.67	64
94	R5	2.67	74
97	R5	2.33	74
98	R5	2.33	74
95	R5	2.67	74
85	R5	5.00	61
96	R5	4.67	64
98	R5	2.33	74
100	R5	2.00	74

TERRANE GEOSCIENCE INC.

GEOTECHNICAL CORE LOG

Terranegeoscience.com

100 - 5435 Portland Place, B3K 2Y7

Halifax, Nova Scotia, Canada

MARATHON GOLD

PROJECT: Marathon Gold Corporation - Valentine Gold Project
Pre-Feasibility Geotechnical Investigation

DEPOSIT: Marathon

BOREHOLE ID: MA-GT-19-06

AZIMUTH: 324.5° **DIP:** -52.5°

COORDINATES: 492596.4 E 5359895.9 N

ELEVATION: 374.7 masl

DATUM: UTM NAD83 z21

START: 0.0 m

END: 260.0 m

CORE SIZE: HQ3 **PAGE:** 3 of 3

DEFINITIONS

RQD - Rock Quality Designation (%)
IRS - Intact Rock Strength (Brown, 1981)
STR - Major Structure (ie. Fault)
RMR76 - Rock Mass Rating (Bieniawski,1976)
FF/m - Number of fractures per meter
TCA - Angle relative to core axis

Depth	Geotech Domain	Lithology	Description	RQD% (RQD (%))	Rock Strength (IRS (MPa))	Fracture Frequency (FF/m)	RMR76 (RMR'76)	Major Structure (Structure (TCA))
244.04	CG	CNGL	Dark green, fine grained chloritic matrix with polymodal inclusions exhibiitng a wide range of formation. Strongly foliated throughout with some sectioned of moderate foliation. Sporadic 1-2 cm wide QC veins from 86.20 to 87.60 m. Clast size increasing down hole and are very attenuated. Chlorite rich and lesser serc. 8 cm Patchy QC vein from 210.20 to 210.28 m.	100	R5	2.00	74	
				93	R5	2.33	64	
250		DYKE	Dark green, fine-grained, w/ sporadic irreg QC veins up to 2 cm wide.	98	R5	1.00	82	
				100	R5	1.33	79	
		CNGL	Dark green, fine grained chloritic matrix with polymodal inclusions exhibiitng a wide range of formation. Strongly foliated throughout with some sectioned of moderate foliation.	97	R5	3.00	74	
260				99	R5	1.33	74	

Axis scales: RQD (%) 0 25 50 75 100; IRS (MPa) 0 50 100 150 200; FF/m 0 5 10 15; RMR'76 0 25 50 75 100; Structure (TCA) 0 30 60 90

TERRANE GEOSCIENCE INC.

GEOTECHNICAL CORE LOG

Terranegeoscience.com

100 - 5435 Portland Place, B3K 2Y7

Halifax, Nova Scotia, Canada

PROJECT: Marathon Gold Corporation - Valentine Gold Project
Pre-Feasibility Geotechnical Investigation

DEPOSIT: Marathon

BOREHOLE ID: MA-GT-19-07

AZIMUTH: 218.7° **DIP:** -51.8°

COORDINATES: 492935.7 E 5360739.7 N

ELEVATION: 338.5 masl

MARATHON GOLD

DATUM: UTM NAD83 z21

START: 0.0 m

END: 401.0 m

CORE SIZE: HQ3

DEFINITIONS

RQD - Rock Quality Designation (%)
IRS - Intact Rock Strength (Brown, 1981)
STR - Major Structure (ie. Fault)
RMR76 - Rock Mass Rating (Bieniawski,1976)
FF/m - Number of fractures per meter
TCA - Angle relative to core axis

Depth: 0, 10, 20, 30, 40, 50, 60, 70, 80, 90, 100, 110, 120, 124.76

Geotech Domain: QEP

Lithology	Description
OVBN	Soil and Boulders.
QEP	Green to grey and tan fg to mg groundmass w/ up to 3mm rounded blue qtz phenocrysts; sporadic up to 3cm QT and QTP vns
DYKE	Green-grey to white; fg to mg w/ 50-70% up to 3mm white anhedral to subhedral pl
QAP	Pale green to grey; aphanitic w/ up to 2mm rounded blue qtz phenocrysts; some local sections of minor early stage breccias w/ dark green-grey chl rich matrix.
QEP	Pale to dark green fg to mg groundmass w/ up to 3mm rounded blue qtz phenocrysts
QAP	dark green; aphanitic groundmass with blue qtz eyes and feldspar phenos up to 3mm; qtz-chl veins up to 1cm @ mod to high angle to CA
DYKE	dark green; fg; weak to mod foliation @ 30-60 deg to CA; thin calc bands along foliation
QAP	grey-green to beige; aphanitic ground mass with local qtz eyes and feldspar phenos; mod to strong fracturing with chl and local serc infilling
QEP	dark green; mg with cg qtz eyes; strong chl alt; weak to mod fracturing; chl infilling fractures
QAP	dark grey-green; aphanitic groundmass with sporadic qtz eyes up to 4mm; mod fracturing; local diss py
QEP	dark grey-green; mg with cg qtz eyes; weak to mod fracturing; chl and serc alteration
QAP	dark green; aphanitic with sporadic qtz eyes up to 2mm; weak to mod fracturing; local qtz-calc veining up to 1cm thick.
QEP	dark green; mg with cg qtz eyes; overprinted by strong chl and a mix of irregular qtz-calc-chl+/-trml veining up to 2cm thick from 115-117m with irregular, anastomosing shear sub-para to CA; lens of AQP with irregular QP veining from 132.80-133.20, 3% pyrite.

RQD (%)	IRS (MPa)	FF/m	RMR'76	Structure (TCA)
75	R5	4.38	57	
80	R5	5.33	61	
89	R5	3.67	61	
82	R5	6.00	61	
80	R5	7.67	61	
37	R5	9.00	46	
58	R5	8.00	51	
42	R5	11.00	46	
73	R5	7.00	57	
81	R5	5.33	61	
97	R5	4.00	64	
90	R5	4.67	61	
75	R5	7.00	57	
89	R5	6.67	61	
91	R5	4.67	64	
80	R5	4.33	61	
88	R5	6.00	61	
97	R5	2.33	74	
100	R5	2.33	74	
84	R5	4.33	61	
83	R5	4.67	61	
93	R5	3.00	74	
99	R5	3.00	74	
97	R5	2.67	74	
99	R5	2.67	74	
94	R5	4.00	74	
100	R5	2.67	74	
99	R5	2.33	74	
101	R5	1.33	87	
92	R5	2.67	79	
95	R5	4.00	74	
94	R5	2.33	74	
93	R5	3.00	74	
100	R5	2.00	79	
78	R5	3.00	71	
95	R5	2.33	74	
93	R5	3.33	74	
90	R5	4.67	61	
100	R5	2.33	74	

TERRANE GEOSCIENCE INC.

GEOTECHNICAL CORE LOG

Terranegeoscience.com

100 - 5435 Portland Place, B3K 2Y7

Halifax, Nova Scotia, Canada

PROJECT: Marathon Gold Corporation - Valentine Gold Project Pre-Feasibility Geotechnical Investigation
DEPOSIT: Marathon
BOREHOLE ID: MA-GT-19-07
AZIMUTH: 218.7° **DIP:** -51.8°
COORDINATES: 492935.7 E 5360739.7 N
ELEVATION: 338.5 masl

MARATHON GOLD

DATUM: UTM NAD83 z21
START: 0.0 m
END: 401.0 m
CORE SIZE: HQ3 **PAGE:** 2 of 4

DEFINITIONS

RQD - Rock Quality Designation (%)
IRS - Intact Rock Strength (Brown, 1981)
STR - Major Structure (ie. Fault)
RMR76 - Rock Mass Rating (Bieniawski,1976)
FF/m - Number of fractures per meter
TCA - Angle relative to core axis

Depth: 124.76 – 244.04 m. Geotech Domain: QEP

Lithology	Description
QEP	dark green; mg with cg qtz eyes; overprinted by strong chl and a mix of irregular qtz-calc-chl+/-trml veining up to 2cm thick from 115-117m with irregular, anastomosing shear sub-para to CA; lens of AQP with irregular QP veining from 132.80-133.20, 3% pyrite.
DYKE	dark green; fg with plag phenos up to 2mm; sporadic qtz-calc veining up to 1cm thick with patchy epidote
QAP	grey-green; aphanitic with 60-70% plag and qtz phenos ranging from 1-3mm; strongly silicified
QEP	grey-green; mg with cg qtz eyes; patchy chl and serc alt; sporadic banded trml veining with qtz-py infilling from 185.20-189.80, up to 5mm thick; <2% pyrite; low to mod angle to CA, <1% veining; <1% QTP veining up to 1cm thick from 192.42-195.18, <1% veining, 1-3% pyrite; local AQP dikes; 0.5-1% diss and stringers of py from 213.5-216.0
DYKE; QTP	dark green; mg with no qtz eyes; plag phenos up to 2mm; strong chl alt; largest vein at gabbro/QEP contact veining up to 45cm thick with a variety of qtz-trml-chl-calc; banded trml locally along vein margins; veins in lower section have patchy plag; pink alt halos up to 5cm on thicker veins; trace up to 2% py in veins.
QEP	dark green; mg with cg qtz eyes up to 4mm; strong chl and mod serc alt; serc conc along fractures
QAP	dark grey-green; aphanitic; mod to strong irregular fracturing; wispy calc and chl infilling fractures; trace to 0.5% py throughout; local rafts of QEP up to 10cm.
QEP	dark to light grey-green; mg with cg qtz eyes up to 5mm; patchy grey-purple silica alt; mafic dikes at low to mod angle to CA with wispy calc; occasional trml bands sub-para to CA.

RQD (%)	IRS (MPa)	FF/m	RMR'76
93	R5	3.00	74
100	R5	2.00	82
91	R5	3.00	74
100	R5	1.67	82
96	R5	4.00	74
76	R5	5.00	61
93	R5	4.33	64
100	R5	1.67	79
92	R5	4.33	64
88	R5	6.67	71
96	R5	3.00	64
84	R5	6.67	61
95	R5	4.00	64
100	R5	3.67	74
90	R5	3.00	71
94	R5	3.00	74
96	R5	3.00	74
94	R5	4.00	74
91	R5	5.33	74
83	R5	6.00	61
90	R4	4.00	56
93	R4	3.67	59
100	R5	2.67	74
88	R5	3.67	71
94	R5	3.00	74
99	R5	2.33	74
91	R4	3.33	69
80	R4	2.33	66
100	R5	3.00	74
75	R5	7.67	57
99	R5	3.00	74
93	R5	2.33	74
95	R5	4.00	64
100	R5	1.00	79
100	R5	2.33	74
92	R5	0.67	79
92	R5	3.00	74
96	R5	2.00	74
98	R5	2.00	74
100	R5	1.67	74

Major Structure: Structure (TCA)

TERRANE GEOSCIENCE INC.

MARATHON GOLD

GEOTECHNICAL CORE LOG

Terranegeoscience.com

100 - 5435 Portland Place, B3K 2Y7

Halifax, Nova Scotia, Canada

PROJECT: Marathon Gold Corporation - Valentine Gold Project
Pre-Feasibility Geotechnical Investigation
DEPOSIT: Marathon
BOREHOLE ID: MA-GT-19-07
AZIMUTH: 218.7° **DIP:** -51.8°
COORDINATES: 492935.7 E 5360739.7 N
ELEVATION: 338.5 masl

DATUM: UTM NAD83 z21
START: 0.0 m
END: 401.0 m
CORE SIZE: HQ3 **PAGE:** 3 of 4

DEFINITIONS

RQD - Rock Quality Designation (%)
IRS - Intact Rock Strength (Brown, 1981)
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FF/m - Number of fractures per meter
TCA - Angle relative to core axis

Depth: 244.04 – 363.32 (scale 250, 260, 270, 280, 290, 300, 310, 320, 330, 340, 350, 360)

Geotech Domain: QEP

Lithology	Description
QEP	dark to light grey-green; mg with cg qtz eyes up to 5mm; patchy grey-purple silica alt; mafic dikes at low to mod angle to CA with wispy calc; occasional trml bands sub-para to CA.
DYKE	dark green; fg; mod foliation @ 10-30 deg to CA; wispy to patchy calc generally along foliation
QEP	grey-green; mg with cg qtz eyes; local patchy peach alt; qtz-calc-chl veining up to 2cm thick @ low to mod angles to CA; local mafic dikes.
QEP; QTP	Med to coarse grained, mottled chl throughout & peppered with fg hem. Dark grey - greenish with weak to mod hem staining.
QEP	Mg to coarse grained grey-green QEP w/pale blue qtz eyes up to 5mm, peppered w/white feldspars up to 2mm. Patchy background chl blebs and veinlets. A few QTP veins w/mod peach alt halos conc near bottom of unit.
DYKE	Fg dark green chloritic mafic dike w/pervasive calc wisps sheared at 35 deg to CA. Small sections of QEP.
QEP	Mg to coarse grained grey-green QEP w/pale blue qtz eyes up to 5mm, peppered w/white feldspars up to 2mm. Patchy background chl blebs and veinlets.
DYKE	Fg dark green chloritic mafic dike w/pervasive calc wisps sheared at 35 deg to CA. Small sections of QEP.
QEP	Mg to coarse grained grey-green QEP w/pale blue qtz eyes up to 5mm, peppered w/white feldspars up to 2mm. Patchy background chl blebs and veinlets. Sporadic QTP veins.
QEP; QTP	Mostly med grained with pale blue qtz eyes although equigranular in places. Most veins exhibit pale pink alt haloes.
QEP	Pale grey-salmon colored, some localized sections of AQP. Mottled chl throughout, hem staining along fractures - increasing in intensity downhole.
DYKE	Fine grained, very dark green, moderately foliated, thin wisps of calc throughout.
QEP; QTP	Pervasive pale pink alt halo throughout most of unit, localized chloritic sections, med to coarse grained. Pale blue qtz eyes throughout.

RQD (%)	IRS (MPa)	FF/m	RMR'76
100	R5	1.67	74
100	R5	2.33	74
100	R5	1.00	87
96	R5	3.33	74
90	R5	4.33	71
87	R5	4.67	61
90	R5	2.67	71
97	R5	3.00	74
98	R5	1.33	79
90	R5	4.00	71
94	R5	2.67	74
97	R5	3.67	74
96	R5	1.00	79
100	R5	0.67	87
93	R5	2.33	74
100	R5	1.00	87
99	R5	1.33	87
100	R5	1.00	87
95	R5	3.00	74
93	R5	3.00	74
100	R5	2.33	74
97	R5	2.00	74
100	R5	2.67	82
100	R5	2.00	74
100	R5	1.67	74
85	R5	5.00	69
97	R5	1.00	82
100	R5	2.33	74
100	R5	1.67	74
96	R5	1.33	79
100	R5	1.33	79
89	R5	4.00	61
88	R5	3.00	71
94	R4	1.67	69
47	R4	18.33	35
80	R5	4.00	65
85	R5	4.67	61
83	R4	5.67	56
97	R5	3.00	64
100	R5	2.67	74
95	R5	2.33	74

Major Structure: Structure (TCA) (scale 0, 30, 60, 90)

TERRANE GEOSCIENCE INC.

GEOTECHNICAL CORE LOG

Terranegeoscience.com

100 - 5435 Portland Place, B3K 2Y7

Halifax, Nova Scotia, Canada

PROJECT: Marathon Gold Corporation - Valentine Gold Project
Pre-Feasibility Geotechnical Investigation

MARATHON GOLD

DEPOSIT: Marathon
BOREHOLE ID: MA-GT-19-07
AZIMUTH: 218.7° **DIP:** -51.8°
COORDINATES: 492935.7 E 5360739.7 N
ELEVATION: 338.5 masl

DATUM: UTM NAD83 z21
START: 0.0 m
END: 401.0 m
CORE SIZE: HQ3 **PAGE:** 4 of 4

DEFINITIONS

RQD - Rock Quality Designation (%)
IRS - Intact Rock Strength (Brown, 1981)
STR - Major Structure (ie. Fault)
RMR76 - Rock Mass Rating (Bieniawski,1976)
FF/m - Number of fractures per meter
TCA - Angle relative to core axis

Depth	Geotech Domain	Lithology	Description
363.32	QEP	QEP; QTP	Pervasive pale pink alt halo throughout most of unit, localized chloritic sections, med to coarse grained. Pale blue qtz eyes throughout.
370		QEP	Med to coarse grained, dark grey-green, mottled with chlorite. Some sporadic QTP veining.
380		QEP; QTP	Med to coarse grained, dark grey-green, mottled with chlorite.
		QEP	Med to coarse grained, dark grey-green, mottled with chlorite. Some sporadic QTP veining.
390		QEP; QTP	Med to coarse grained, dark green - grey, mottled chl throughout, interstial chl veining as well. Coarse grained pale blue qtz eyes.
		QEP	Mg to coarse grained grey-green QEP w/pale blue qtz eyes up to 5mm, peppered w/white feldspars up to 2mm. Patchy background chl blebs and veinlets. Sporadic QTP veins.
400 / 401		QEP; QTP	Med grained, almost equigranular, pervasive bleached and pink alteration throughout most of unit.

RQD% (RQD (%), 0–100)	Rock Strength (IRS (MPa), 0–200)	Fracture Frequency (FF/m, 0–15)	RMR76 (RMR'76, 0–100)	Major Structure (Structure (TCA), 0–90)
95	R5	2.33	74	
99	R5	2.67	74	
92	R5	4.67	64	
99	R5	2.67	74	
100	R5	2.67	74	
98	R5	1.67	74	
97	R5	2.67	74	
96	R5	3.67	74	
98	R5	2.67	74	
100	R5	3.00	74	
98	R5	3.00	74	
94	R5	4.67	74	
94	R4	2.00	69	

GEOTECHNICAL LINE MAPPING SHEET

	x	y	z
Start Line:	486650	5356334	
End Line:	486632	5356375	
Datum:	NAD83 zone 21		
Projection:			

Project: Marathon Gold Corp. Pre-Feasibility Geotechnical Investigation

Deposit: Leprechaun

Pg. 1 of 2

				ORIENTATION				
STN	Rock Type	Type	Weathering	Strike[1.]	Dip	Spacing (m)[2.]	Persist.	Roughness
0.0	TRJ	FOL'N	W2	225	60	0.10	0.30	N/A
1.1	TRJ	FOL'N	W2	242	70	2.00	0.70	N/A
1.6	TRJ	J1	W2	122	88		0.60	N/A
2.2	TRJ	FOL'N	W2	244	55	1.10	2.50	N/A
3.2	TRJ	FOL'N	W2	236	61	0.50	0.50	N/A
5.8	TRJ	J2	W2	300	72		0.10	N/A
5.9	TRJ		W2	030	80		0.10	N/A
6.0	TRJ	FOL'N	W1	228	63	0.20	0.20	N/A
6.2	TRJ	FOL'N	W1	244	78	1.10	0.30	N/A
8.2	TRJ	FOL'N	W1	238	87	0.70	0.90	N/A
9.7	TRJ	FOL'N	W2	234	79	1.30	1.00	N/A
10.1	TRJ	J1	W2	148	82		0.25	N/A
10.6	TRJ	FOL'N	W2	246	63	2.00	0.70	N/A
12.8	TRJ	FOL'N	W1	222	60	2.00	3.00	N/A
15.2	SHR MD	FOL'N	W3	229	80	0.50	3.50	N/A
15.5	S - MD	FOL'N	W3	224	75	0.20	2.70	N/A
17.8	S - MD	FOL'N	W3	244	51	2.00	3.10	N/A
21.1	S - MD	FOL'N	W3	246	66	0.50	1.80	N/A
22.3	TRJ		W2	050	89	0.25	0.70	N/A
22.4	TRJ	FOL'N	W2	248	71	0.30	3.20	N/A
23.8	S - MD	FOL'N	W3	242	68	0.05	3.10	N/A
25.7	TRJ		W1	030	83		1.20	N/A
28.1	TRJ	FOL'N	W2	248	79	0.10	0.30	N/A
36.2	S - MD	FOL'N	W3	254	60	0.15	2.30	N/A
37.3	TRJ	FOL'N	W1	243	52	0.30	0.20	N/A
37.5	TRJ		W1	032	82		0.15	N/A
40.7	TRJ	FOL'N	W1	248	72	1.50	0.30	N/A
41.8	S - MD	FOL'N	W3	241	55	0.05	2.70	N/A
42.9	TRJ	FOL'N	W1	238	60	0.50	0.50	N/A
43.5	TRJ		W1	033	88			N/A

Weathering

W1 - No visible sign of weathering

W2 - Partial (<5%) staining or discoloration of rock surface, usually by limonite. No effect on rock strength

W3 - Staining or discoloration extends throughout rock. Original rock color no longer visible, slightly affects strength

W4 - Limonite or bleaching affects the whole rock with signs of chemical or physical decompision, strength affected

W5 - Rock is almost completely decomposed to soil.

Roughness - Large Scale Roughness (1 m or greater)

Planar (Pl) - < 1.0 cm of deflection over 1 m,

Planar to Wavy ~1.5 cm - 2.0 cm deflection over 1 m,

Wavy > 2.0 cm deflection over 1 m.

NOTES:

1. Right Hand Rule
2. Spacing to next closest discontinuity in same family
3. Generally want atleast 40 measurements per line

GEOTECHNICAL LINE MAPPING SHEET

	x	y	z
Start Line:	486650	5356334	
End Line:	486632	5356375	
Datum:	NAD83 zone 21		
Projection:			

Project: Marathon Gold Corp. Pre-Feasibility Geotechnical Investigation

Deposit: Leprechaun

Pg.__2__ of __2__

				ORIENTATION				
STN	Rock Type	Type	Weathering	Strike[1.]	Dip	Spacing[2.]	Persist.	Roughness
45.9	TRJ	FOL'N	W1	242	67	0.5	1.3	N/A

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GEOTECHNICAL LINE MAPPING SHEET

TERRANE GEOSCIENCE INC.

	x	y	z
Start Line:	493164	5360663	
End Line:	493199	5360651	
Datum:	NAD83 zone 21		
Projection:			

Project: Marathon Gold Corp. Pre-Feasibility Geotechnical Investigation

Pg. 1 of 2

Deposit: Marathon

				ORIENTATION				
STN	Rock Type	Type	Weathering	Strike[1.]	Dip	Spacing (m)[2.]	Persist.	Roughness
0.0	Mafic int.	J1	W2	300	58	0.10	3.00	N/A
0.5	Mafic int.	J2	W2	288	59	0.07	1.50	N/A
1.3	Mafic int.	J3	W2	285	60	0.03	0.20	N/A
1.9	Mafic int.	J4	W2	297	61	0.07	0.20	N/A
2.3	Mafic int.	FOL'N	W1	230	62	0.20	0.75	N/A
2.4	Mafic int.	J2	W1	158	80	0.25	0.60	N/A
3.2	Dyke	J1	W2	291	60	1.50	5.00	N/A
3.4	Mafic int.	J3	W2	108	44		17.00	N/A
4.8	Mafic int.	J1	W1	290	72	0.50	4.00	N/A
5.2	Mafic int.	J4	W2	038	68	0.25	0.60	N/A
6.5		J4	W2	035	72	1.00	0.25	N/A
7.2	QTP		W2	080	71	0.40		N/A
9.3		QTV	W1	085	31		15.00	N/A
10.7		FOL'N	W2	218	80		1.50	N/A
12.5			W1	215	87	0.5	3.50	N/A
12.6	QEP	J1	W2	290	64	2.50	1.70	N/A
12.0		FOL'N	W2	226	82	0.20	5.50	N/A
15.7		FOL'N	W2	203	71		1.20	N/A
15.8	QTP		W1	108	31	0.40	3.20	N/A
16.9		J1/FOL'N	W2	280	69		1.80	N/A
17.2		J1	W1	272	57		0.30	N/A
17.6	Dyke	CT	W1	321	62	0.30	6.30	N/A
19.9	QEP		W2	190	40		2.20	N/A
21.0	QEP sheared	FOL'N	W2	228	75	1.20	2.70	N/A
21.9	sheared QEP	FLT	W2	002	51		2.50	N/A
24.1	QEP	FOL'N	W1	218	71	0.30	8.00	N/A
26.6	QEP	FOL'N	W1	232	72	1.00	8.00	N/A
26.9		FLT	W2	282	39		1.50	N/A
29.0	QEP	FOL'N	W1	223	87	0.70	8.00	N/A
29.6	QEP	FOL'N	W1	202	64	0.30	3.00	N/A

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Wavy > 2.0 cm deflection over 1 m.

NOTES:

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GEOTECHNICAL LINE MAPPING SHEET

TERRANE GEOSCIENCE INC.

	x	y	z
Start Line:	493164	5360663	
End Line:	493199	5360651	
Datum:	NAD83 zone 21		
Projection:			

Project: Marathon Gold Corp. Pre-Feasibility Geotechnical Investigation

Pg. 2 of 2

Deposit: Marathon

				ORIENTATION				
STN	Rock Type	Type	Weathering	Strike[1.]	Dip	Spacing (m)[2.]	Persist.	Roughness
30.9	Dyke	CT	W2	138	80		4.40	N/A
32.3	QEP	FOL'N	W1	232	52	1.00	8.00	N/A
33.7	QEP	S2	W3	218	78	0.03	8.00	N/A
33.8	QEP	S2	W3	234	76	0.03	8.00	N/A
34.1		S2	W3	217	72	0.03	8.00	N/A
34.3		SZ	W3	212	78	0.03	8.00	N/A
34.7		SZ	W3	232	80	0.03	8.00	N/A
35.0	QV	S2	W3	225	69	0.03	8.00	N/A
35.1	QEP	FOL'N	W1	228	72	1.00	8.00	N/A
36.6		J	W2	110	45		3.50	N/A
37.1	S2 with dyke	S2	W3	227	68	0.05	6.30	N/A
37.7	S2 with dyke	SZ	W3	224	70	0.05	6.00	N/A
37.8	S2 with dyke	S2	W3	215	73	0.05	6.00	N/A
38.9	S2 with dyke	S2	W2	226	78	0.05	6.00	N/A
40.1	Dyke/QEP CT	FOL'N	W1	224	88	0.30	8.00	N/A
42.1	QEP		W1	208	89	0.70	1.00	N/A
45.0	QEP	J	W1	119	32	2.00	5.00	N/A
								N/A
								N/A
								N/A
								N/A
								N/A
								N/A
								N/A
								N/A
								N/A
								N/A
								N/A
								N/A
								N/A

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3. Generally want atleast 40 measurements per line

Natural Resources Canada
CanmetMINING

555 Booth Street
Ottawa, Canada K1A 0G1

Ressources naturelles Canada
CanmetMINES

555, rue Booth
Ottawa, Canada K1A 0G1

CanmetMINING File Number: P-000924.002

November 21, 2019

Tony Gilman, M.Sc., P.Eng.
Terrane Geoscience Inc.
100-5435 Portland Place
Halifax, Nova Scotia

RE: Geomechanical Lab Testing of Rock Core, Marathon Gold Project

Dear Mr. Gilman:

The following letter report briefly summarizes the methodology and results of the geomechanical testing program completed on behalf of Terrane Geoscience Inc. (TGI) at CanmetMINING's Rock Mechanics Laboratory in Ottawa, Ontario. This work has been completed under CanmetMINING Project No. P-000924.002, in accordance with the agreed upon scope of work summarized in the CanmetMINING Service Offer, dated July 29, 2019.

The geomechanical testing program described herein included sample preparation; measurement of intact rock physical properties; and, determination of mechanical properties of the rock core specimens by uniaxial compression strength (UCS), triaxial compression strength testing (TCS), Brazilian tensile strength (BTS) testing, and direct shear strength (DSS) testing. All work was completed in accordance with internal standard operating procedures and applicable ASTM Standards. Key personnel involved in this testing program include the following:

- Steve Gaines, Rock Mechanics Engineer – *Project Lead*
- Ted Anderson, Senior Technologist – *Laboratory testing*
- Gilles Brisson, Technician – *Sample preparation and dimensioning*

Core samples were supplied by TGI and received at the CanmetMINING lab on August 29, 2019. Samples were received wrapped in plastic film, with the proposed geomechanical test identified in marker on the core. Approximately 0.3 m of core was provided for each test, therefore the provided material was sub-sampled to obtain the required sample dimensions. All effort was made to sub-sample representative sections of the provided core material.

1.0 Sample Preparation, Physical Properties and Moisture Content

Samples were prepared in the lab in accordance with internal Standard Operating Procedure (SOP) T-2121 and ASTM D4345-08. Fresh water was used for cutting and end preparation. Ultrasonic pulse velocities (P- and S-wave) and dynamic elastic constants of each UCS sample were determined prior to testing following ASTM D2845-08. It should be noted that this ASTM has been withdrawn without a replacement; however, this method is still considered valid for measurement of pulse velocities and calculation of the dynamic elastic constants.

Sample specifications and physical properties, including dynamic elastic constants, are summarized in Table A.1, Attachment A.

2.0 Compression Strength Testing

A total of 14 uniaxial (unconfined) and eight triaxial (confined) tests were conducted on rock core samples to determine strength and elastic properties (Young's modulus and Poisson's ratio). Samples were placed in heat shrink tubing and instrumented with mechanical gauges, which consisted of three linear variable displacement transducers (LVDTs) to measure axial displacement and a MTS chain extensometer to measure radial displacement.

Tests were conducted in accordance with internal standard operating procedures (SOP-T 2122) and standard test methods, specifically, ASTM D7012-14. Testing was completed using the servo-controlled MTS 815 loading frame at a constant axial displacement rate of 0.0008 mm/s. Triaxial compression tests were completed at confinement levels of 3 or 7 MPa, and the same axial displacement rate.

Compression strength test results are summarized in Table A.2, Attachment A. Stress-strain plots are included with detailed test data in Attachment B. Photographs of specimens, before and after failure, are included in Attachment C.

3.0 Brazilian Test (Splitting Tensile Strength Test)

Splitting tensile strength tests, also referred to as the Brazilian test or indirect tensile strength test, were conducted on eight specimens in accordance with internal standard procedures (SOP-T 2104) and standard test method ASTM D3967-16. Samples were tested using the MTS 815 loading frame at a constant load rate of 0.15 kN/s.

Brazilian tensile strength test results are summarized in Table A.3, Attachment B. Photographs of failed specimens are included in Attachment D.

4.0 Direct Shear Strength Testing

Direct shear strength tests were conducted on nine specimens, each a natural discontinuity. Constant normal load/force tests were conducted in general accordance with ASTM D5607-16 and International Society for Rock Mechanics (ISRM) suggested methods (Muralha *et al.*, 2014). Each test was completed following the multi-stage test procedure, consisting of application of an increasing target normal pressures, with repositioning of the discontinuity back to its original position prior to each successive stage.

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The direct shear testing machine consists of a lower steel box on rollers, free to move in the horizontal direction and an upper box fixed in the horizontal direction, but allowed to move in the vertical direction. Two LVDTs measure normal displacement, while a linear potentiometer measures horizontal (shear) displacement. Shear and normal load/stress are measured by one load cell in each orientation. Normal load and shear displacement are computer controlled, with data recorded at a frequency of 1 Hz.

The upper and lower halves of the specimen were encapsulated in their respective steel box using hydrostone. Once set and cured, testing commenced with application of a predetermined normal load applied to the upper box to reach the target pressure (accounting for the weight of load cell, shear box and sample). With the target load reached and normal displacement stable, shear displacement (i.e. horizontal displacement of the lower box) was initiated at a rate of between 0.3 to 0.5 mm/min, remaining constant during the test. Shearing continued for a minimum of approximately 5% of the sample length, or until an observed residual shear strength was observed.

Results of the direct shear strength testing are summarized in Table A.4, Attachment A, with complete test results provided in Attachment E.

5.0 Closure

The report will be held confidential for a period of one year. At the end of the confidentiality period all documentation, including test results, will automatically be declassified unless otherwise specified.

Should you have any questions regarding the data report and/or the work carried out, please do not hesitate to contact me at 613-947-2170, or email at steven.gaines@canada.ca.

Regards,

Steven Gaines, M.A.Sc., P.Eng., P.Geo.
Rock Mechanics Engineer
CanmetMINING, Transformative Technologies and Specialized Services

Attachments (5):

Attachment A – Summary of Test Results
Attachment B – Detailed Compression Test Data
Attachment C – Photographs of UCS and TCS Test Specimens
Attachment D – Photographs of Brazilian Test Specimens
Attachment E – Direct Shear Test Results

CC. Contracts Officer (CMIN-BA)

References (ASTM and ISRM Test Methods used during the test program)**:**

ASTM D2845-08, Standard Test Method for Laboratory Determination of Pulse Velocities and Ultrasonic Elastic Constants of Rock, ASTM International, West Conshohocken, PA (USA)

ASTM D3967-16, Standard Test Method for Splitting Tensile Strength of Intact Rock Core Specimens, ASTM International, West Conshohocken, PA (USA)

ASTM D4543-08e1, Standard Practices for Preparing Rock Core as Cylindrical Test Specimens and Verifying Conformance to Dimensional and Shape Tolerances, ASTM International, West Conshohocken, PA (USA)

ASTM D5607-16, Standard Test Methods for Performing Laboratory Direct Shear Strength Tests of Rock Specimens Under Constant Normal Force, ASTM International, West Conshohocken, PA (USA)

ASTM D7012-14e1, Standard Test Methods for Compressive Strength and Elastic Moduli of Intact Rock Core Specimens under Varying States of Stress and Temperatures, ASTM International, West Conshohocken, PA (USA)

Muralha, J., G. Grasselli, B. Tatone, M. Blumel, P. Chryssanthankis and J. Yunjing. 2014. ISRM Suggested Method for Laboratory Determination of the Shear Strength of Rock Joints: Revised Version. Rock Mech. Rock Eng., 47: 291-302.

Notes:

(1) Test results apply only to tested rock specimens. CanmetMINING makes no representation or warranty respecting the results arising therefrom, either expressly or implied by law or otherwise, including but not limited to implied warranties or conditions of merchantability or fitness for a particular purpose.

(2) The test program was carried out at CanmetMINING's Rock Mechanics Testing Laboratory located in Ottawa, Ontario. The address of the laboratory is:

Natural Resources Canada
CanmetMINING – Transformative Technologies and Specialized Services
Bells Corners Complex, Building 10
1 Haanel Drive
Ottawa, Ontario
Canada K1A 1M1

Attachment A – Summary of Test Results

Table A.1 - Physical Properties and Dynamic Moduli

Specimen Identification		Specifications (prepared)					Physical Properties and Dynamic Moduli					
Specimen ID	**Depth (m)**	**Diameter (mm)**	**Length (mm)**	**L : D**	**Volume (cm^3)**	**Mass (g)**	**Bulk Density (g/cm^3)**	**P-wave Velocity (km/s)**	**S-wave Velocity (km/s)**	**Young's Modulus (GPa)**	**Shear Modulus (GPa)**	**Poisson's Ratio**
Uniaxial Compression Strength (UCS) Test Specimens												
MA-GT-19-05-U1	98.03	60.59	137.04	2.26	395.09	1202.59	3.04	5.93	3.50	92.1	37.4	0.23
MA-GT-19-05-U2	323.35	60.46	137.01	2.27	393.35	1063.42	2.70	4.71	2.90	54.2	22.7	0.20
MA-GT-19-06-U1	101.64	60.42	136.99	2.27	392.77	1073.41	2.73	5.29	3.29	70.0	29.5	0.19
MA-GT-19-07-U1	81.24	60.92	137.05	2.25	399.43	1077.98	2.70	4.78	2.92	55.3	23.0	0.20
MA-GT-19-07-U2	179.24	60.69	137.05	2.26	396.46	1069.66	2.70	4.81	2.91	55.3	22.8	0.21
MA-GT-19-07-U3	307.01	60.79	137.00	2.25	397.67	1072.15	2.70	4.64	2.92	54.0	23.0	0.17
VL-GT-19-01-U1	111.63	60.89	136.90	2.25	398.69	1070.94	2.69	4.32	2.64	44.9	18.7	0.20
VL-GT-19-01-U2	353.56	60.74	136.91	2.25	396.67	1079.69	2.72	5.21	3.09	63.8	26.0	0.23
VL-GT-19-02-U1	233.38	60.24	136.91	2.27	390.16	1089.28	2.79	5.37	3.30	72.7	30.4	0.20
VL-GT-19-02-U2	349.25	60.39	136.91	2.27	392.15	1059.01	2.70	4.91	2.96	57.4	23.6	0.21
VL-GT-19-02-U3	233.23	60.18	136.99	2.28	389.70	1081.02	2.77	5.55	3.40	76.9	32.1	0.20
VL-GT-19-03-U1	54.74	60.74	136.99	2.26	396.99	1121.35	2.82	5.25	3.01	64.3	25.6	0.25
VL-GT-19-03-U2	84.54	60.71	136.99	2.26	396.55	1101.39	2.78	4.98	3.02	61.4	25.4	0.21
VL-GT-19-03-U3	84.68	60.68	137.00	2.26	396.23	1095.05	2.76	4.89	2.98	59.3	24.6	0.20
VL-GT-19-04-U1	75.71	60.58	137.04	2.26	395.04	1198.15	3.03	5.37	3.16	75.0	30.4	0.23
VL-GT-19-04-U2	118.52	60.86	136.90	2.25	398.21	1066.82	2.68	4.32	2.64	44.8	18.6	0.20
Triaxial Compression Strength (TCS) Test Specimens												
MA-GT-19-05-T1	102.48	60.64	137.00	2.26	395.67	1171.49	2.96	5.68	3.27	79.3	31.7	0.25
MA-GT-19-05-T2	227.59	60.45	137.04	2.27	393.35	1064.57	2.71	4.74	2.83	52.9	21.6	0.22
MA-GT-19-06-T1	99.75	60.39	136.99	2.27	392.38	1076.16	2.74	5.46	3.32	72.9	30.2	0.21
MA-GT-19-07-T1	363.7	60.86	137.05	2.25	398.64	1064.34	2.67	4.74	2.86	53.1	21.9	0.21
VL-GT-19-01-T1	228.11	60.98	136.91	2.25	399.90	1074.21	2.69	3.65	2.16	30.9	12.6	0.23
VL-GT-19-02-T1	141.78	60.49	136.91	2.26	393.41	1090.12	2.77	5.68	3.35	76.6	31.0	0.23
VL-GT-19-02-T2	349.61	60.42	136.90	2.27	392.51	1059.34	2.70	4.84	2.94	56.4	23.4	0.21
VL-GT-19-04-T1	75.1	60.60	137.00	2.26	395.19	1198.81	3.03	5.37	3.18	75.4	30.7	0.23
Brazilian Tensile Strength (BTS) Test Specimens												
MA-GT-19-05-B1	323.49	60.49	40.01	0.66	114.98	309.88	2.70	--	--	--	--	--
MA-GT-19-06-B1	101.78	60.44	40.01	0.66	114.79	313.51	2.73	--	--	--	--	--
MA-GT-19-07-B1	179.38	60.68	40.00	0.66	115.68	312.72	2.70	--	--	--	--	--
MA-GT-19-07-B2	307.15	60.81	39.99	0.66	116.14	310.47	2.67	--	--	--	--	--
VL-GT-19-01-B1	111.59	60.87	39.99	0.66	116.37	312.69	2.69	--	--	--	--	--
VL-GT-19-02-B1	349.51	60.36	39.99	0.66	114.43	307.65	2.69	--	--	--	--	--
VL-GT-19-04-B1	75.85	60.60	40.00	0.66	115.37	349.35	3.03	--	--	--	--	--
VL-GT-19-04-B2	118.48	60.85	40.00	0.66	116.32	311.54	2.68	--	--	--	--	--

Notes:

"--" = not measured, no data

- Samples tested under as-received moisture conditions (dry)
- As-received bulk density, assumed dry bulk density

Table A.2 - Compression Test Results

Specimen and Test Data					Strength and Elastic Properties				Description of Failure Mode
Specimen ID	Depth (m)	Date Tested	Load Rate (mm/s)	Confinement (MPa)	Peak Strength (MPa)	Axial Strain at Peak (%)	Young's Modulus[1] (GPa)	Poisson's Ratio[1]	
Uniaxial Compression Strength (UCS) Test Specimens									
MA-GT-19-05-U1	98.03	25-Sep-19	0.0008	0	62.4	0.095	72.8	0.16	- undulating shear across entire sample (65^o)
MA-GT-19-05-U2	323.35	25-Sep-19	0.0008	0	120.1	0.216	65.0	0.09	- shear along structure (35^o), from bottom
MA-GT-19-06-U1	101.64	25-Sep-19	0.0008	0	56.2	0.121	54.7	0.12	- shear along structure/foliation (55^o)
MA-GT-19-07-U1	81.24	25-Sep-19	0.0008	0	57.1	0.122	53.6	0.07	- multiple shear, internal defects
MA-GT-19-07-U2	179.24	25-Sep-19	0.0008	0	66.1	0.136	56.4	0.07	- undulating shear (60^o), influenced by internal micro-structure
MA-GT-19-07-U3	307.01	25-Sep-19	0.0008	0	113.2	0.190	67.0	0.10	- massive shear and axial splitting
VL-GT-19-01-U1	111.63	25-Sep-19	0.0008	0	76.4	0.166	52.2	0.08	- planar shear (50^o) from top, influenced by structure/foliation
VL-GT-19-01-U2	353.56	25-Sep-19	0.0008	0	68.6	0.130	59.1	0.07	- planar shear (65^o)
VL-GT-19-02-U1	233.38	25-Sep-19	0.0008	0	41.9	0.099	46.9	0.05	- shear along structure/foliation (55^o), from bottom
VL-GT-19-02-U2	349.25	26-Sep-19	0.0008	0	96.1	0.184	59.3	0.09	- axial splitting
VL-GT-19-02-U3	233.23	17-Oct-19	0.0008	0	43.8	0.082	53.7	0.11	- axial splitting with shear along structure/foliation (80^o)
VL-GT-19-03-U1	54.74	26-Sep-19	0.0008	0	78.2	0.239	41.2*	0.12*	- undulating shear (50^o), influenced by structure/foliation
VL-GT-19-03-U2	84.54	26-Sep-19	0.0008	0	63.4	0.185	42.5*	0.11*	- undulating shear (50^o), influenced by structure/foliation
VL-GT-19-03-U3	84.68	17-Oct-19	0.0008	0	66.1	0.178	42.3	0.11	- shear across sample on structure/foliation (50^o)
VL-GT-19-04-U1	75.71	26-Sep-19	0.0008	0	301.6	0.408	83.1	0.16	- massive shear, brittle/violent failure
VL-GT-19-04-U2	118.52	26-Sep-19	0.0008	0	40.2	0.132	41.1	0.04	- planar shear (40^o), internal defects
Triaxial Compression Strength (TCS) Test Specimens									
MA-GT-19-05-T1	102.48	30-Sep-19	0.0008	3	204.8	0.305	81.6	0.17	- multiple shear, primary undulating shear at 70^o, from bottom
MA-GT-19-05-T2	227.59	01-Oct-19	0.0008	3	182.5	0.318	70.6	0.12	- planar shear, 60^o
MA-GT-19-06-T1	99.75	01-Oct-19	0.0008	3	56.1	0.113	59.7	0.12	- shear along structure/foliation (60^o), from bottom
MA-GT-19-07-T1	363.70	01-Oct-19	0.0008	7	164.8	0.261	76.3	0.09	- planar shear (35^o) with multiple failure along internal defects
VL-GT-19-01-T1	228.11	02-Oct-19	0.0008	3	46.9	0.189	35.2	0.15	- multiple shear, internal defects/micro-structure
VL-GT-19-02-T1	141.78	02-Oct-19	0.0008	7	134.2	0.252	76.8	0.10	- planar shear (80^o) along structure, axial splitting
VL-GT-19-02-T2	349.61	02-Oct-19	0.0008	7	198.0	0.402	68.3	0.17	- multiple shear with primary shear at 70^o
VL-GT-19-04-T1	75.10	02-Oct-19	0.0008	7	322.3	0.421	89.9	0.22	- multiple shear

Notes:
- Testing completed in accordance with ASTM D7012 -14, using the MTS 815 load frame (axial displacement control)
- All samples tested 'as-received' (dry)

1) Elastic properties calculated b/w 40-60% of peak strength unless otherwise noted

* calculated b/w 30-50% of peak strength

Table A.3 - Splitting Tensile (Brazilian) Test Results

Specimen and Test Data				Strength Properties		Description of Failure Mode
Specimen ID	Date Tested	Moisture Condition	Load Rate (kN/s)	Maximum Applied Load, $P_{Failure}$ (kN)	Splitting Tensile Strength, σ_t (MPa)	
MA-GT-19-05-B1	18-Sep-19	as-received (dry)	0.15	60.2	15.8	- Diametral splitting
MA-GT-19-06-B1	18-Sep-19	as-received (dry)	0.15	33.6	8.8	- Diametral splitting with some crushing at the sample ends. Loaded normal to fabric dip.
MA-GT-19-07-B1	18-Sep-19	as-received (dry)	0.15	76.5	20.1	- Diametral splitting, multiple
MA-GT-19-07-B2	18-Sep-19	as-received (dry)	0.15	42.6	11.2	- Diametral splitting
VL-GT-19-01-B1	18-Sep-19	as-received (dry)	0.15	49.8	13.0	- Diametral splitting
VL-GT-19-02-B1	18-Sep-19	as-received (dry)	0.15	59.5	15.7	- Diametral splitting
VL-GT-19-04-B1	18-Sep-19	as-received (dry)	0.15	68.6	18.0	- Diametral splitting
VL-GT-19-04-B2	18-Sep-19	as-received (dry)	0.15	44.2	11.6	- Diametral splitting, massive failure/crushing at ends

Notes:
- Testing completed in accordance with ASTM D3967-16, using the MTS 815 load frame (axial load control)

Table A.4 - Direct Shear Test Results

Specimen and Test Data								Shear Properties				Mohr-Coulomb
Specimen ID	Depth (m)	Date Tested	Description	Initial JRC	Initial Area[1] (mm^2)	Test Stage	Shear Rate (mm/min)	Average Normal Load (kN)	Average Normal Stress[1] (MPa)	Peak Shear Stress (MPa)	Residual Shear Stress (MPa)	Residual Friction Angle (degrees)[2]
MA-GT-19-06-DS1	56.39	21-Oct-19	natural joint, dip = 40° (to core axis), smooth waxy surface with faint lineations, tested parallel to lineation	0 - 2	4167	1	0.4	3.92	0.94	--	0.41	23.3
						2	0.4	8.00	1.92	--	0.82	
						3	0.3	12.11	2.91	--	1.25	
MA-GT-19-06-DS2	157.59	21-Oct-19	natural joint, dip angle ~ 43° , well defined slickenside lineations at 25° to dip dir., undulating normal to lineation, tested parallel to lineation	0 - 2	4105	1	0.4	3.88	0.94	--	0.49	25.5
						2	0.5	7.92	1.92	--	0.85	
						3	0.5	12.01	2.91	--	1.42	
MA-GT-19-06-DS3	76.41	18-Nov-19	natural joint along foliation, dip angle ~ 40°, tested parallel to dip direction	4 - 6	4479	1	0.2	2.10	0.47	1.04	0.35	33.8
						2	0.4	8.63	1.93	--	1.29	
						3	0.3	17.48	3.90	--	2.61	
MA-GT-19-06-DS4	202.12	19-Nov-19	natural joint, dip angle ~45° to core axis, small steps/ridges, waxy coating on surface	6 - 8	3838	1	0.3	1.80	0.47	1.18	0.31	34.7
						2	0.4	7.36	1.92	--	1.30	
						3	0.3	14.93	3.89	--	2.72	
MA-GT-19-07-DS1	77.64	18-Oct-19	natural joint, dip angle ~ 58° , well defined slickenside lineations at 45° to dip dir., tested parallel to lineation	2 - 4	3258	1	0.4	3.05	0.94	--	0.69	32.6
						2	0.4	6.22	1.91	--	1.28	
						3	0.4	9.43	2.89	--	1.78	
VL-GT-19-01-DS1	282.26	16-Oct-19	natural joint, rough surface, no infilling, waxy coating on joint surface, dip = 63° (to core axis)	12 - 14	3210	1	0.4	2.97	0.93	1.38	0.62	32.3
						2	0.3	6.12	1.91	--	1.25	
						3	0.3	9.28	2.89	--	1.78	
VL-GT-19-02-DS1	199.40	20-Nov-19	natural joint, rough, dip angle ~30° to core axis	12 - 14	5463	1	0.4	2.57	0.47	1.15	0.33	36.3
						2	0.3	10.58	1.94	--	1.47	
						3	0.4	21.40	3.92	--	2.86	
VL-GT-19-03-DS1	54.55	16-Oct-19	natural joint, undulating surface, waxy coating, no infilling, dip ~ 60° (to core axis)	8 - 10	3229	1	0.3	3.03	0.94	0.71	0.49	27.8
						2	0.5	6.15	1.91	1.34	1.06	
						3	0.2	9.31	2.88	--	1.48	
VL-GT-19-04-DS1	112.62	19-Nov-19	natural joint, rough with minor undulation, dip ~45° (to core axis)	10 - 12	3901	1	0.3	1.83	0.47	0.49	0.30	32.2
						2	0.4	7.49	1.92	--	1.10	
						3	0.3	15.21	3.90	--	2.49	

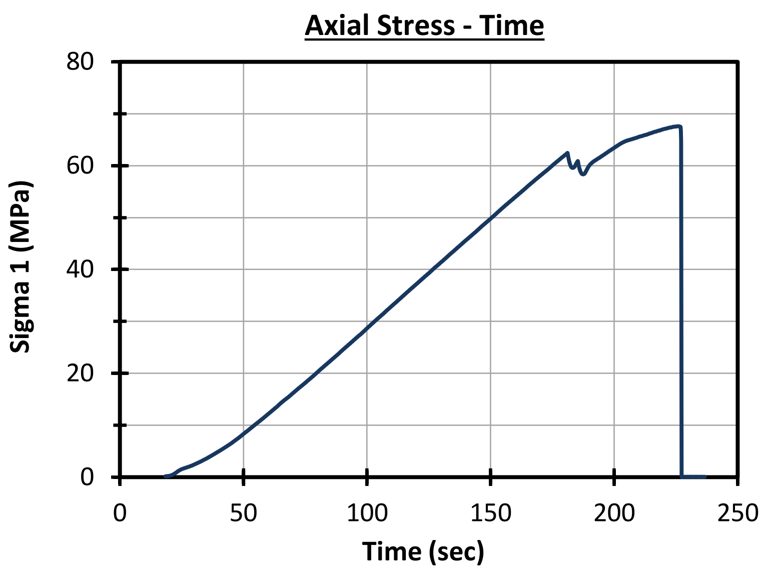
Notes:

1 - Average normal stress is based on the initial surface area of the discontinuity and assumed to be constant during the test

2 - Residual friction angle is calculated from the best fit linear regression line of the shear stress vs. normal stress plot assuming zero cohesion

Attachment B – Detailed Compression Test Data

Uniaxial Compression Strength Test: MA-GT-19-05-U1

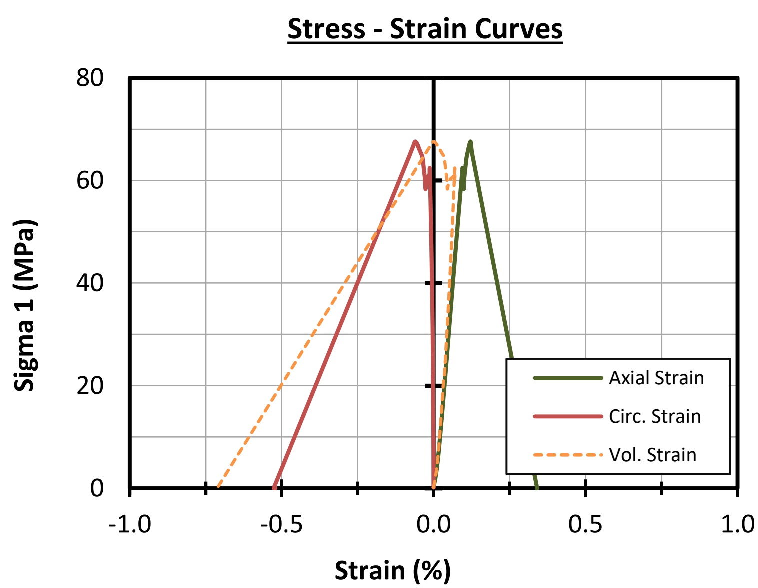


Test and Specimen Data:

Specimen ID:	**MA-GT-19-05-U1**	**Moisture Condition:**	as-received (dry)
Sample Depth (m):	98.03	**Test Completed on:**	25-Sep-19
Length (mm):	137.04	**Load Control:**	axial
Diameter (mm):	60.59	**Loading Rate (mm/s):**	0.0008

Test Results:

Uniaxial Compressive Strength (MPa)	62.4
Axial Strain at Peak (%)	0.095
Maximum Volumetric Strain (%)	0.071
Young's Modulus, E (GPa)[1]	72.8
Poisson's Ratio[2]	0.16

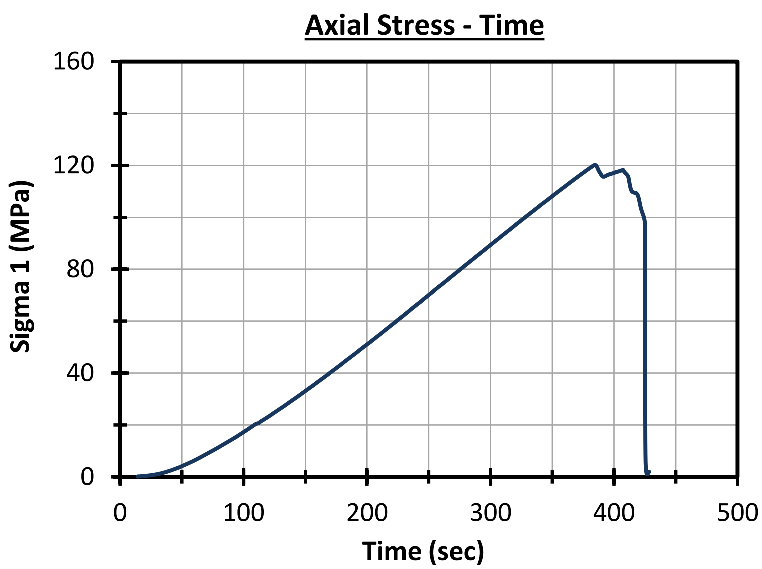


Failure Description: - undulating shear across entire sample (65^{o})

Notes:

1) Average slope of linear portion of axial stress-strain curve, b/w 40-60 % peak strength
2) Calculated over the same range of peak strength as E

Uniaxial Compression Strength Test: MA-GT-19-05-U2

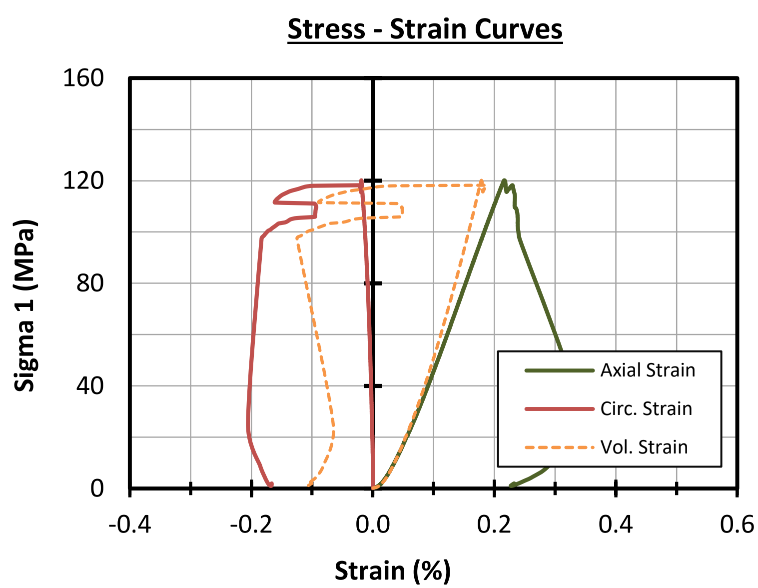


Test and Specimen Data:

Specimen ID:	**MA-GT-19-05-U2**	**Moisture Condition:**	as-received (dry)
Sample Depth (m):	323.35	**Test Completed on:**	25-Sep-19
Length (mm):	137.01	**Load Control:**	axial
Diameter (mm):	60.46	**Loading Rate (mm/s):**	0.0008

Test Results:

Uniaxial Compressive Strength (MPa)	120.1
Axial Strain at Peak (%)	0.216
Maximum Volumetric Strain (%)	0.179
Young's Modulus, E (GPa)[1]	65.0
Poisson's Ratio[2]	0.09

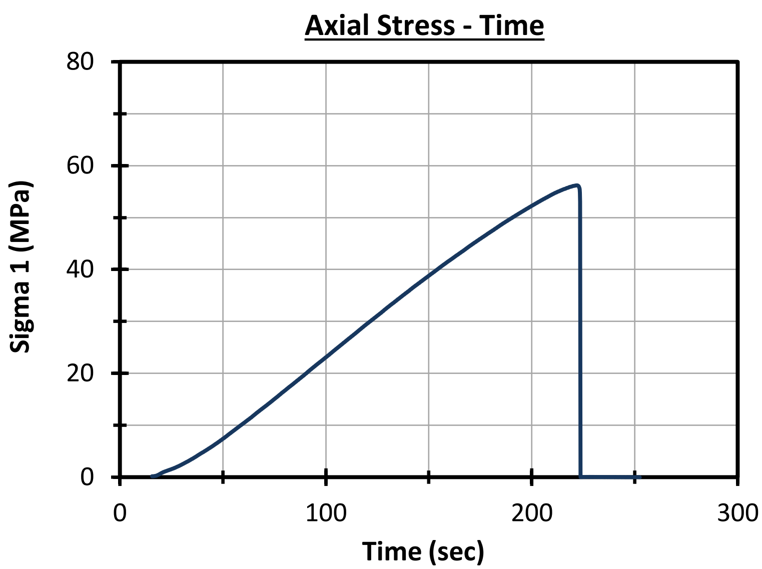


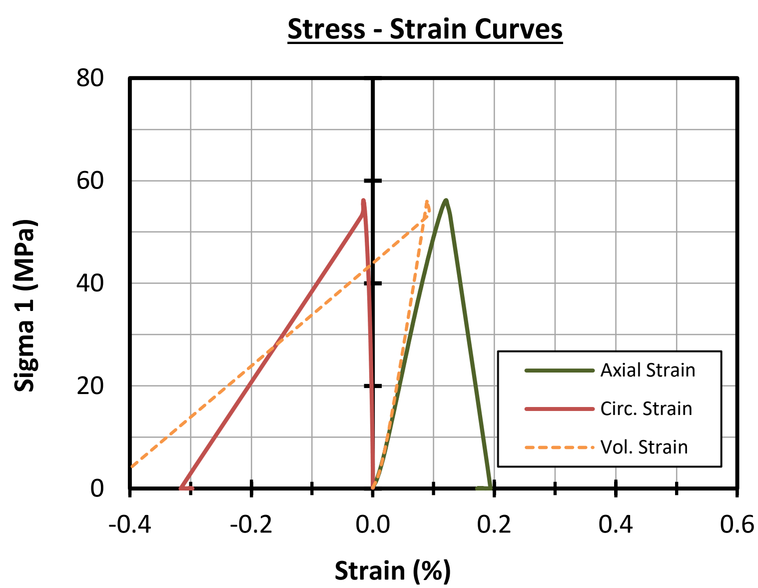
Failure Description: - shear along structure (35^{o}), from bottom

Notes:

1) Average slope of linear portion of axial stress-strain curve, b/w 40-60 % peak strength
2) Calculated over the same range of peak strength as E

Uniaxial Compression Strength Test: MA-GT-19-06-U1





Test and Specimen Data:

Specimen ID:	**MA-GT-19-06-U1**	**Moisture Condition:**	as-received (dry)
Sample Depth (m):	101.64	**Test Completed on:**	25-Sep-19
Length (mm):	136.99	**Load Control:**	axial
Diameter (mm):	60.42	**Loading Rate (mm/s):**	0.0008

Test Results:

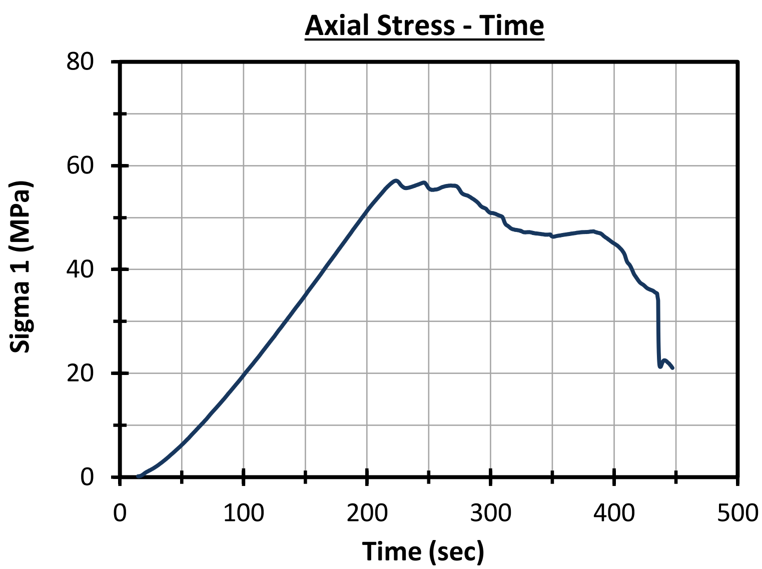
Uniaxial Compressive Strength (MPa)	56.2
Axial Strain at Peak (%)	0.121
Maximum Volumetric Strain (%)	0.089
Young's Modulus, E (GPa)[1]	54.7
Poisson's Ratio[2]	0.12

Failure Description: - shear along structure/foliation (55^{o})

Notes:

1) Average slope of linear portion of axial stress-strain curve, b/w 40-60 % peak strength
2) Calculated over the same range of peak strength as E

Uniaxial Compression Strength Test: MA-GT-19-07-U1

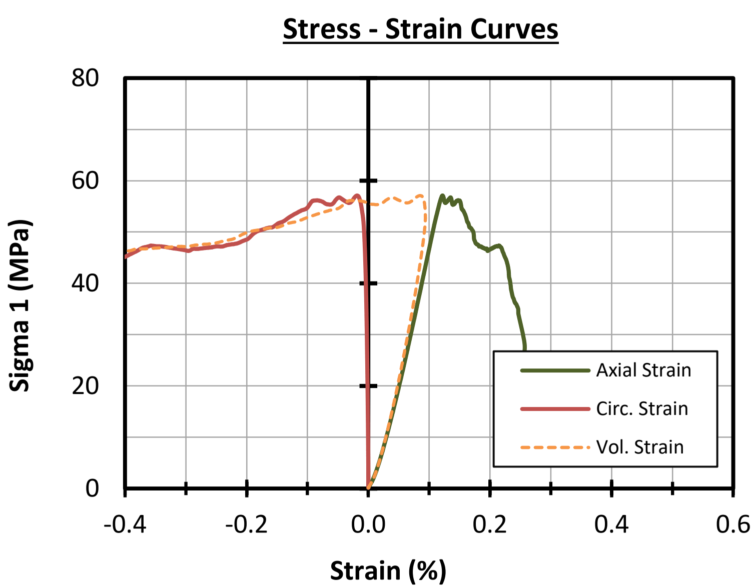


Test and Specimen Data:

Specimen ID:	**MA-GT-19-07-U1**	**Moisture Condition:**	as-received (dry)
Sample Depth (m):	81.24	**Test Completed on:**	25-Sep-19
Length (mm):	137.05	**Load Control:**	axial
Diameter (mm):	60.92	**Loading Rate (mm/s):**	0.0008

Test Results:

Uniaxial Compressive Strength (MPa)	57.1
Axial Strain at Peak (%)	0.122
Maximum Volumetric Strain (%)	0.094
Young's Modulus, E (GPa)[1]	53.6
Poisson's Ratio[2]	0.07

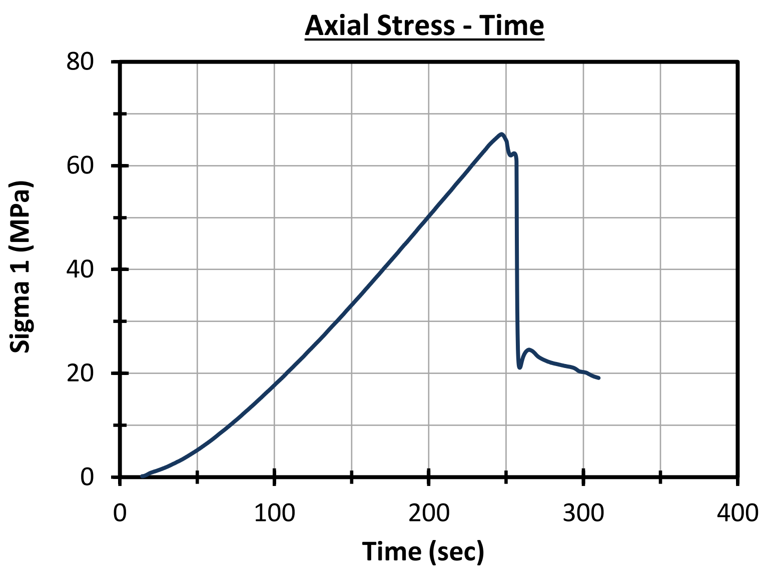


Failure Description: - multiple shear, internal defects

Notes:

1) Average slope of linear portion of axial stress-strain curve, b/w 40-60 % peak strength
2) Calculated over the same range of peak strength as E

Uniaxial Compression Strength Test: MA-GT-19-07-U2

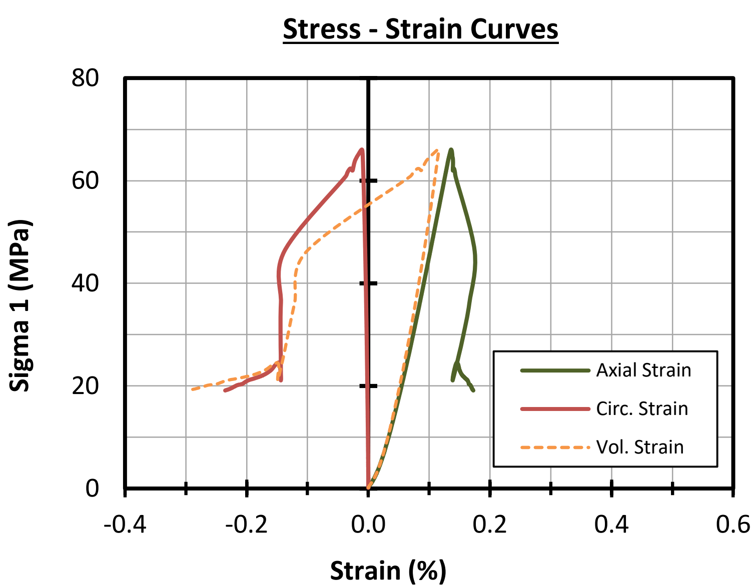


Test and Specimen Data:

Specimen ID:	**MA-GT-19-07-U2**	**Moisture Condition:**	as-received (dry)
Sample Depth (m):	179.24	**Test Completed on:**	25-Sep-19
Length (mm):	137.05	**Load Control:**	axial
Diameter (mm):	60.69	**Loading Rate (mm/s):**	0.0008

Test Results:

Uniaxial Compressive Strength (MPa)	66.1
Axial Strain at Peak (%)	0.136
Maximum Volumetric Strain (%)	0.116
Young's Modulus, E (GPa)[1]	56.4
Poisson's Ratio[2]	0.07

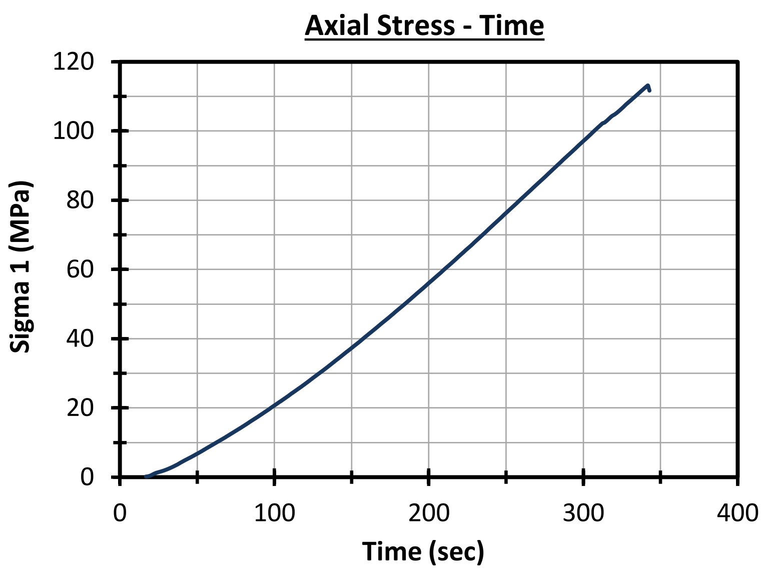


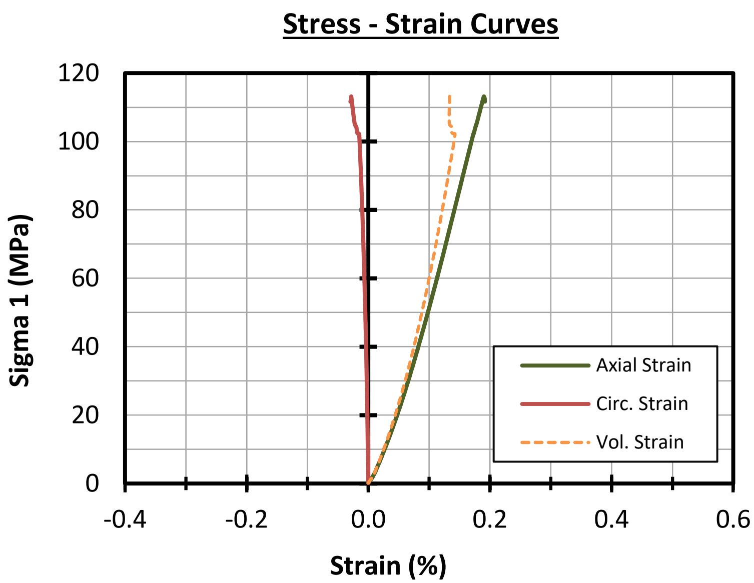
Failure Description: - undulating shear (60°), influenced by internal micro-structure

Notes:

1) Average slope of linear portion of axial stress-strain curve, b/w 40-60 % peak strength
2) Calculated over the same range of peak strength as E

Uniaxial Compression Strength Test: MA-GT-19-07-U3





Test and Specimen Data:

Specimen ID:	**MA-GT-19-07-U3**	**Moisture Condition:**	as-received (dry)
Sample Depth (m):	307.01	**Test Completed on:**	25-Sep-19
Length (mm):	137.00	**Load Control:**	axial
Diameter (mm):	60.79	**Loading Rate (mm/s):**	0.0008

Test Results:

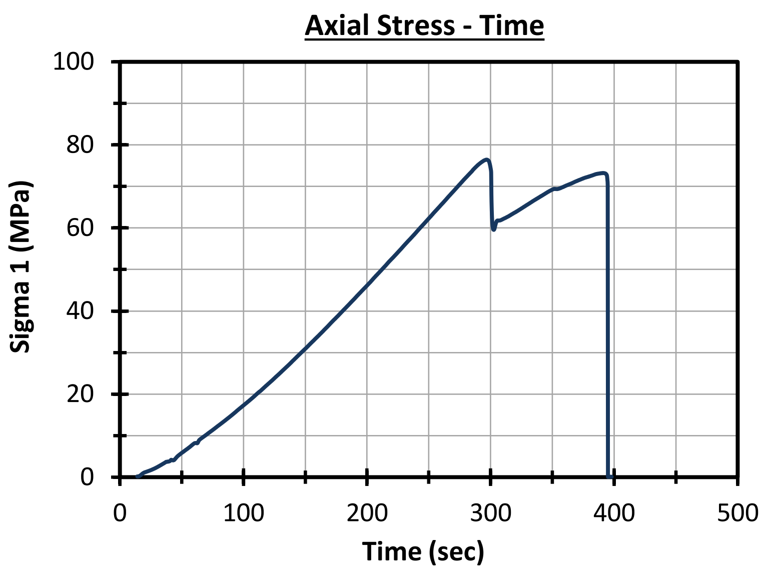
Uniaxial Compressive Strength (MPa)	113.2
Axial Strain at Peak (%)	0.190
Maximum Volumetric Strain (%)	0.143
Young's Modulus, E (GPa)[1]	67.0
Poisson's Ratio[2]	0.10

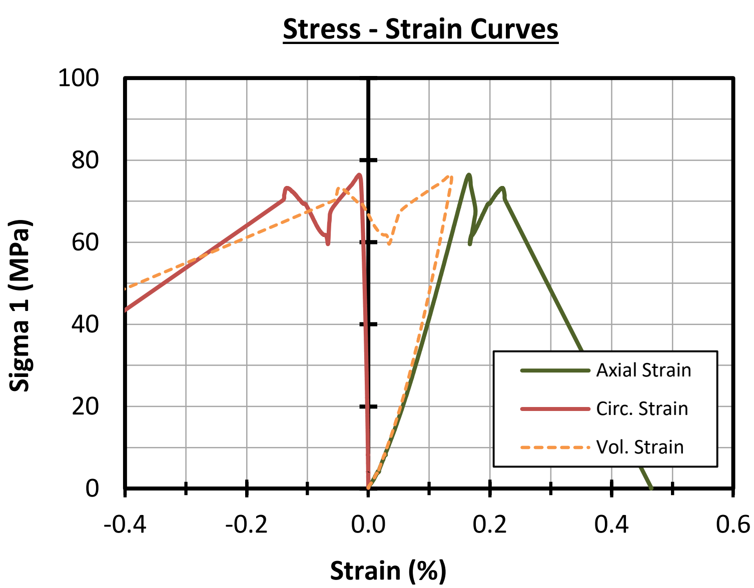
Failure Description: - massive shear and axial splitting

Notes:

1) Average slope of linear portion of axial stress-strain curve, b/w 40-60 % peak strength
2) Calculated over the same range of peak strength as E

Uniaxial Compression Strength Test: VL-GT-19-01-U1





Test and Specimen Data:

Specimen ID:	**VL-GT-19-01-U1**	**Moisture Condition:**	as-received (dry)
Sample Depth (m):	111.63	**Test Completed on:**	25-Sep-19
Length (mm):	136.90	**Load Control:**	axial
Diameter (mm):	60.89	**Loading Rate (mm/s):**	0.0008

Test Results:

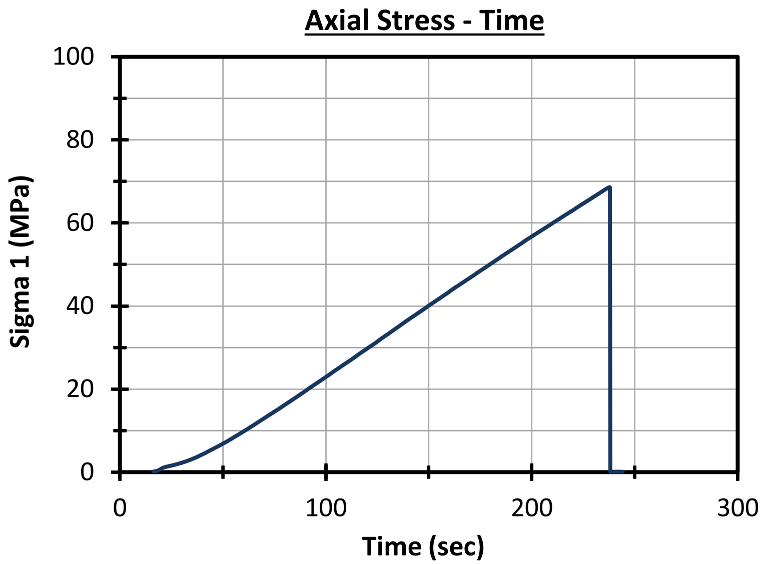
Uniaxial Compressive Strength (MPa)	76.4
Axial Strain at Peak (%)	0.166
Maximum Volumetric Strain (%)	0.137
Young's Modulus, E (GPa)[1]	52.2
Poisson's Ratio[2]	0.08

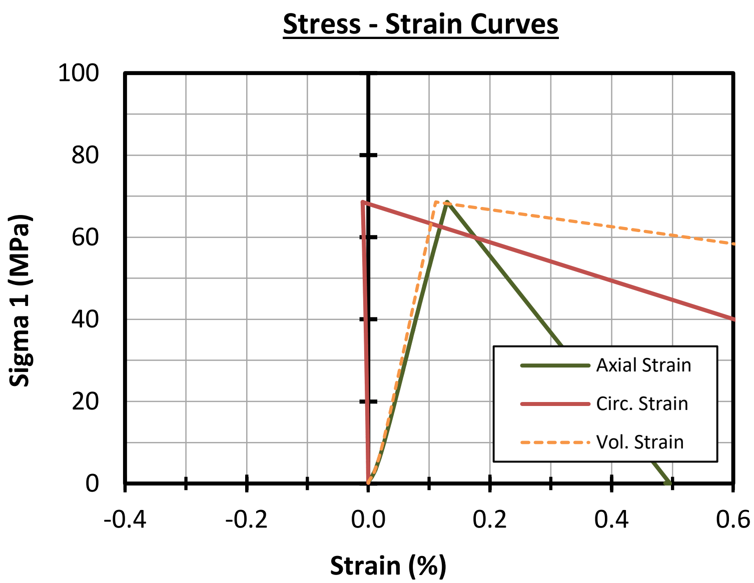
Failure Description: - planar shear (50^{o}) from top, influenced by structure/foliation

Notes:

1) Average slope of linear portion of axial stress-strain curve, b/w 40-60 % peak strength
2) Calculated over the same range of peak strength as E

Uniaxial Compression Strength Test: VL-GT-19-01-U2





Test and Specimen Data:

Specimen ID:	**VL-GT-19-01-U2**	**Moisture Condition:**	as-received (dry)
Sample Depth (m):	353.56	**Test Completed on:**	25-Sep-19
Length (mm):	136.91	**Load Control:**	axial
Diameter (mm):	60.74	**Loading Rate (mm/s):**	0.0008

Test Results:

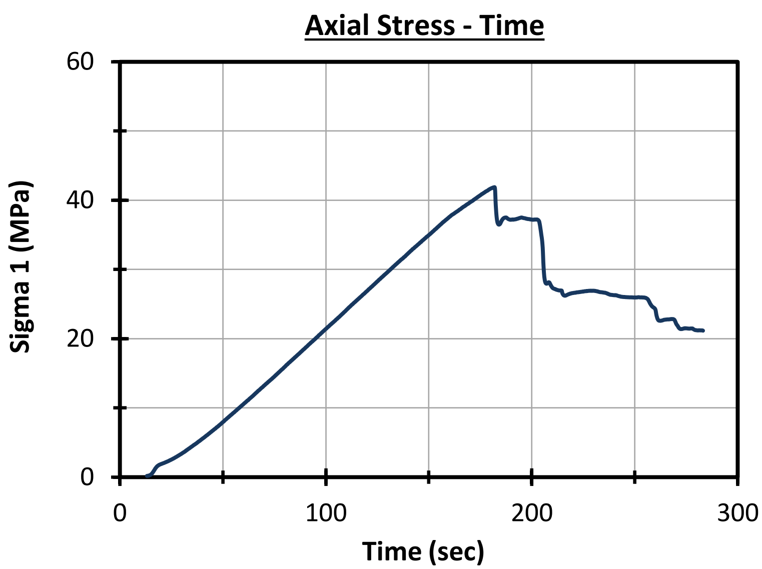
Uniaxial Compressive Strength (MPa)	68.6
Axial Strain at Peak (%)	0.130
Maximum Volumetric Strain (%)	0.111
Young's Modulus, E (GPa)[1]	59.1
Poisson's Ratio[2]	0.07

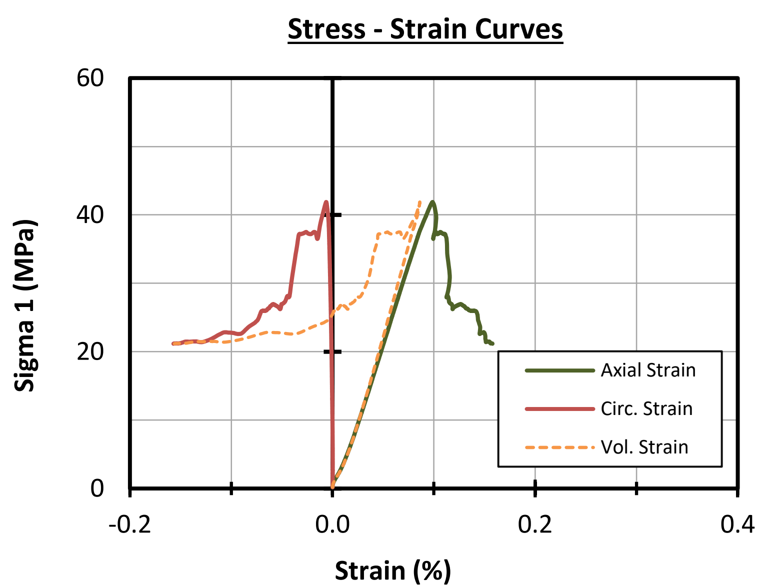
Failure Description: - planar shear (65^{o})

Notes:

1) Average slope of linear portion of axial stress-strain curve, b/w 40-60 % peak strength
2) Calculated over the same range of peak strength as E

Uniaxial Compression Strength Test: VL-GT-19-02-U1





Test and Specimen Data:

Specimen ID:	**VL-GT-19-02-U1**	**Moisture Condition:**	as-received (dry)
Sample Depth (m):	233.38	**Test Completed on:**	25-Sep-19
Length (mm):	136.91	**Load Control:**	axial
Diameter (mm):	60.24	**Loading Rate (mm/s):**	0.0008

Test Results:

Uniaxial Compressive Strength (MPa)	41.9
Axial Strain at Peak (%)	0.099
Maximum Volumetric Strain (%)	0.086
Young's Modulus, E (GPa)[1]	46.9
Poisson's Ratio[2]	0.05

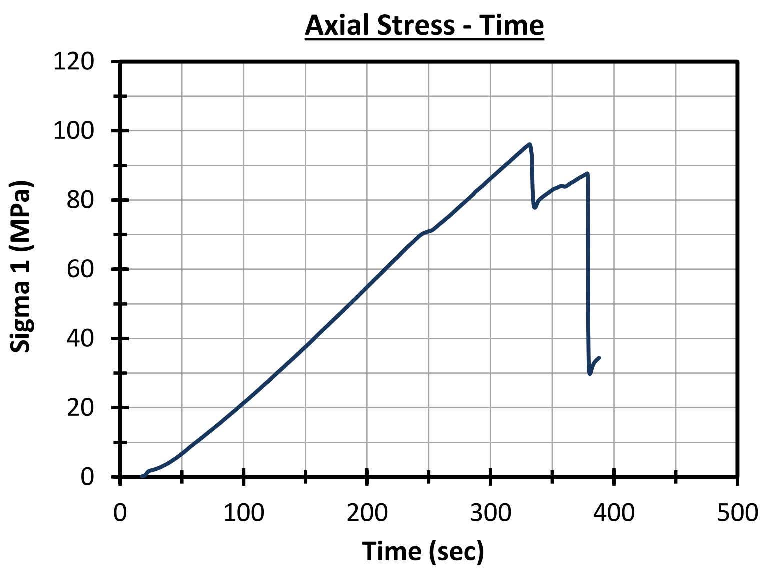
Failure Description: - shear along structure/foliation (55^{o}), from bottom

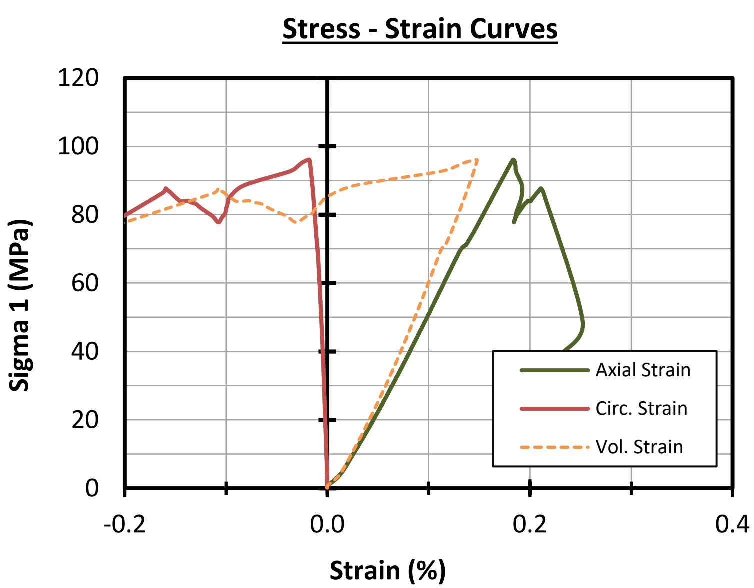
Notes:

1) Average slope of linear portion of axial stress-strain curve, b/w 40-60 % peak strength
2) Calculated over the same range of peak strength as E

Tested by: GB, TA
Reviewed by: SG

Uniaxial Compression Strength Test: VL-GT-19-02-U2





Test and Specimen Data:

Specimen ID:	**VL-GT-19-02-U2**	**Moisture Condition:**	as-received (dry)
Sample Depth (m):	349.25	**Test Completed on:**	26-Sep-19
Length (mm):	136.91	**Load Control:**	axial
Diameter (mm):	60.39	**Loading Rate (mm/s):**	0.0008

Test Results:

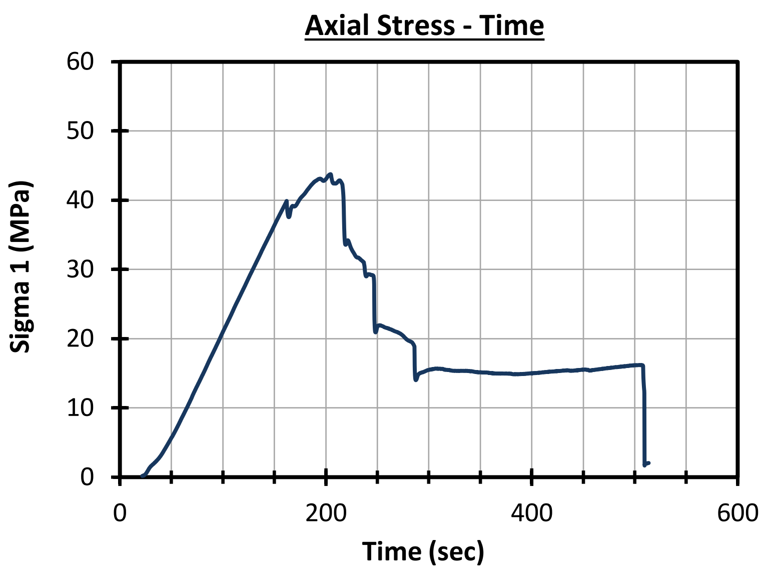
Uniaxial Compressive Strength (MPa)	96.1
Axial Strain at Peak (%)	0.184
Maximum Volumetric Strain (%)	0.148
Young's Modulus, E (GPa)[1]	59.3
Poisson's Ratio[2]	0.09

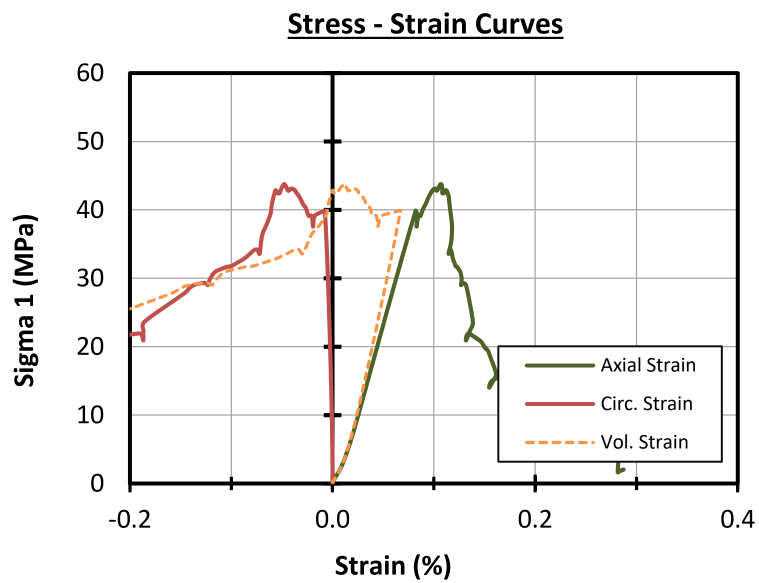
Failure Description: - axial splitting

Notes:

1) Average slope of linear portion of axial stress-strain curve, b/w 40-60 % peak strength
2) Calculated over the same range of peak strength as E

Uniaxial Compression Strength Test: VL-GT-19-02-U3





Test and Specimen Data:

Specimen ID:	**VL-GT-19-02-U3**	**Moisture Condition:**	as-received (dry)
Sample Depth (m):	233.23	**Test Completed on:**	17-Oct-19
Length (mm):	136.99	**Load Control:**	axial
Diameter (mm):	60.18	**Loading Rate (mm/s):**	0.0008

Test Results:

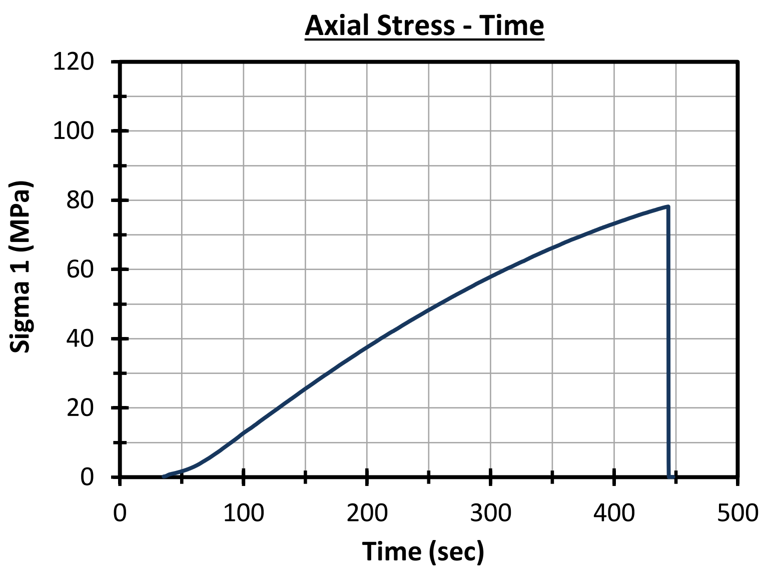
Uniaxial Compressive Strength (MPa)	43.8
Axial Strain at Peak (%)	0.082
Maximum Volumetric Strain (%)	0.067
Young's Modulus, E (GPa)[1]	53.7
Poisson's Ratio[2]	0.11

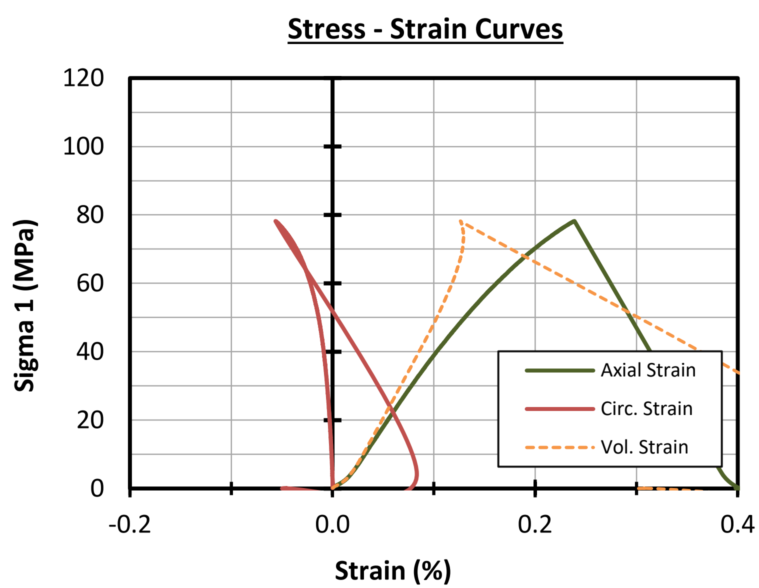
Failure Description: - axial splitting with shear along structure/foliation (80^{o})

Notes:

1) Average slope of linear portion of axial stress-strain curve, b/w 40-60 % peak strength
2) Calculated over the same range of peak strength as E

Uniaxial Compression Strength Test: VL-GT-19-03-U1





Test and Specimen Data:

Specimen ID:	**VL-GT-19-03-U1**	**Moisture Condition:**	as-received (dry)
Sample Depth (m):	54.74	**Test Completed on:**	26-Sep-19
Length (mm):	136.99	**Load Control:**	axial
Diameter (mm):	60.74	**Loading Rate (mm/s):**	0.0008

Test Results:

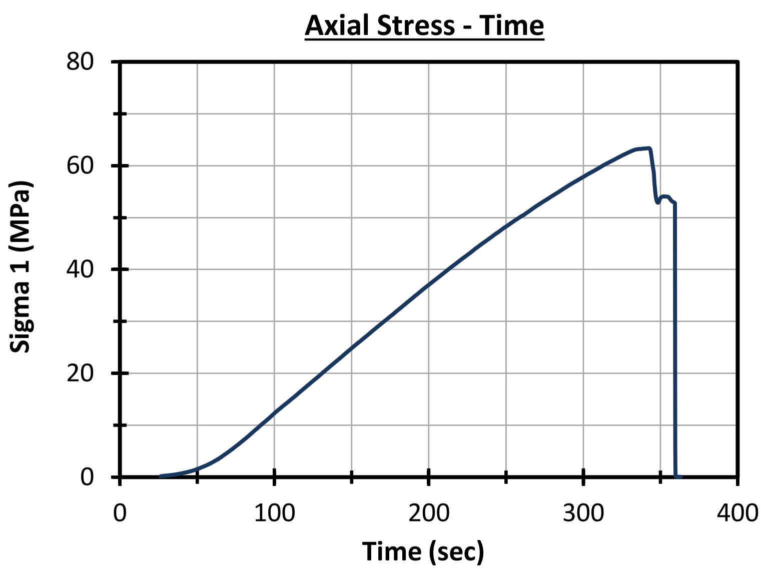
Uniaxial Compressive Strength (MPa)	78.2
Axial Strain at Peak (%)	0.239
Maximum Volumetric Strain (%)	0.129
Young's Modulus, E (GPa)[1]	41.2
Poisson's Ratio[2]	0.12

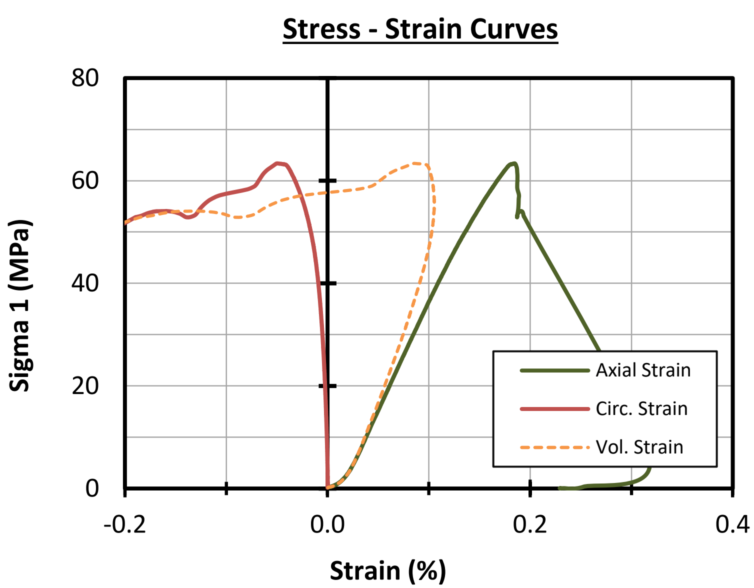
Failure Description: - undulating shear (50^{o}), influenced by structure/foliation

Notes:

1) Average slope of linear portion of axial stress-strain curve, b/w **30-50 %** peak strength
2) Calculated over the same range of peak strength as E

Uniaxial Compression Strength Test: VL-GT-19-03-U2





Test and Specimen Data:

Specimen ID:	**VL-GT-19-03-U2**	**Moisture Condition:**	as-received (dry)
Sample Depth (m):	84.54	**Test Completed on:**	26-Sep-19
Length (mm):	136.99	**Load Control:**	axial
Diameter (mm):	60.71	**Loading Rate (mm/s):**	0.0008

Test Results:

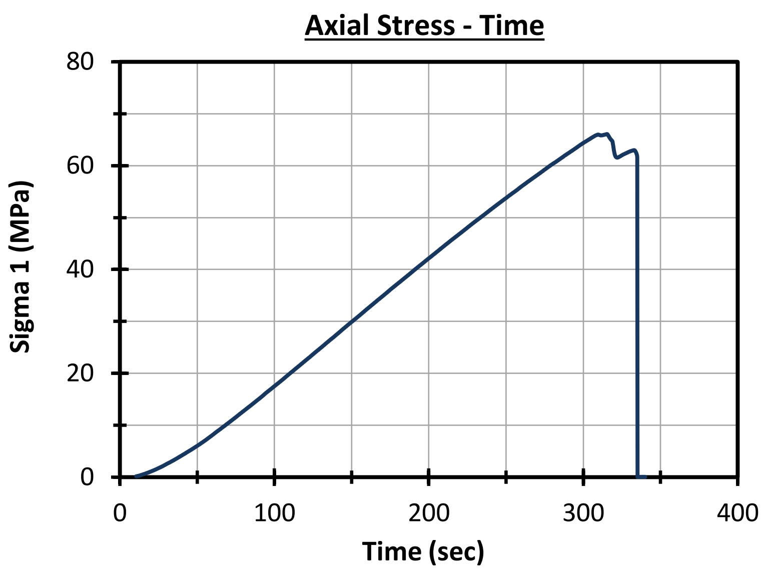
Uniaxial Compressive Strength (MPa)	63.4
Axial Strain at Peak (%)	0.185
Maximum Volumetric Strain (%)	0.105
Young's Modulus, E (GPa)[1]	42.5
Poisson's Ratio[2]	0.11

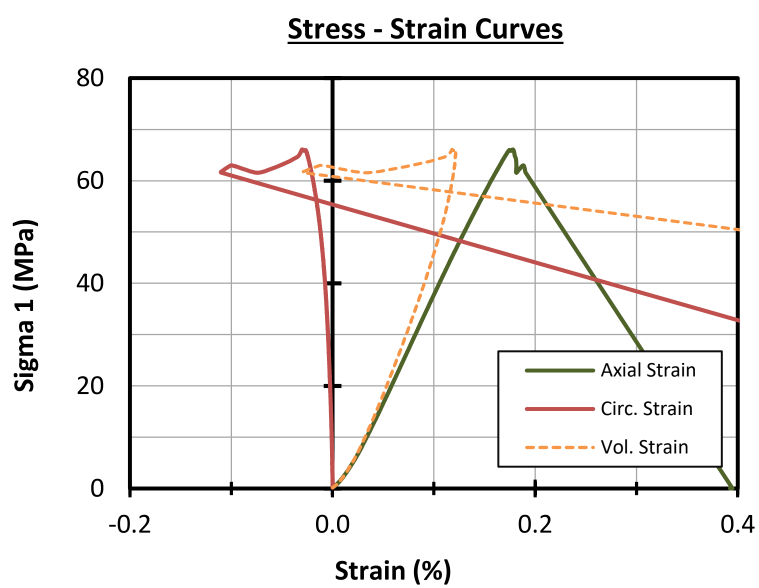
Failure Description: - undulating shear (50^o), influenced by structure/foliation

Notes:

1) Average slope of linear portion of axial stress-strain curve, b/w **30-50 %** peak strength
2) Calculated over the same range of peak strength as E

Uniaxial Compression Strength Test: VL-GT-19-03-U3





Test and Specimen Data:

Specimen ID:	**VL-GT-19-03-U3**	**Moisture Condition:**	as-received (dry)
Sample Depth (m):	84.68	**Test Completed on:**	17-Oct-19
Length (mm):	137.00	**Load Control:**	axial
Diameter (mm):	60.68	**Loading Rate (mm/s):**	0.0008

Test Results:

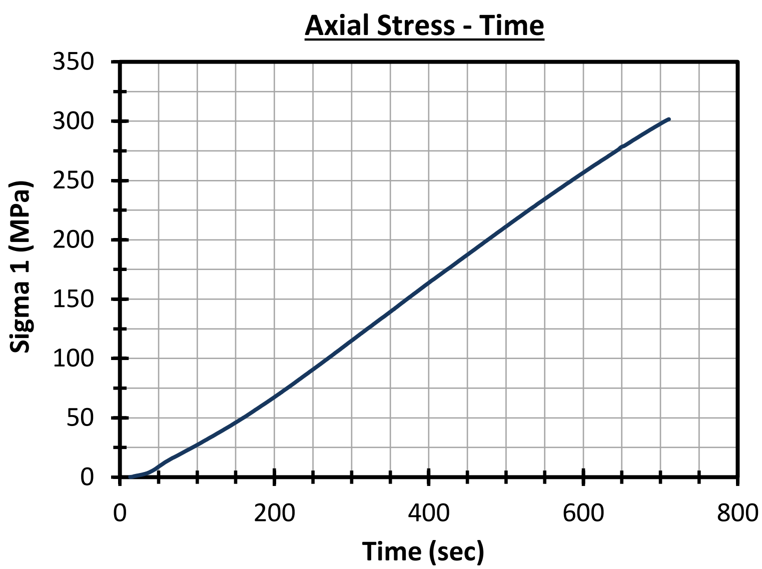
Uniaxial Compressive Strength (MPa)	66.1
Axial Strain at Peak (%)	0.178
Maximum Volumetric Strain (%)	0.122
Young's Modulus, E (GPa)[1]	42.3
Poisson's Ratio[2]	0.11

Failure Description: - shear across sample on structure/foliation (50°)

Notes:

1) Average slope of linear portion of axial stress-strain curve, b/w 40-60 % peak strength
2) Calculated over the same range of peak strength as E

Uniaxial Compression Strength Test: VL-GT-19-04-U1

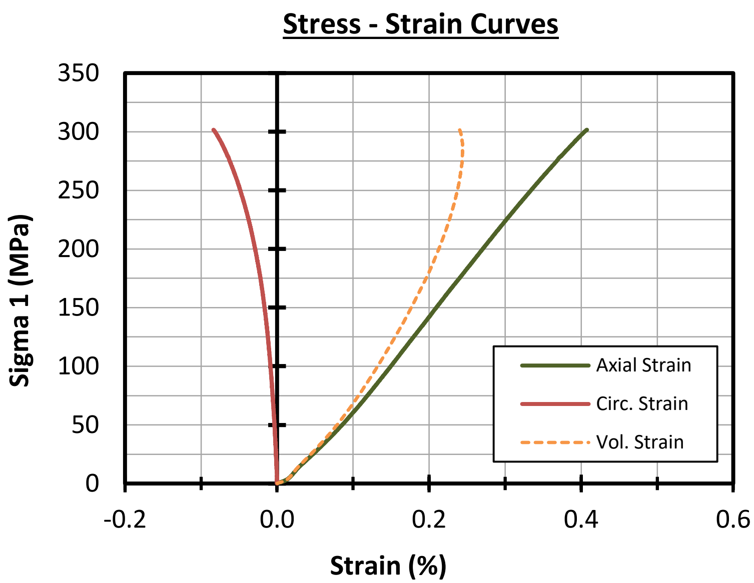


Test and Specimen Data:

Specimen ID:	**VL-GT-19-04-U1**	**Moisture Condition:**	as-received (dry)
Sample Depth (m):	75.71	**Test Completed on:**	26-Sep-19
Length (mm):	137.04	**Load Control:**	axial
Diameter (mm):	60.58	**Loading Rate (mm/s):**	0.0008

Test Results:

Uniaxial Compressive Strength (MPa)	301.6
Axial Strain at Peak (%)	0.408
Maximum Volumetric Strain (%)	0.244
Young's Modulus, E (GPa)[1]	83.1
Poisson's Ratio[2]	0.16

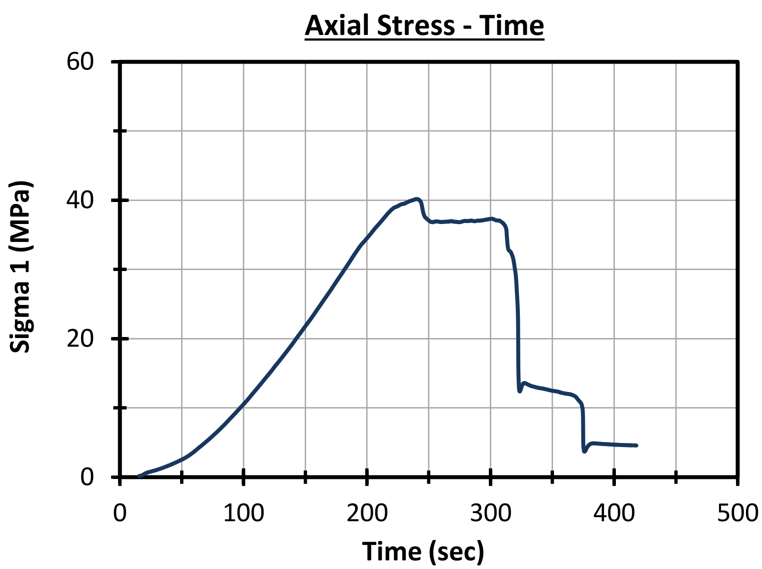


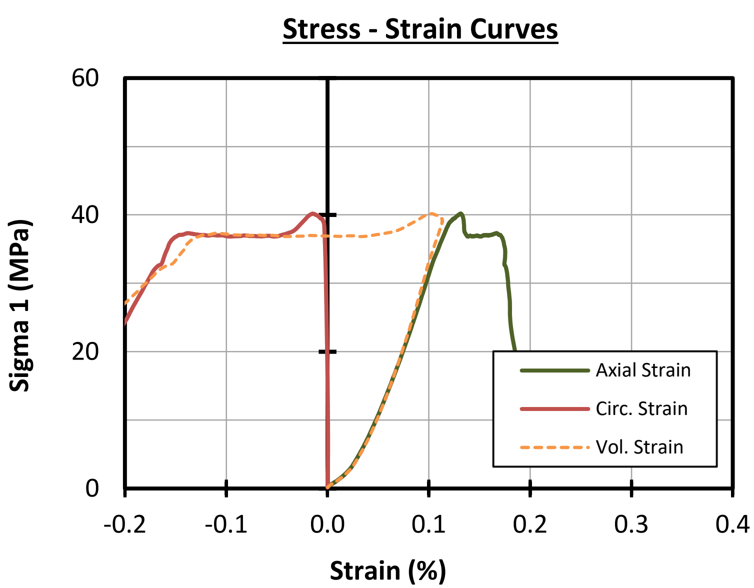
Failure Description: - massive shear, brittle/violent failure

Notes:

1) Average slope of linear portion of axial stress-strain curve, b/w 40-60 % peak strength
2) Calculated over the same range of peak strength as E

Uniaxial Compression Strength Test: VL-GT-19-04-U2





Test and Specimen Data:

Specimen ID:	**VL-GT-19-04-U2**	**Moisture Condition:**	as-received (dry)
Sample Depth (m):	118.52	**Test Completed on:**	26-Sep-19
Length (mm):	136.90	**Load Control:**	axial
Diameter (mm):	60.86	**Loading Rate (mm/s):**	0.0008

Test Results:

Uniaxial Compressive Strength (MPa)	40.2
Axial Strain at Peak (%)	0.132
Maximum Volumetric Strain (%)	0.114
Young's Modulus, E (GPa)[1]	41.1
Poisson's Ratio[2]	0.04

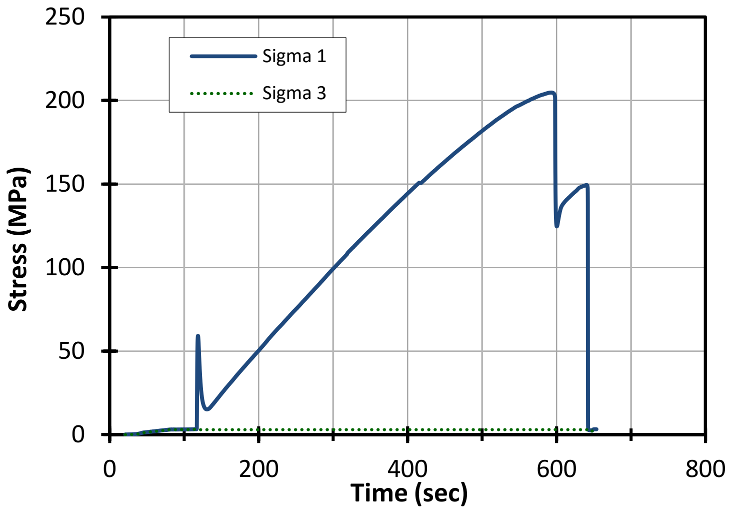
Failure Description: - planar shear (40^{o}), internal defects

Notes:

1) Average slope of linear portion of axial stress-strain curve, b/w 40-60 % peak strength
2) Calculated over the same range of peak strength as E

Triaxial Compression Strength Test: MA-GT-19-05-T1

Stress - Time



Test and Specimen Data:

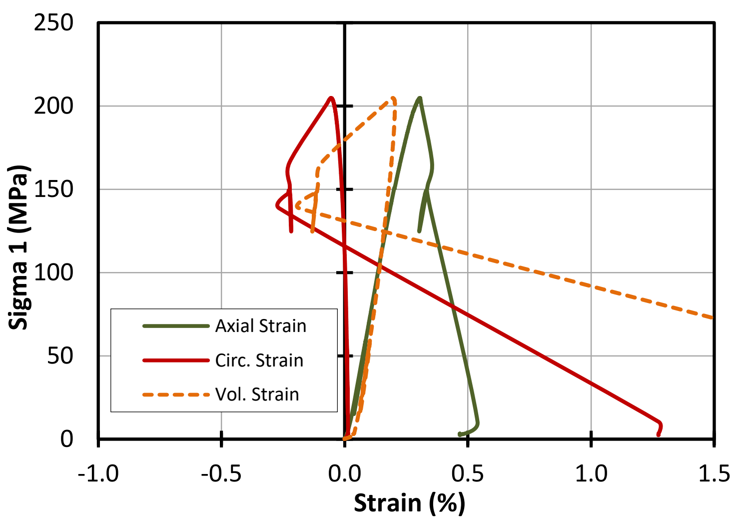
Specimen ID:	**MA-GT-19-05-T1**	**Moisture Condition:**	as-received
Depth (m):	102.48	**Test Completed on:**	30-Sep-19
Length (mm):	137.00	**Load Control:**	axial
Diameter (mm):	60.64	**Loading Rate (mm/s):**	0.0008

Confinement (MPa) **3.0**

Test Results:

Peak Strength (MPa)	204.8
Axial Strain at Peak (%)	0.305
Maximum Volumetric Strain (%)	0.204
Young's Modulus (GPa)[1]	81.6
Poisson's Ratio[1]	0.17

Stress - Strain Curves



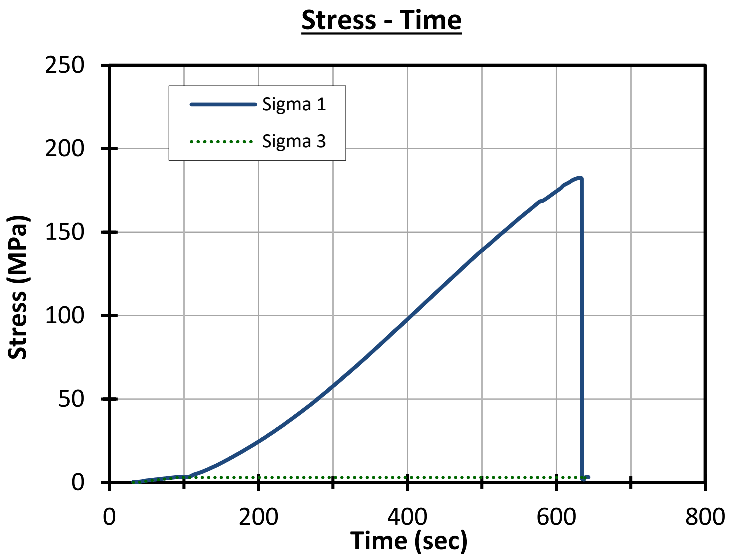
Failure Description: - multiple shear, primary undulating shear at 70^{o} from bottom

Notes:

1) Average slope of linear portion of axial stress-strain curve, b/w 40-60 % peak strength

2) Calculated over the same range of peak strength as E

Triaxial Compression Strength Test: MA-GT-19-05-T2



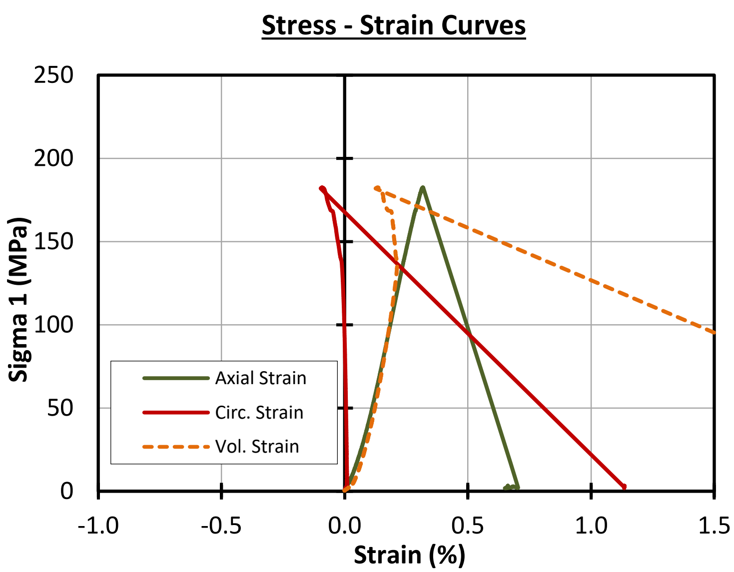
Test and Specimen Data:

Specimen ID:	**MA-GT-19-05-T2**	**Moisture Condition:**	as-received
Depth (m):	227.59	**Test Completed on:**	1-Oct-19
Length (mm):	137.04	**Load Control:**	axial
Diameter (mm):	60.45	**Loading Rate (mm/s):**	0.0008

Confinement (MPa) **3.0**

Test Results:

Peak Strength (MPa)	182.5
Axial Strain at Peak (%)	0.318
Maximum Volumetric Strain (%)	0.214
Young's Modulus (GPa)[1]	70.6
Poisson's Ratio[1]	0.12



Failure Description: - planar shear (60°)

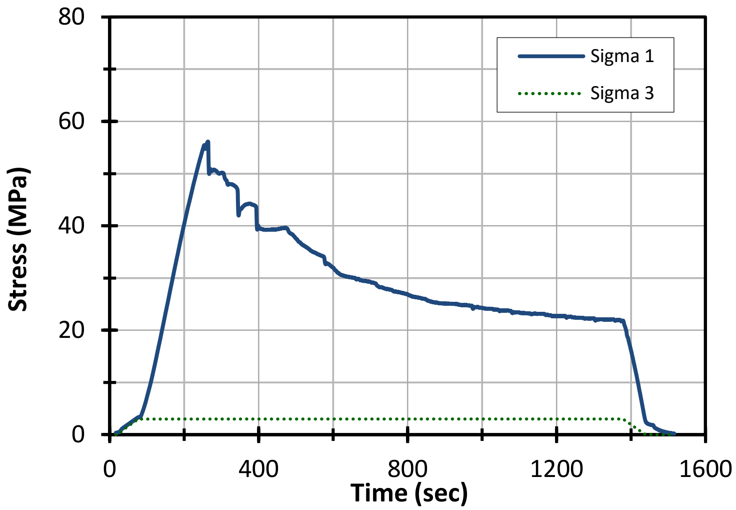
Notes:

1) Average slope of linear portion of axial stress-strain curve, b/w 40-60 % peak strength

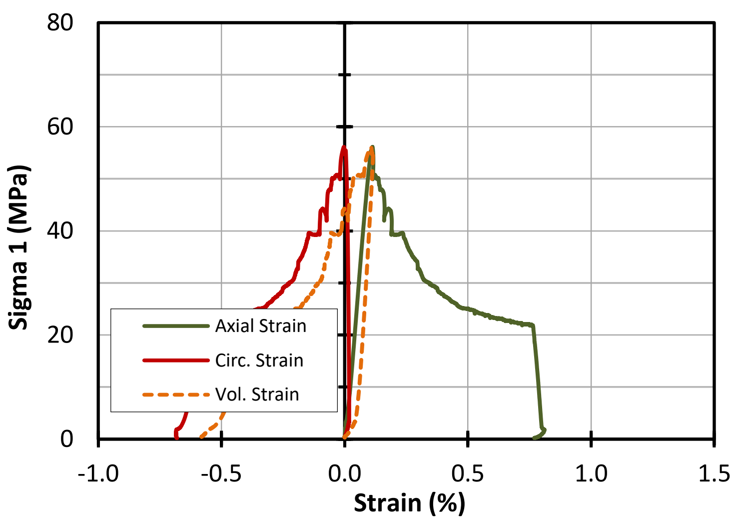
2) Calculated over the same range of peak strength as E

Triaxial Compression Strength Test: MA-GT-19-06-T1

Stress - Time



Stress - Strain Curves



Test and Specimen Data:

Specimen ID:	**MA-GT-19-06-T1**	**Moisture Condition:**	as-received
Depth (m):	99.75	**Test Completed on:**	1-Oct-19
Length (mm):	136.99	**Load Control:**	axial
Diameter (mm):	60.39	**Loading Rate (mm/s):**	0.0008

Confinement (MPa) **3.0**

Test Results:

Peak Strength (MPa)	56.1
Axial Strain at Peak (%)	0.113
Maximum Volumetric Strain (%)	0.116
Young's Modulus (GPa)[1]	59.7
Poisson's Ratio[1]	0.12

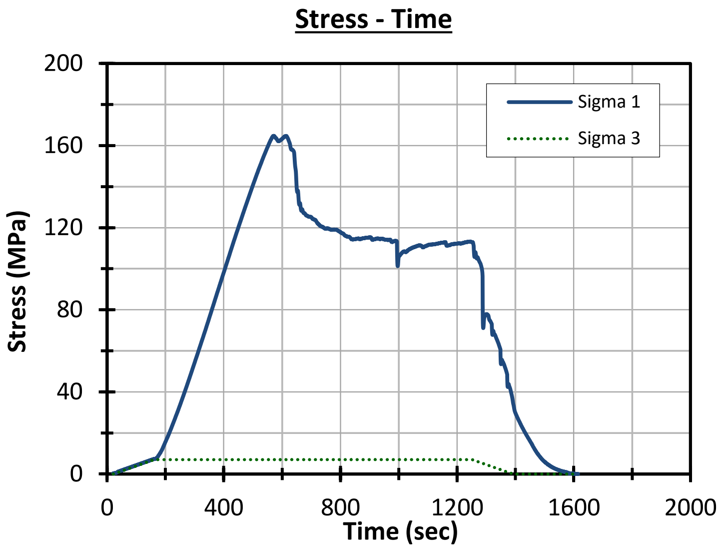
Failure Description: - shear along structure/foliation (60^{o}), from bottom

Notes:

1) Average slope of linear portion of axial stress-strain curve, b/w 40-60 % peak strength

2) Calculated over the same range of peak strength as E

Triaxial Compression Strength Test: MA-GT-19-07-T1



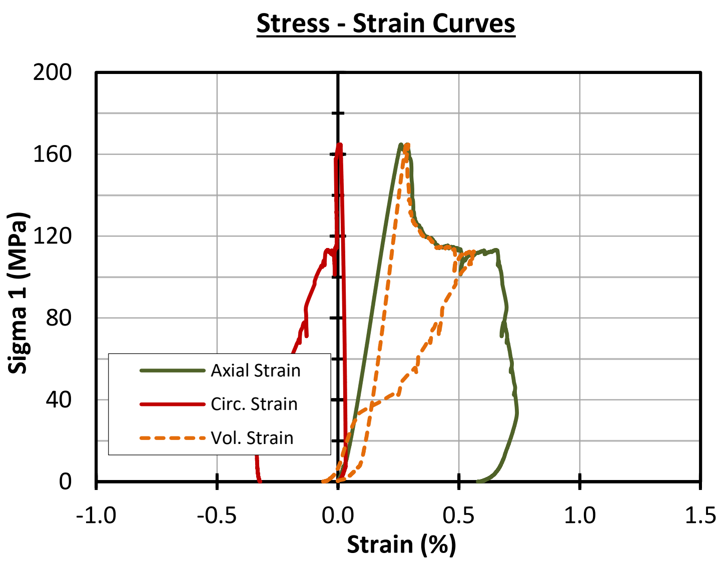
Test and Specimen Data:

Specimen ID:	**MA-GT-19-07-T1**	**Moisture Condition:**	as-received
Depth (m):	363.7	**Test Completed on:**	1-Oct-19
Length (mm):	137.05	**Load Control:**	axial
Diameter (mm):	60.86	**Loading Rate (mm/s):**	0.0008

Confinement (MPa) **7.0**

Test Results:

Peak Strength (MPa)	164.8
Axial Strain at Peak (%)	0.261
Maximum Volumetric Strain (%)	0.282
Young's Modulus (GPa)[1]	76.3
Poisson's Ratio[1]	0.09



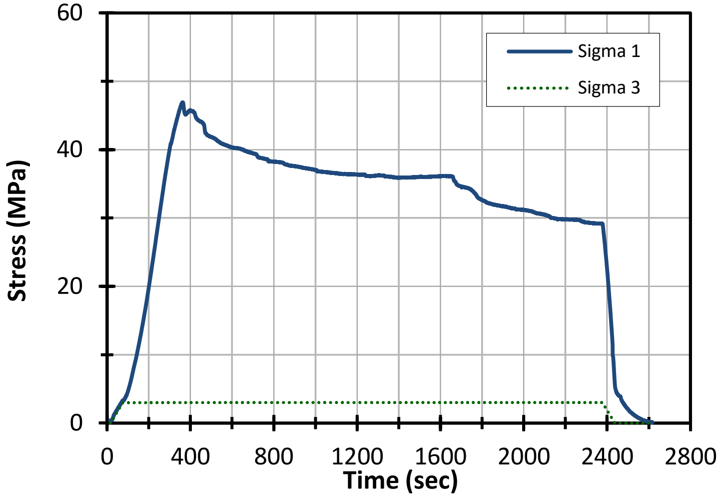
Failure Description: - planar shear (35^{o}) with multiple failure along internal defects

Notes:

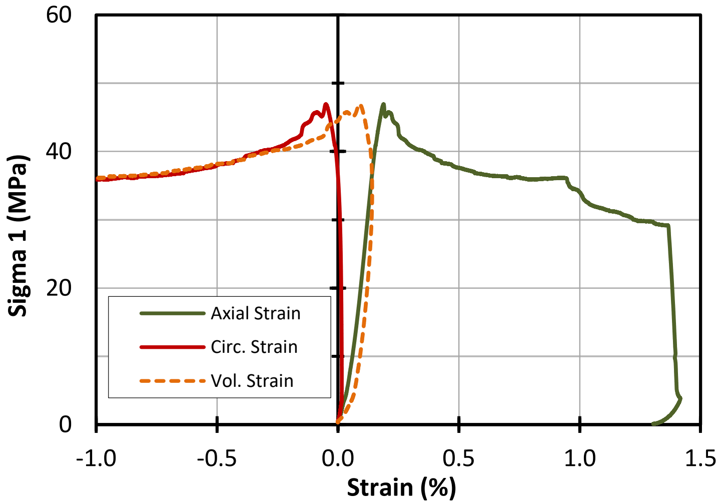
1) Average slope of linear portion of axial stress-strain curve, b/w 40-60 % peak strength
2) Calculated over the same range of peak strength as E

Triaxial Compression Strength Test: VL-GT-19-01-T1

Stress - Time



Stress - Strain Curves



Test and Specimen Data:

Specimen ID:	**VL-GT-19-01-T1**	**Moisture Condition:**	as-received
Depth (m):	228.11	**Test Completed on:**	2-Oct-19
Length (mm):	136.91	**Load Control:**	axial
Diameter (mm):	60.98	**Loading Rate (mm/s):**	0.0008

Confinement (MPa) **3.0**

Test Results:

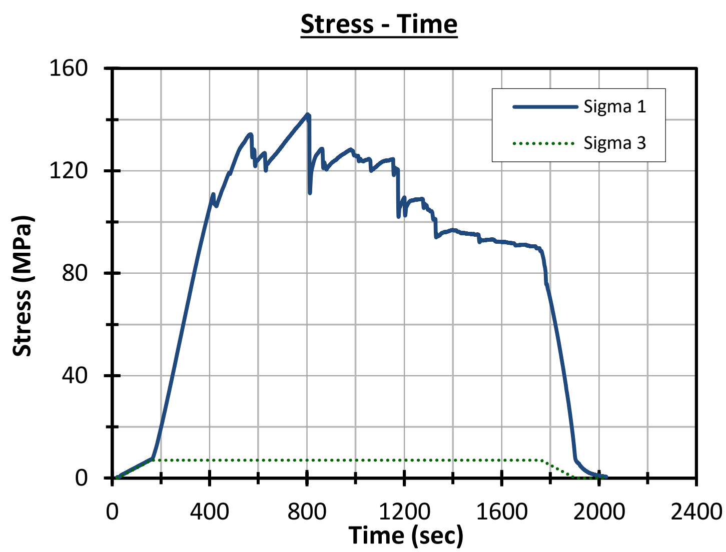
Peak Strength (MPa)	46.9
Axial Strain at Peak (%)	0.189
Maximum Volumetric Strain (%)	0.142
Young's Modulus (GPa)[1]	35.2
Poisson's Ratio[1]	0.15

Failure Description: - multiple massive shear, internal defects/micro-structure

Notes:

1) Average slope of linear portion of axial stress-strain curve, b/w 40-60 % peak strength
2) Calculated over the same range of peak strength as E

Triaxial Compression Strength Test: VL-GT-19-02-T1



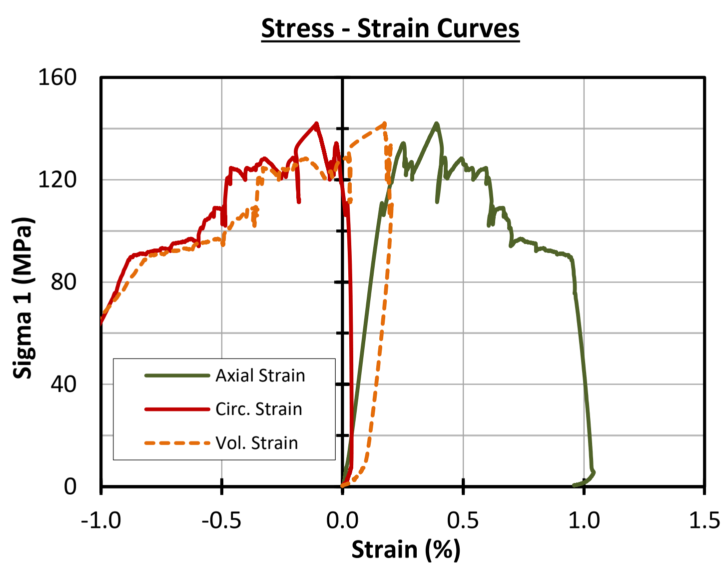
Test and Specimen Data:

Specimen ID:	**VL-GT-19-02-T1**	**Moisture Condition:**	as-received
Depth (m):	141.78	**Test Completed on:**	2-Oct-19
Length (mm):	136.91	**Load Control:**	axial
Diameter (mm):	60.49	**Loading Rate (mm/s):**	0.0008

Confinement (MPa) **7.0**

Test Results:

Peak Strength (MPa)	134.2 *
Axial Strain at Peak (%)	0.252
Maximum Volumetric Strain (%)	0.205
Young's Modulus (GPa)[1]	76.8
Poisson's Ratio[1]	0.10



Failure Description: - planar shear (80^{o}) along structure, axial splitting

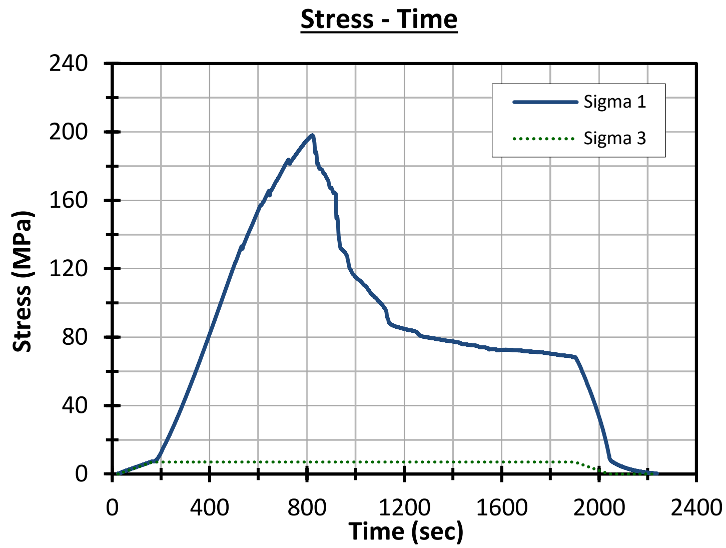
Notes:

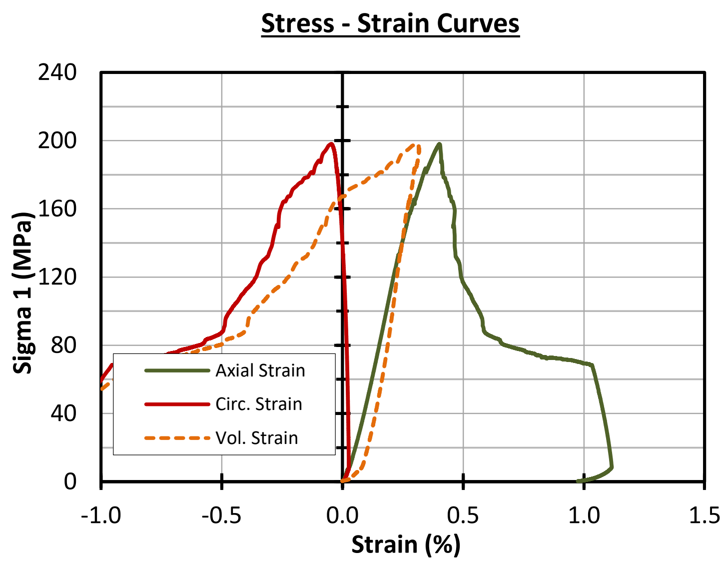
* Peak strength estimated at first 'significant' failure event

1) Average slope of linear portion of axial stress-strain curve, b/w 40-60 % peak strength

2) Calculated over the same range of peak strength as E

Triaxial Compression Strength Test: VL-GT-19-02-T2





Test and Specimen Data:

Specimen ID:	**VL-GT-19-02-T2**	**Moisture Condition:**	as-received
Depth (m):	349.61	**Test Completed on:**	2-Oct-19
Length (mm):	136.90	**Load Control:**	axial
Diameter (mm):	60.42	**Loading Rate (mm/s):**	0.0008

Confinement (MPa) **7.0**

Test Results:

Peak Strength (MPa)	198.0
Axial Strain at Peak (%)	0.402
Maximum Volumetric Strain (%)	0.317
Young's Modulus (GPa)[1]	68.3
Poisson's Ratio[1]	0.17

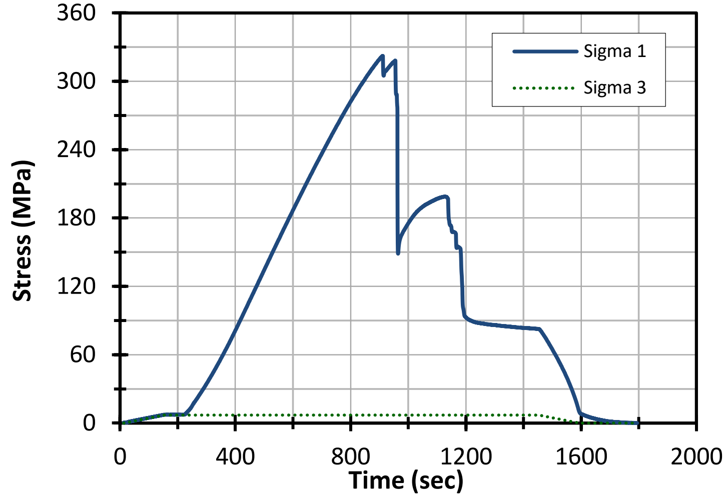
Failure Description: - multiple shear with primary shear at 70°

Notes:

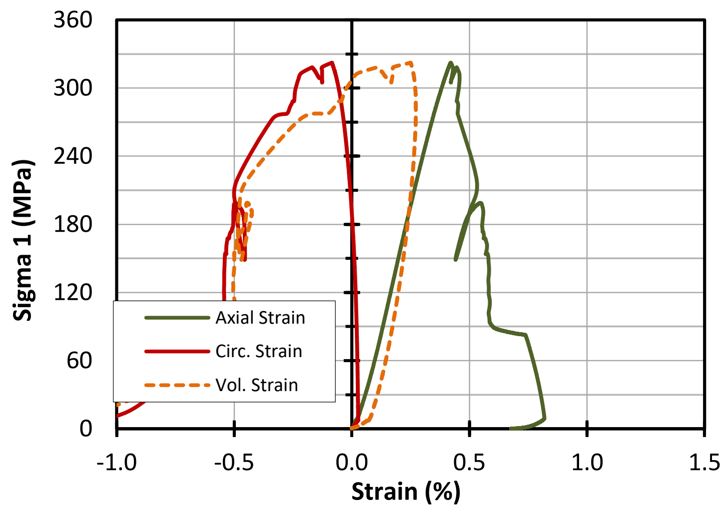
1) Average slope of linear portion of axial stress-strain curve, b/w 40-60 % peak strength
2) Calculated over the same range of peak strength as E

Triaxial Compression Strength Test: VL-GT-19-04-T1

Stress - Time



Stress - Strain Curves



Test and Specimen Data:

Specimen ID:	**VL-GT-19-04-T1**	**Moisture Condition:**	as-received
Depth (m):	75.1	**Test Completed on:**	2-Oct-19
Length (mm):	137.00	**Load Control:**	axial
Diameter (mm):	60.60	**Loading Rate (mm/s):**	0.0008

Confinement (MPa) **7.0**

Test Results:

Peak Strength (MPa)	322.3
Axial Strain at Peak (%)	0.421
Maximum Volumetric Strain (%)	0.272
Young's Modulus (GPa)[1]	89.9
Poisson's Ratio[1]	0.22

Failure Description: - multiple shear

Notes:

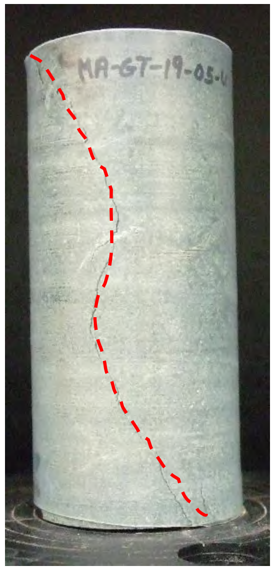
1) Average slope of linear portion of axial stress-strain curve, b/w 40-60 % peak strength
2) Calculated over the same range of peak strength as E

Attachment C – Photographs of Compression Test Specimens

Uniaxial Compression Test – Sample Photographs

MA-GT-19-05-U1
(uniaxial compression)

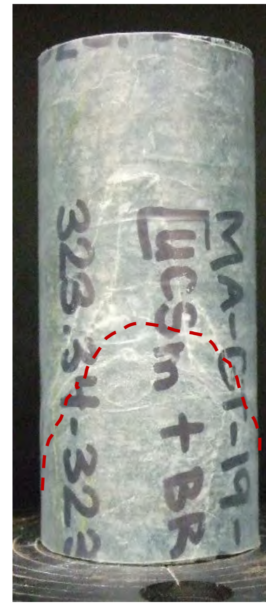
Pre-test



Post-test

MA-GT-19-05-U2
(uniaxial compression)

Pre-test



Post-test

MA-GT-19-06-U1
(uniaxial compression)

Pre-test

Post-test

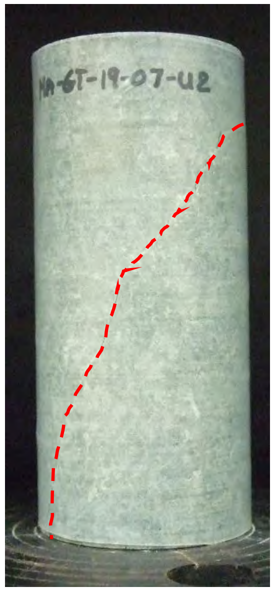
MA-GT-19-07-U1
(uniaxial compression)

Pre-test

Post-test

MA-GT-19-07-U2
(uniaxial compression)

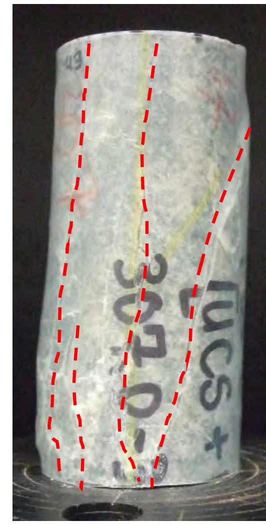
Pre-test



Post-test

MA-GT-19-07-U3
(uniaxial compression)

Pre-test



Post-test

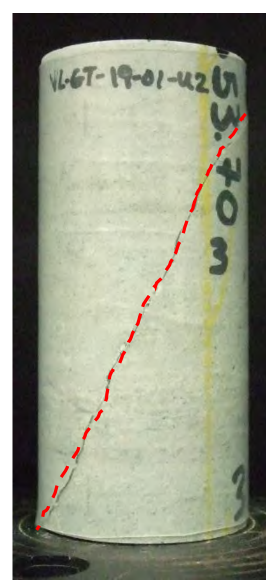
VL-GT-19-01-U1
(uniaxial compression)

Pre-test

Post-test

VL-GT-19-01-U2
(uniaxial compression)

Pre-test



Post-test

VL-GT-19-02-U1
(uniaxial compression)

Pre-test

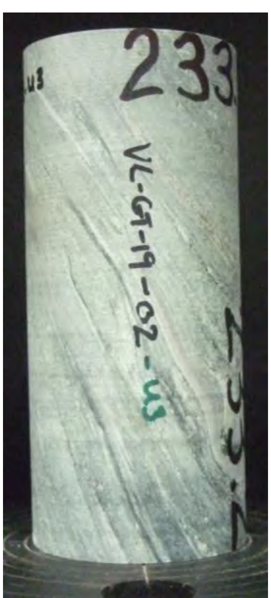
Post-test

VL-GT-19-02-U2
(uniaxial compression)

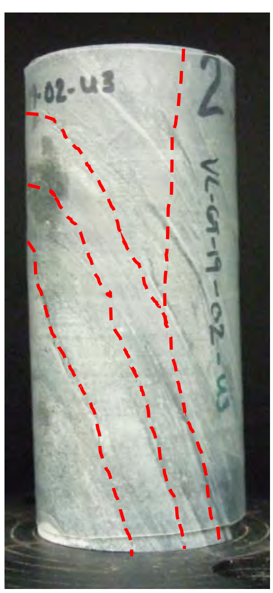
Pre-test

Post-test

VL-GT-19-02-U3
(uniaxial compression)



Pre-test



Post-test

VL-GT-19-03-U1
(uniaxial compression)

Pre-test

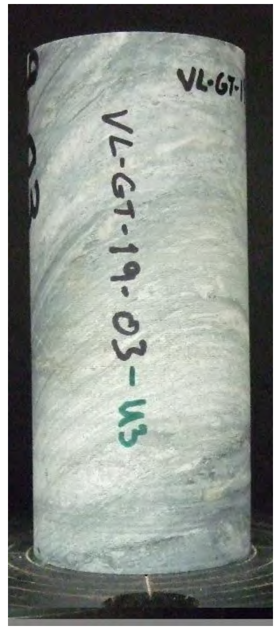
Post-test

VL-GT-19-03-U2
(uniaxial compression)

Pre-test

Post-test

VL-GT-19-03-U3
(uniaxial compression)

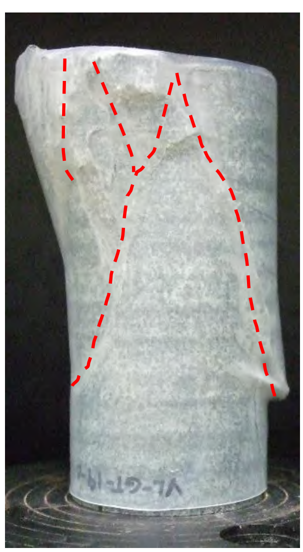


Pre-test

Post-test

VL-GT-19-04-U1
(uniaxial compression)

Pre-test



Post-test

VL-GT-19-04-U2
(uniaxial compression)

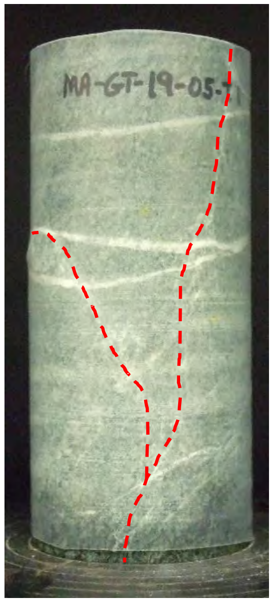
Pre-test

Post-test

Triaxial Compression Test – Sample Photographs

MA-GT-19-05-T1
(triaxial compression)

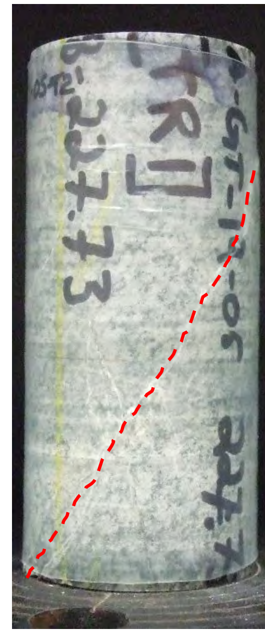
Pre-test



Post-test

MA-GT-19-05-T2
(triaxial compression)

Pre-test



Post-test

MA-GT-19-06-T1
(triaxial compression)

Pre-test

Post-test

MA-GT-19-07-T1
(triaxial compression)

Pre-test

Post-test

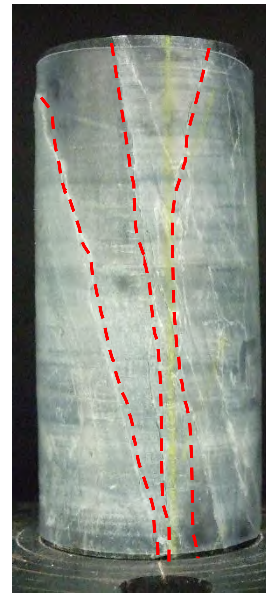
VL-GT-19-01-T1
(triaxial compression)

Pre-test

Post-test

VL-GT-19-02-T1
(triaxial compression)

Pre-test



Post-test

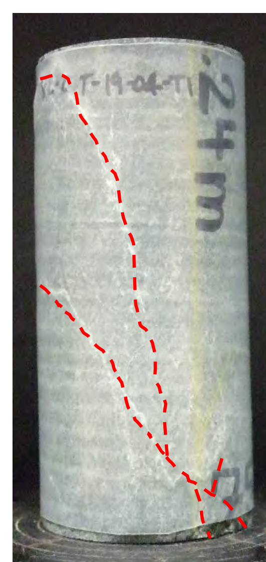
VL-GT-19-02-T2
(triaxial compression)

Pre-test

Post-test

VL-GT-19-04-T1
(triaxial compression)

Pre-test

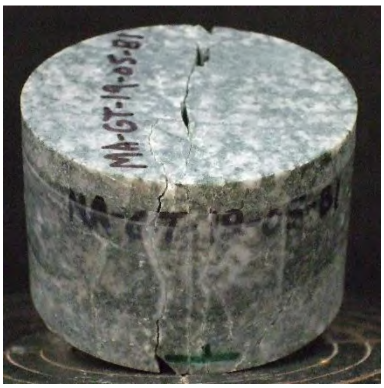


Post-test

Attachment D – Photographs of Brazilian Test Specimens

MA-GT-19-05-B1

Pre-test



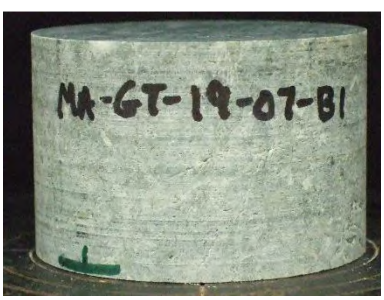
Post-test

MA-GT-19-06-B1

Pre-test

Post-test

MA-GT-19-07-B1



Pre-test

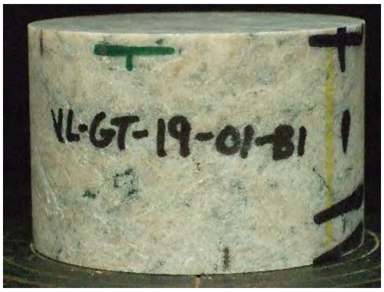
Post-test

MA-GT-19-07-B2

Pre-test

Post-test

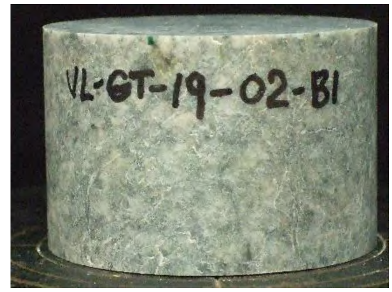
VL-GT-19-01-B1



Pre-test

Post-test

VL-GT-19-02-B1



Pre-test

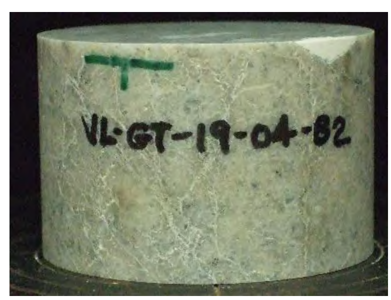
Post-test

VL-GT-19-04-B1

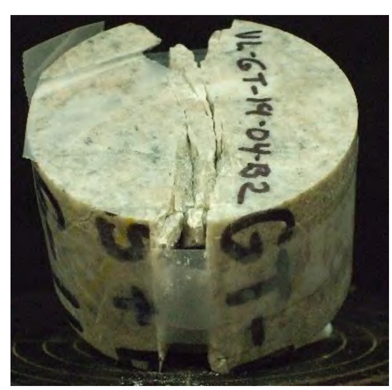
Pre-test

Post-test

VL-GT-19-04-B2



Pre-test



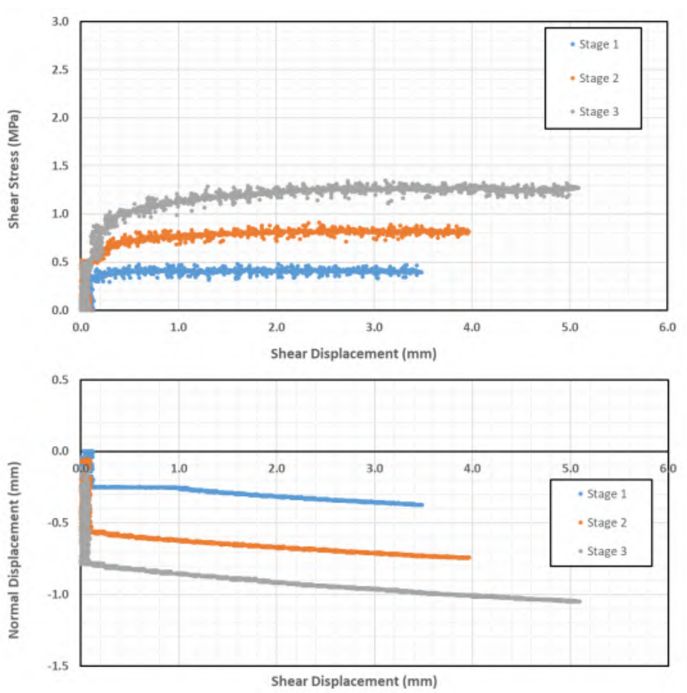
Post-test

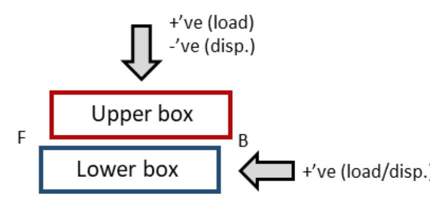
Attachment E – Direct Shear Test Results

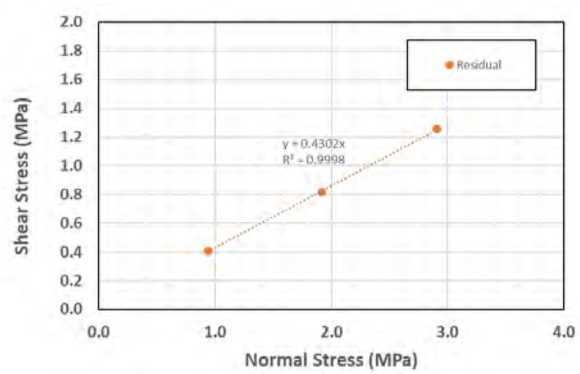
Direct Shear Test: MA-GT-19-06-DS1 (56.39 m)

Specimen and Test Information

Sample ID:	MA-GT-19-06-DS1	**Length (mm):**	87.7
Depth:	56.39 m	**Width (mm):**	60.5
Discontinuity Description:	natural joint, dip = 40° (to core axis), smooth waxy surface with faint lineations, tested parallel to lineation	**Initial Area (mm^2):**	4167
Initial Joint Roughness (JRC):	0 - 2		
Test Type:	Multi-stage (3) with re-positioning		







Results

	Stage 1	*Stage 2*	*Stage 3*
Avg. Normal Load (kN)	3.92	8.00	12.11
Avg. Normal Stress (MPa)[1]	0.94	1.92	2.91
Peak Shear Stress (MPa)[2]	--	--	--
Residual Shear Stress (MPa)	0.41	0.82	1.25

Notes:

1 - based on initial surface area of discontinuity - uncorrected during test

2 - no peak shear strength identified

Interpretation (Mohr-Coulomb)

Peak FA =	--	degrees
Peak C' =	--	MPa
Residual FA =	23.3	degrees

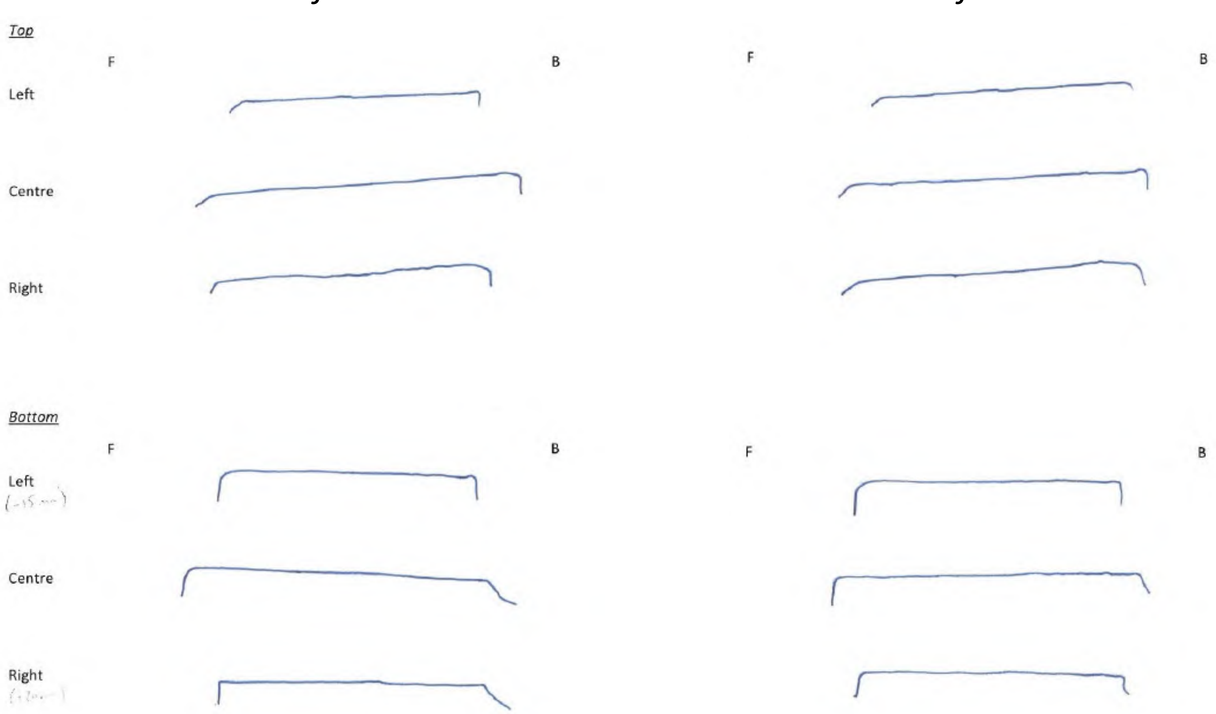
Direct Shear Test: MA-GT-19-06-DS1 (56.39 m)

Photos

	Before Test		*After Test*
Upper		*B* *F*	
Lower		*B* *F*	

Surface Profiles

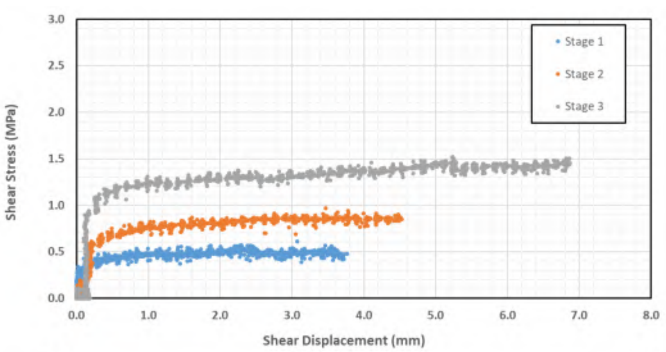
Before Test *After Test*

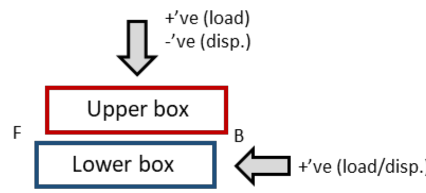


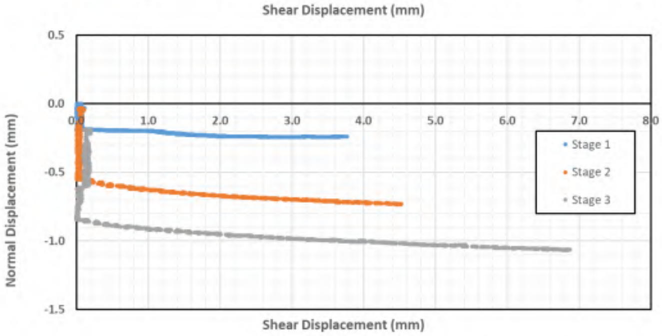
Direct Shear Test: MA-GT-19-06-DS2 (157.59 m)

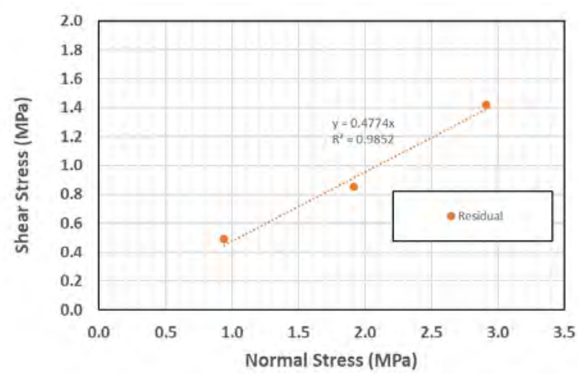
Specimen and Test Information

Sample ID:	MA-GT-19-06-DS2	**Length (mm):**	86.4
Depth:	157.59 m	**Width (mm):**	60.5
Discontinuity Description:	natural joint, dip angle ~ 43°, well defined slickenside lineations at 25° to dip dir., undulating normal to lineation, tested parallel to lineation	**Initial Area (mm^2):**	4105
Initial Joint Roughness (JRC):	0 - 2		
Test Type:	Multi-stage (3) with re-positioning		









Results

	Stage 1	*Stage 2*	*Stage 3*
Avg. Normal Load (kN)	3.88	7.92	12.01
Avg. Normal Stress (MPa)[1]	0.94	1.92	2.91
Peak Shear Stress (MPa)[2]	--	--	--
Residual Shear Stress (MPa)	0.49	0.85	1.42

Notes:

1 - based on initial surface area of discontinuity - uncorrected during test

2 - no peak shear strength identified

Interpretation (Mohr-Coulomb)

Peak FA =	--	degrees
Peak C' =	--	MPa
Residual FA =	25.5	degrees

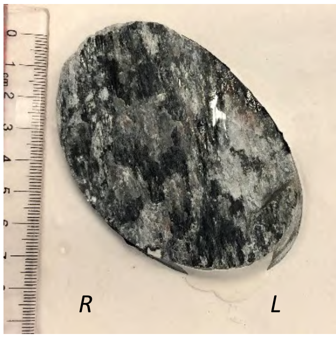
Direct Shear Test: MA-GT-19-06-DS2 (157.59 m)

Photos

Before Test | *After Test*

B

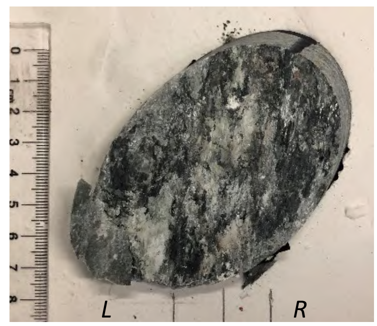
Upper



F

B

Lower



F

Surface Profiles

Top

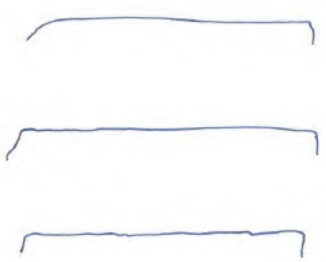
Before Test | *After Test*

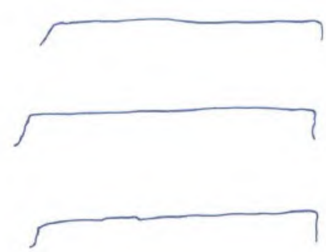
F B | F B

Left

Centre

Right





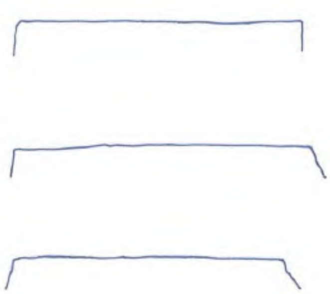
Bottom

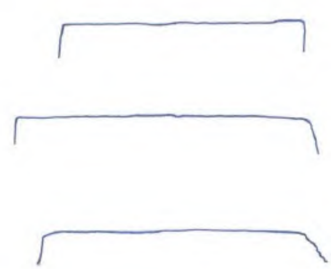
F B | F B

Left

Centre

Right





Direct Shear Test: MA-GT-19-06-DS3 (76.41 m)

Specimen and Test Information

Sample ID: MA-GT-19-06-DS3

Length (mm): 93.8

Depth: 76.41 m

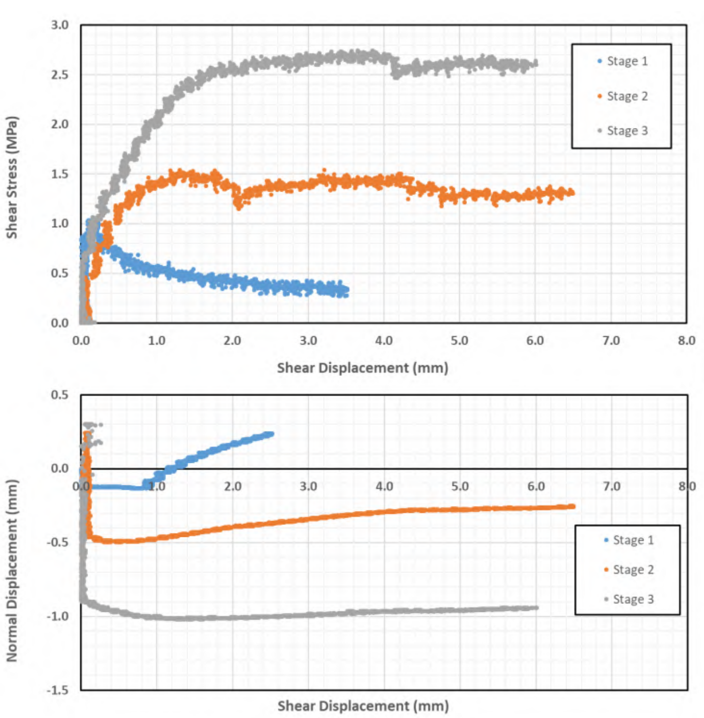
Width (mm): 60.8

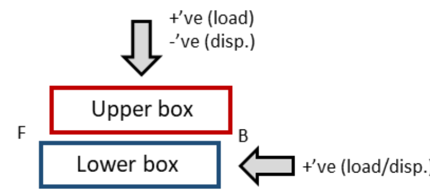
Test Type: Multi-stage (3) with re-positioning

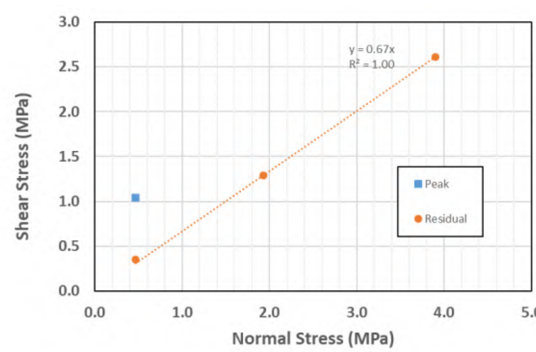
Initial Area (mm^2): 4479

Initial Joint Roughness (JRC): 4 - 6

Discontinuity Description: natural joint, dip angle ~ 40° along foliation, tested along dip direction







Results

	Stage 1	*Stage 2*	*Stage 3*
Avg. Normal Load (kN)	2.10	8.63	17.48
Avg. Normal Stress (MPa)[1]	0.47	1.93	3.90
Peak Shear Stress (MPa)[2]	1.04	--	--
Residual Shear Stress (MPa)	0.35	1.29	2.61

Notes:

1 - based on initial surface area of discontinuity - uncorrected during test

2 - asperities are damaged following the initial shear stage

Interpretation (Mohr-Coulomb)

Peak FA =	--	degrees
Peak C' =	--	MPa
Residual FA =	33.8	degrees

Direct Shear Test: MA-GT-19-06-DS3 (76.41 m)

Photos

Before Test | *After Test*

Upper

B

F

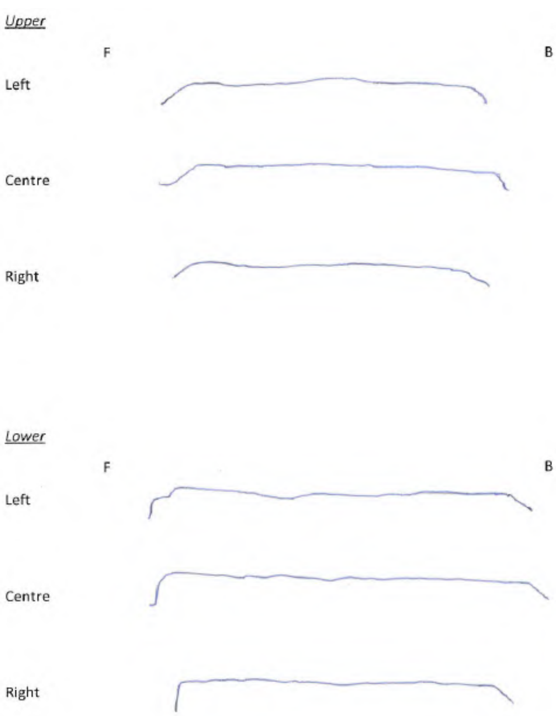
Lower

B

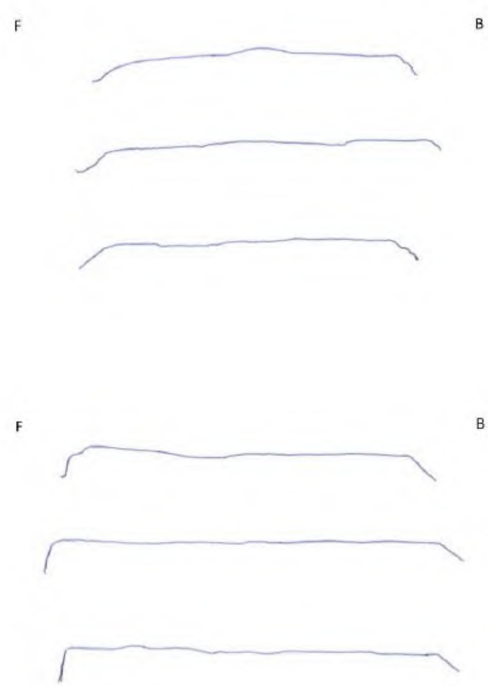
F

Surface Profiles

Before Test



After Test



Direct Shear Test: MA-GT-19-06-DS4 (202.12 m)

Specimen and Test Information

Sample ID: MA-GT-19-06-DS4

Depth: 202.12 m

Test Type: Multi-stage (3) with re-positioning

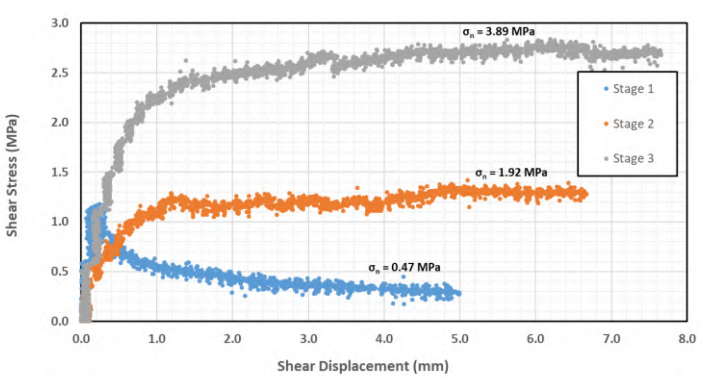
Initial Joint Roughness (JRC): 6 - 8

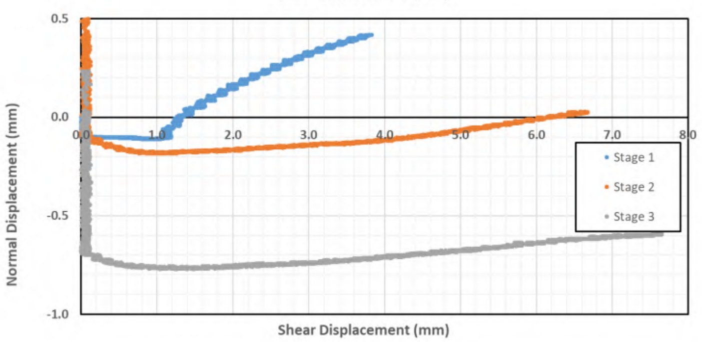
Discontinuity Description: natural joint, dip angle ~45° to core axis, small steps/ridges, waxy coating on surface

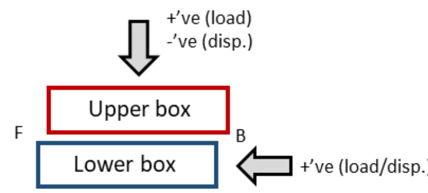
Length (mm): 80.5

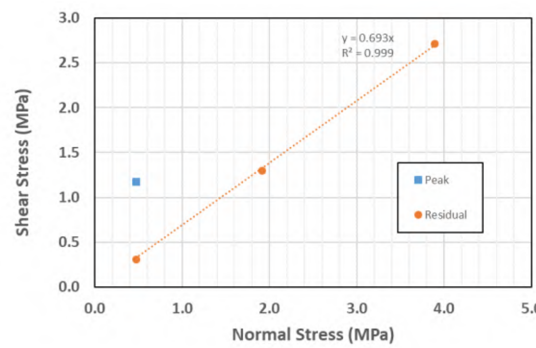
Width (mm): 60.7

Initial Area (mm^2): 3838









Results

	Stage 1	*Stage 2*	*Stage 3*
Avg. Normal Load (kN)	1.80	7.36	14.93
Avg. Normal Stress (MPa)[1]	0.47	1.92	3.89
Peak Shear Stress (MPa)[2]	1.18	--	--
Residual Shear Stress (MPa)	0.31	1.30	2.72

Notes:

1 - based on initial surface area of discontinuity - uncorrected during test

2 - asperities are damaged following the initial shear stage

Interpretation (Mohr-Coulomb)

Peak FA =	--	degrees
Peak C' =	--	MPa
Residual FA =	34.7	degrees

Direct Shear Test: MA-GT-19-06-DS4 (202.12 m)

Photos

Before Test | *After Test*

Upper

B

F

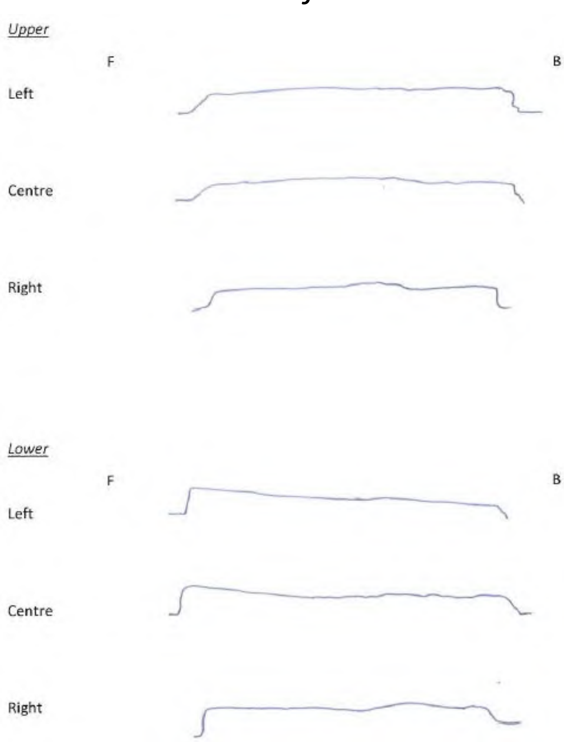
Lower

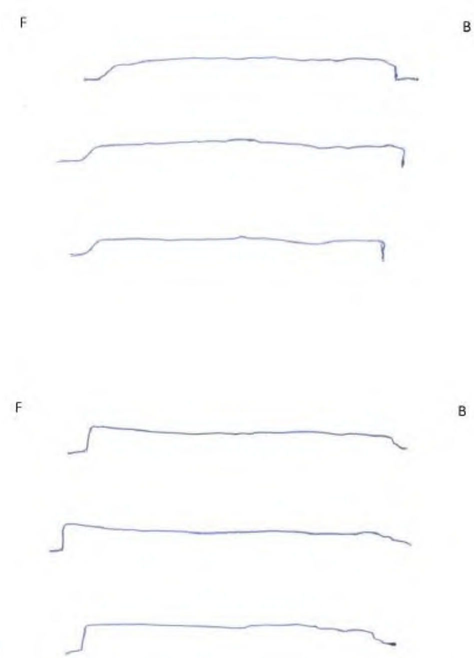
B

F

Surface Profiles

Before Test | *After Test*

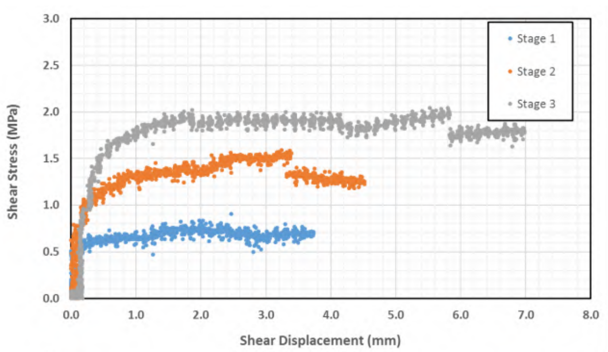


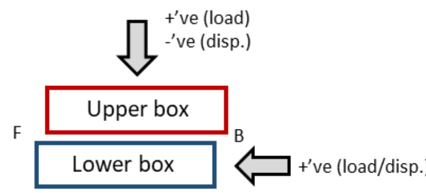


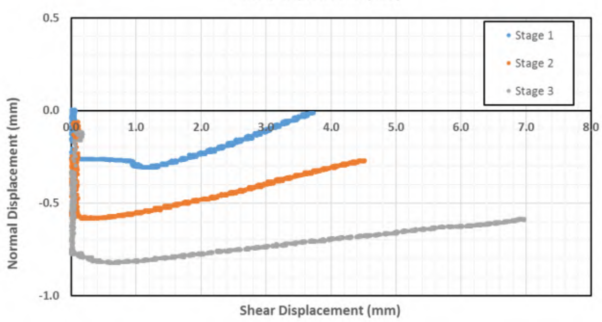
Direct Shear Test: MA-GT-19-07-DS1 (77.64 m)

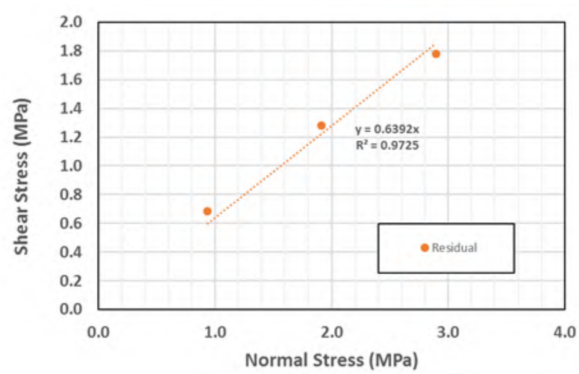
Specimen and Test Information

Sample ID:	MA-GT-19-07-DS1	**Length (mm):**	68.0
Depth:	77.64 m	**Width (mm):**	61.0
Discontinuity Description:	natural joint, dip angle ~ 58°, well defined slickenside lineations at 45° to dip dir., tested parallel to lineation	**Initial Area (mm^2):**	3258
Initial Joint Roughness (JRC):	2 - 4		
Test Type:	Multi-stage (3) with re-positioning		









Results

	Stage 1	*Stage 2*	*Stage 3*
Avg. Normal Load (kN)	3.05	6.22	9.43
Avg. Normal Stress (MPa)[1]	0.94	1.91	2.89
Peak Shear Stress (MPa)[2]	--	--	--
Residual Shear Stress (MPa)	0.69	1.28	1.78

Notes:

1 - based on initial surface area of discontinuity - uncorrected during test

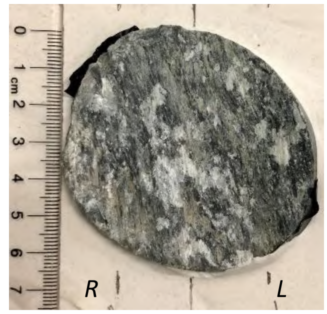
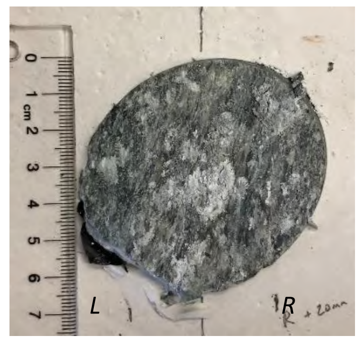
2 - no peak shear strength identified

Interpretation (Mohr-Coulomb)

Peak FA =	--	degrees
Peak C' =	--	MPa
Residual FA =	32.6	degrees

Direct Shear Test: MA-GT-19-07-DS1 (77.64 m)

Photos

	Before Test		*After Test*
		B	
Upper			
		F	
		B	
Lower			
		F	

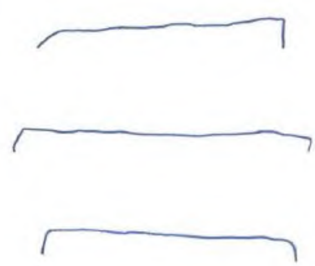
Surface Profiles

Before Test — *After Test*

Top

F B F B

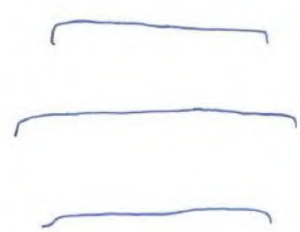
Left
Centre
Right



Bottom

F B F B

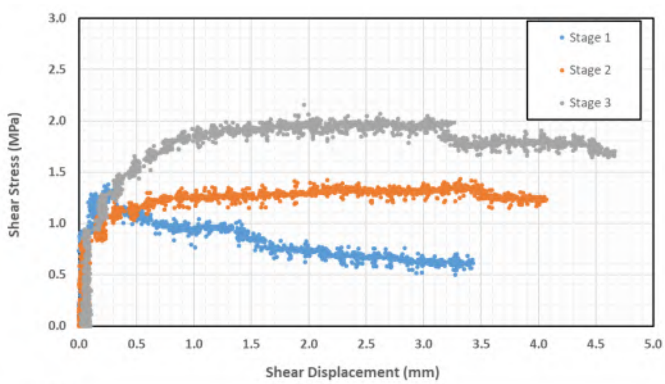
Left
Centre
Right

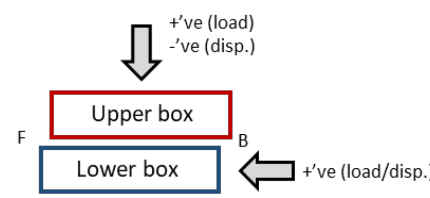


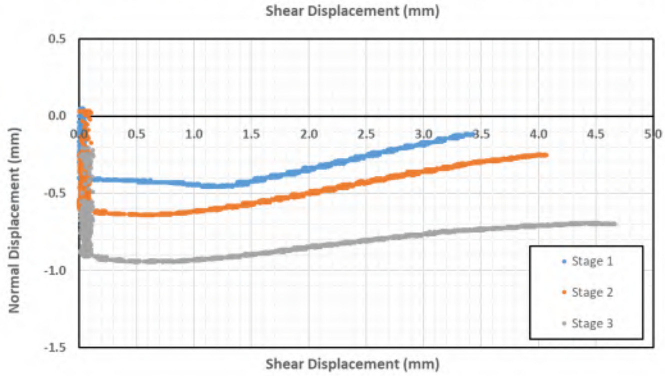
Direct Shear Test: VL-GT-19-01-DS1 (282.26 m)

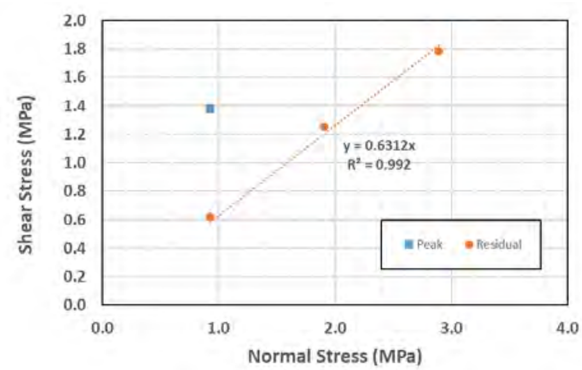
Specimen and Test Information

Sample ID:	VL-GT-19-01-DS1	**Length (mm):**	67
Depth:	282.26 m	**Width (mm):**	61
Discontinuity Description:	natural joint, rough surface, no infilling, waxy coating on joint surface, dip = 63o (to core axis)	**Initial Area (mm^2):**	3210
Initial Joint Roughness (JRC):	12 - 14		
Test Type:	Multi-stage (3) with re-positioning		









Results

	Stage 1	*Stage 2*	*Stage 3*
Avg. Normal Load (kN)	2.97	6.12	9.28
Avg. Normal Stress (MPa)[1]	0.93	1.91	2.89
Peak Shear Stress (MPa)	1.38	--	--
Residual Shear Stress (MPa)	0.62	1.25	1.78

Notes:

1 - based on initial surface area of discontinuity - uncorrected during test

Interpretation (Mohr-Coulomb)

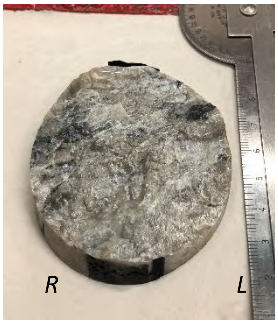
Peak FA =	--	degrees
Peak C' =	--	MPa
Residual FA =	32.3	degrees

Direct Shear Test: VL-GT-19-01-DS1 (282.26 m)

Photos

Before Test *After Test*

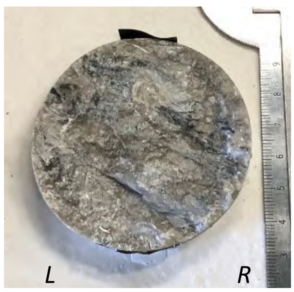
Upper



B

F

Lower

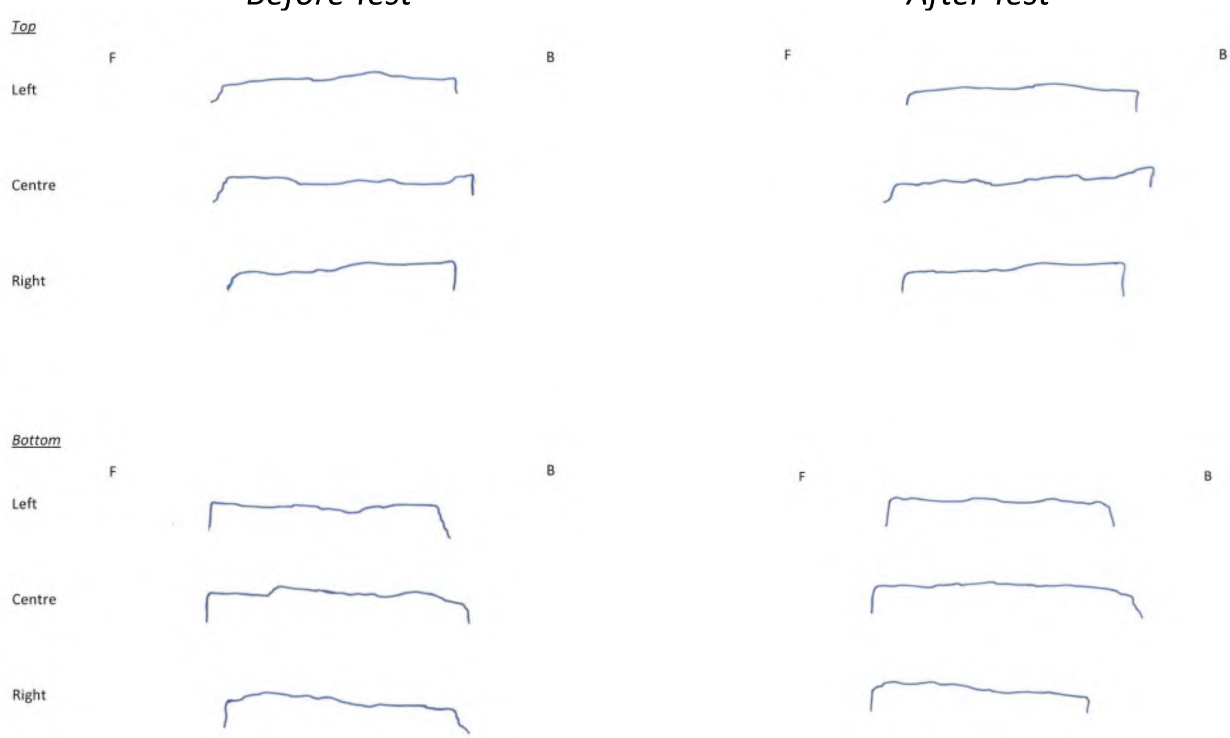


B

F

Surface Profiles

Before Test *After Test*

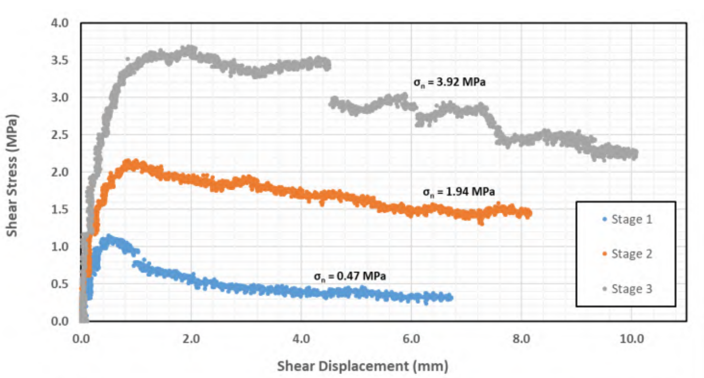


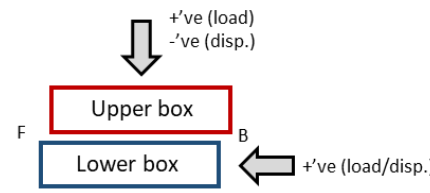
Direct Shear Test: VL-GT-19-02-DS1 (199.40 m)

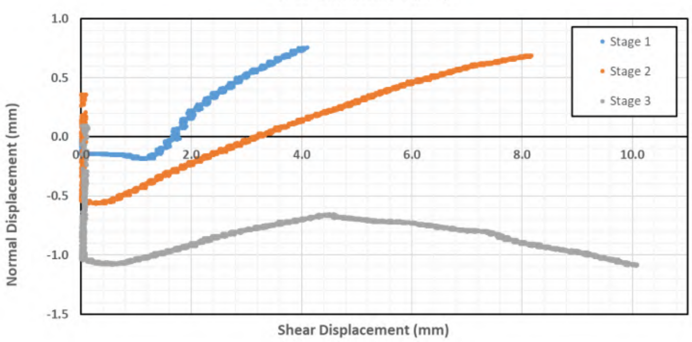
Specimen and Test Information

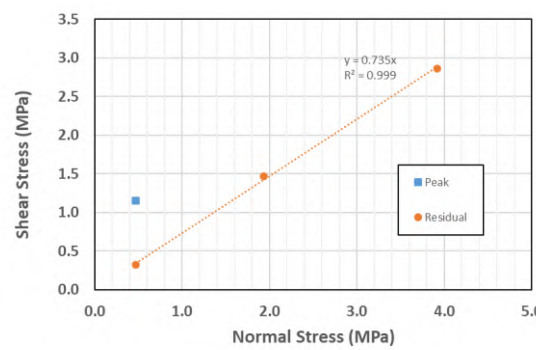
Sample ID:	VL-GT-19-02-DS1	**Length (mm):**	114.4
Depth:	199.40 m	**Width (mm):**	60.8
Test Type:	Multi-stage (3) with re-positioning	**Initial Area (mm^2):**	5463
Initial Joint Roughness (JRC):	12 - 14		

Discontinuity Description: natural joint, rough, dip angle ~30° to core axis









Results

	Stage 1	*Stage 2*	*Stage 3*
Avg. Normal Load (kN)	2.57	10.58	21.40
Avg. Normal Stress (MPa)[1]	0.47	1.94	3.92
Peak Shear Stress (MPa)[2]	1.15	--	--
Residual Shear Stress (MPa)	0.33	1.47	2.86

Notes:

1 - based on initial surface area of discontinuity - uncorrected during test

2 - asperities are damaged following the initial shear stage

Interpretation (Mohr-Coulomb)

Peak FA =	--	degrees
Peak C' =	--	MPa
Residual FA =	36.3	degrees

Direct Shear Test: VL-GT-19-02-DS1 (199.40 m)

Photos

Before Test | *After Test*

Upper

B

F

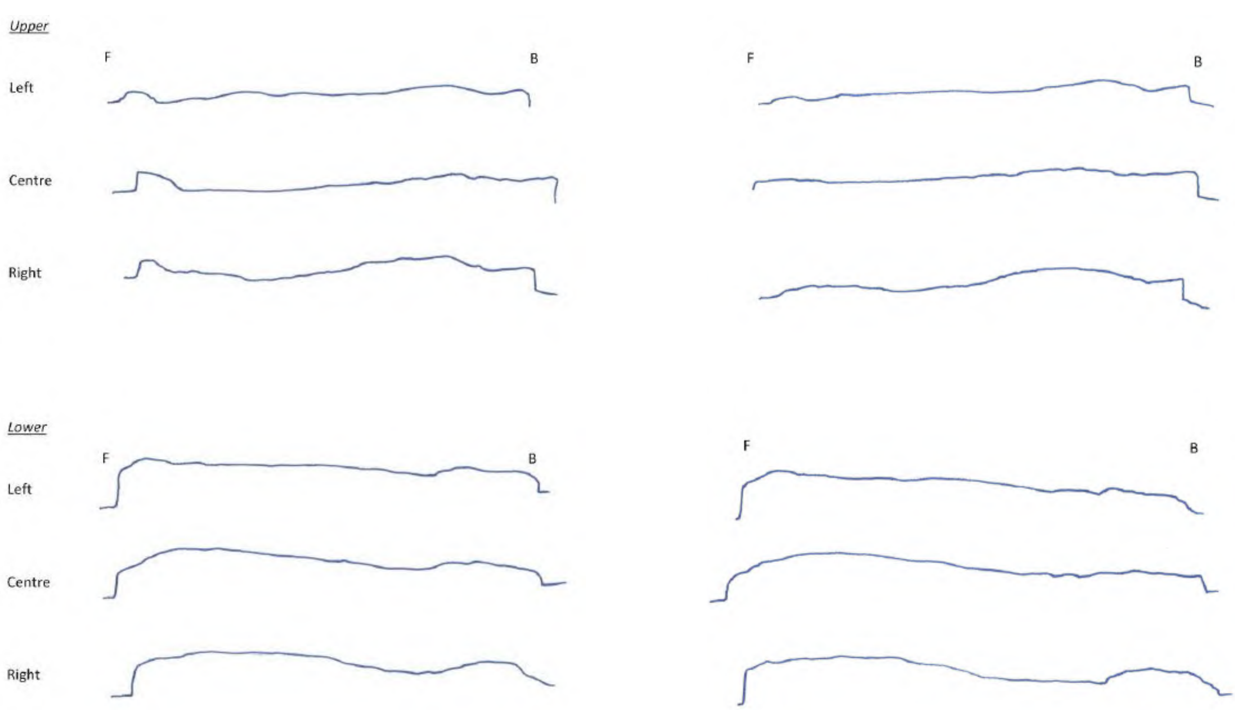
Lower

B

F

Surface Profiles

Before Test | *After Test*



Direct Shear Test: VL-GT-19-03-DS1 (54.55 m)

Specimen and Test Information

Sample ID: VL-GT-19-03-DS1

Depth: 54.55 m

Discontinuity Description: natural joint, undulating surface, waxy coating, no infilling, dip ~ 60° (to core axis)

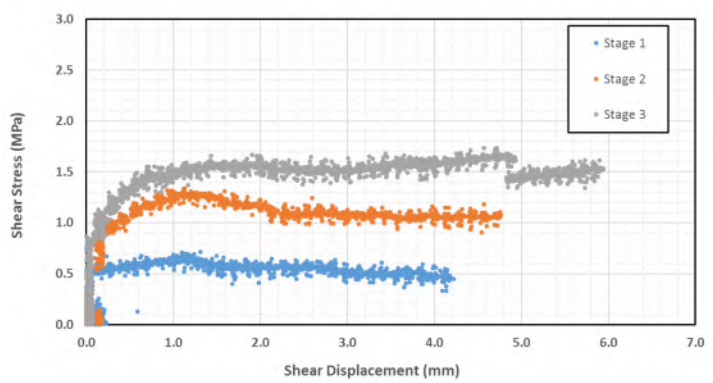
Initial Joint Roughness (JRC): 8 - 10

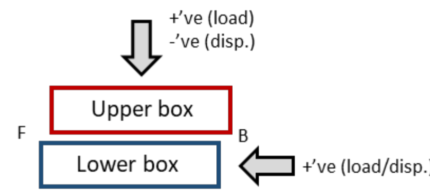
Test Type: Multi-stage (3) with re-positioning

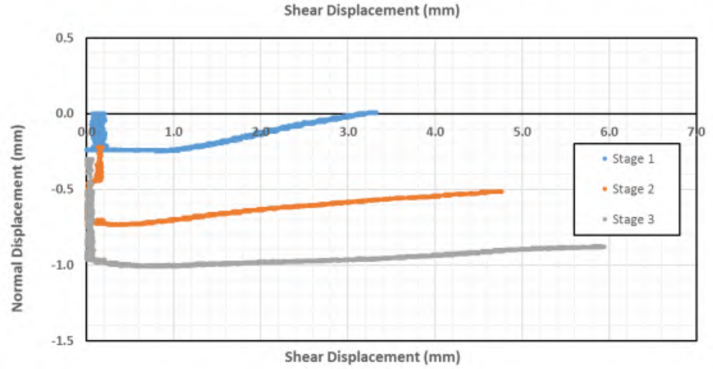
Length (mm): 67.4

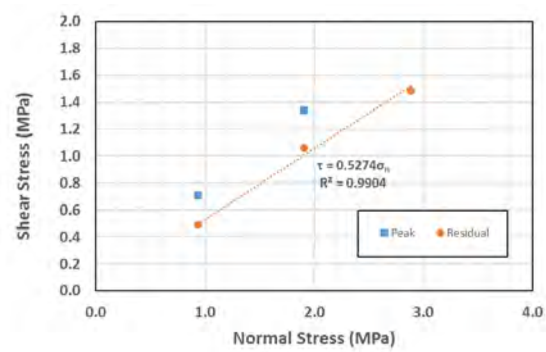
Width (mm): 61.0

Initial Area (mm²): 3229









Results

	Stage 1	*Stage 2*	*Stage 3*
Avg. Normal Load (kN)	3.03	6.15	9.31
Avg. Normal Stress (MPa)[1]	0.94	1.91	2.88
Peak Shear Stress (MPa)[2]	0.71	1.34	--
Residual Shear Stress (MPa)	0.49	1.06	1.48

Interpretation (Mohr-Coulomb)

Peak FA =	--	degrees
Peak C' =	--	MPa
Residual FA =	27.8	degrees

Notes:

1 - based on initial surface area of discontinuity - uncorrected during test

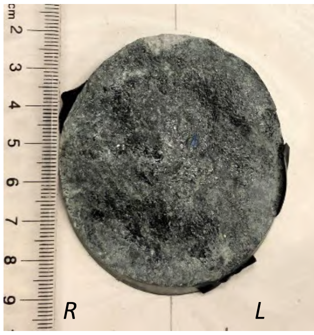
2 - surface is damaged following initial shear stage, therefore subsequent interpreted peak' values are underestimated and can result in an incorrect interpretation of discontinuity strength

Direct Shear Test: VL-GT-19-03-DS1 (54.55 m)

Photos

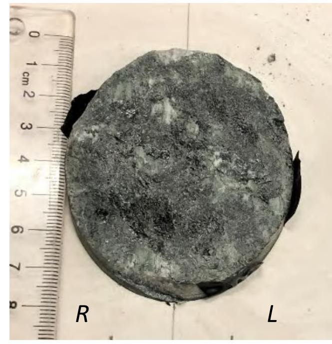
Before Test *After Test*

Upper

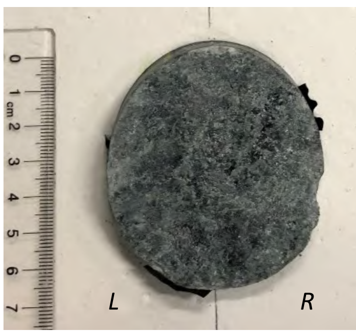


B

F

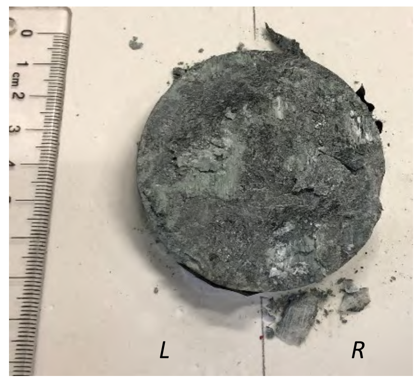


Lower



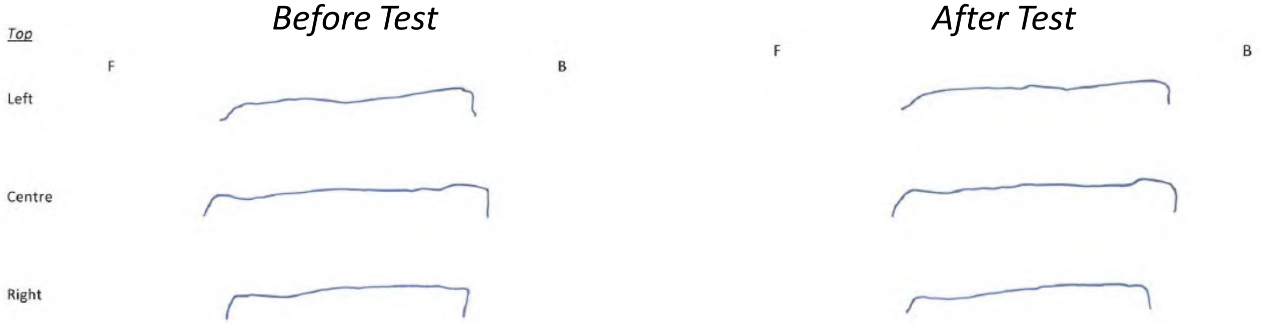
B

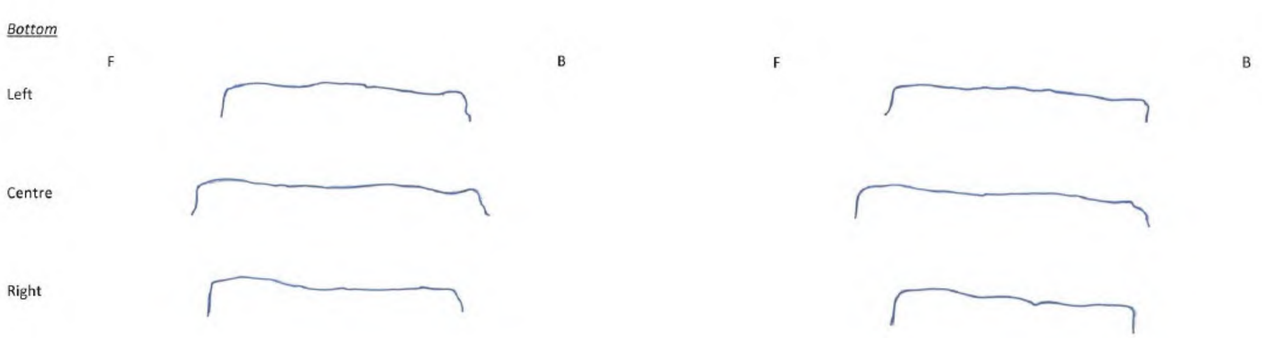
F



Surface Profiles

Before Test *After Test*





Direct Shear Test: VL-GT-19-04-DS1 (112.62 m)

Specimen and Test Information

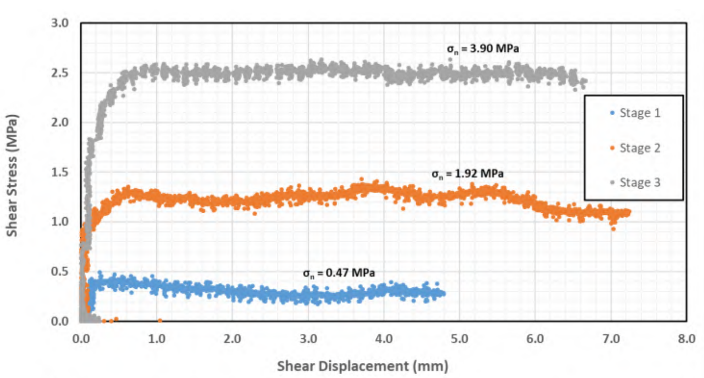
Sample ID: VL-GT-19-04-DS1 **Length (mm):** 81.7

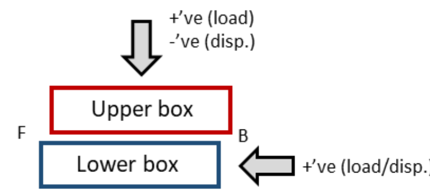
Depth: 112.62 m **Width (mm):** 60.8

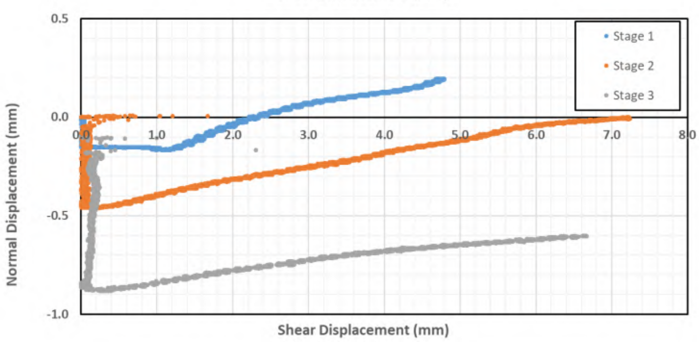
Test Type: Multi-stage (3) with re-positioning **Initial Area (mm^2):** 3901

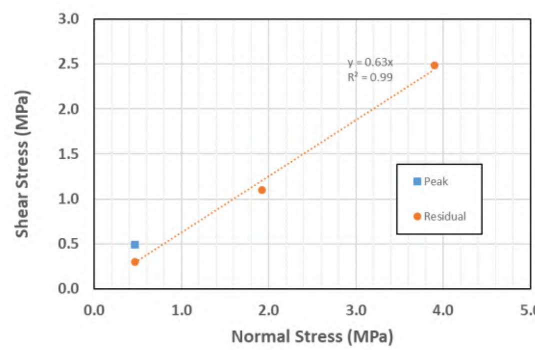
Initial Joint Roughness (JRC): 10 - 12

Discontinuity Description: natural joint, rough with minor undulation, dip angle ~ 45°









Results

	Stage 1	*Stage 2*	*Stage 3*
Avg. Normal Load (kN)	1.83	7.49	15.21
Avg. Normal Stress (MPa)[1]	0.47	1.92	3.90
Peak Shear Stress (MPa)[2]	0.49	--	--
Residual Shear Stress (MPa)	0.30	1.10	2.49

Notes:

1 - based on initial surface area of discontinuity - uncorrected during test

2 - asperities are damaged following the initial shear stage

Interpretation (Mohr-Coulomb)

Peak FA =	--	degrees
Peak C' =	--	MPa
Residual FA =	32.2	degrees

Direct Shear Test: VL-GT-19-04-DS1 (112.62 m)

Photos

Before Test | *After Test*

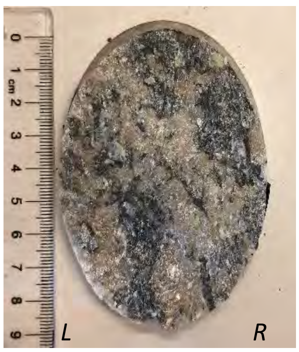
Upper

B

F

Lower

B



F

Surface Profiles

Before Test | *After Test*

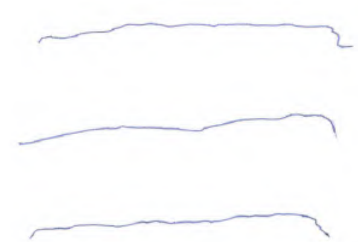
Upper

F B

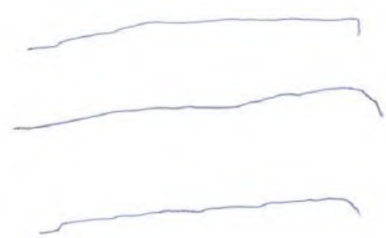
Left

Centre

Right



F B



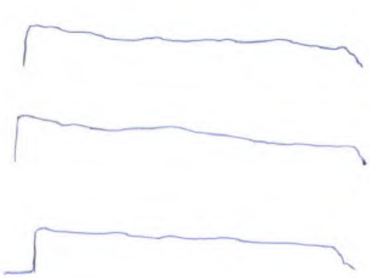
Lower

F B

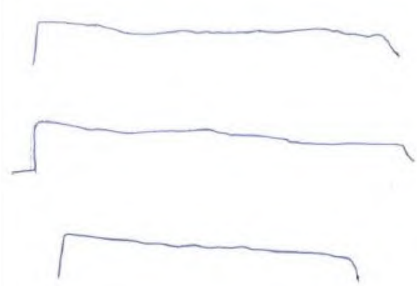
Left

Centre

Right



F B



VALENTINE GOLD PROJECT PRE-FEASIBILITY GEOTECHNICAL INVESTIGATION - MARATHON DEPOSIT

PROJECT:	Valentine Gold Project - Pre-Feasibility Geotechnical Investigation
DATE:	22-Oct-19
PROJECT No:	19-0015-H
SUBJECT:	Structural Analysis - Marathon Deposit
Prepared by:	Andrew Guest, P. Eng, Katrina Morgenroth, E.I.T.
Checked by:	Tony L. Gilman, M. Sc., P. Geo, P. Eng

NOTES:
1. Blue dot [•] signifies the drillhole collar (trend/plunge).

* All Stereonets are lower hemisphere equal angle projections

** Plots show poles to discontinuity set planes

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Figure 1 - MA-GT-19-05 - Orientated Core Structural Data

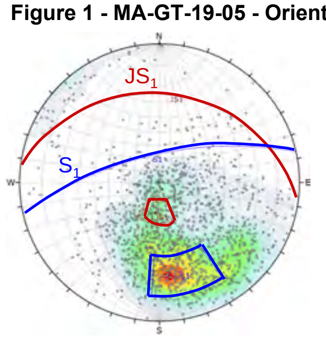

Figure 2 - MA-GT-19-06 - Orientated Core Structural Data

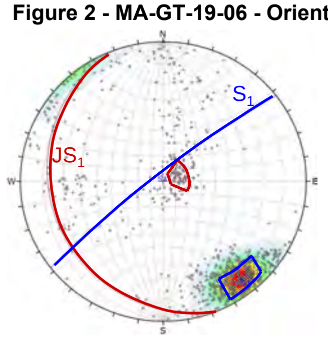

Figure 3 - MA-GT-19-07 - Orientated Core Structural Data

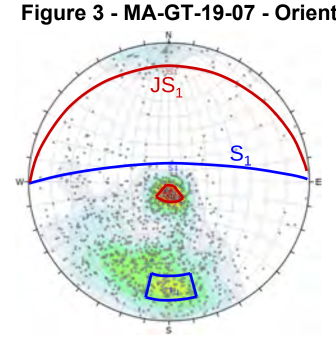

Figure 10 - Marathon Deposit - All Data - Combined

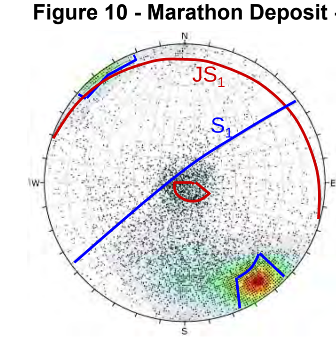

Figure 4 - MA-GT-19-05 - Televiewer Structural Data

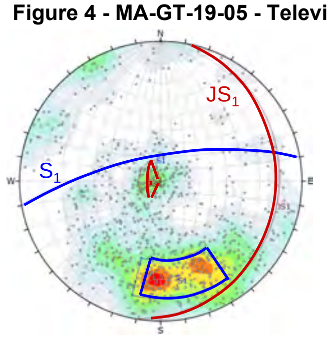

Figure 5 - MA-GT-19-06 - Televiewer Structural Data

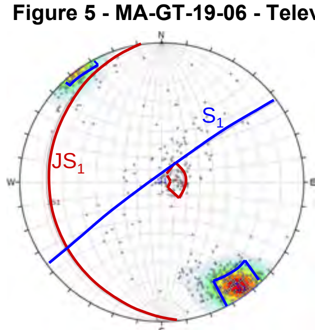

Figure 6 - MA-GT-19-07 - Televiewer Structural Data

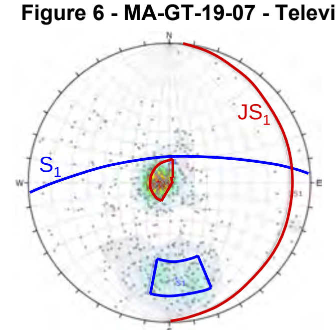

Figure 11 - Marathon Deposit - Line Mapping Data

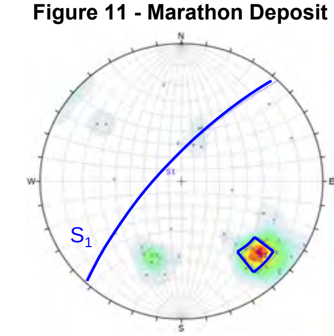

Figure 7 - MA-GT-19-05 - Structural Data - Combined[1.]

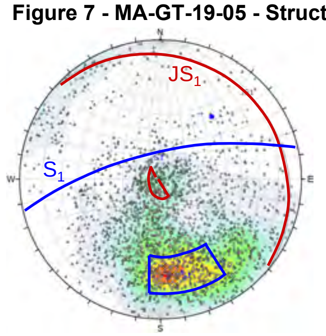

Figure 8 - MA-GT-19-06 - Structural Data - Combined[1.]

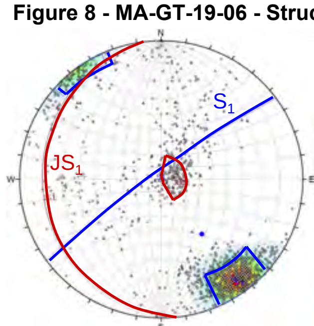

Figure 9 - MA-GT-19-07 - Structural Data - Combined[1.]

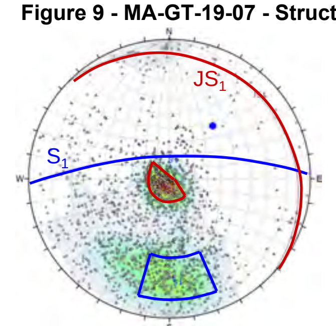

Table 1 - Structural Analysis Summary

Drillhole	Discontinuity Set	Orientated Core		Televiewer Data		Combined Data	
		Strike	Dip	Strike	Dip	Strike	Dip
MA-GT-19-05	S_1	257°	67°	260°	69°	257°	68°
MA-GT-19-05	JS_1	277°	25°	010°	10°	311°	08°
MA-GT-19-06	S_1	232°	83°	234°	84°	234°	83°
MA-GT-19-06	JS_1	158°	13°	172°	12°	173°	10°
MA-GT-19-07	S_1	269°	75°	266°	69°	268°	71°
MA-GT-19-07	JS_1	268°	11°	003°	09°	313°	08°

Table 2 - Marathon Deposit - Line Mapping - Summary

Discontinuity Set	Average Strike	Average Dip
S_1	223°	73°

Table 3 - Marathon Deposit - Structural Analysis Summary

Discontinuity Set	Average Strike	Average Dip
S_1	234°	83°
JS_1	288°	7°

VALENTINE GOLD PROJECT PRE-FEASIBILITY GEOTECHNICAL INVESTIGATION - LEPRECHAUN DEPOSIT

PROJECT:	Valentine Gold Project - Pre-Feasibility Geotechnical Investigation
DATE:	22-Oct-19
PROJECT No:	19-0015-H
SUBJECT:	Structural Analysis - Leprechaun Deposit
Prepared by:	Andrew Guest, P. Eng, Katrina Morgenroth, E.I.T.
Checked by:	Tony L. Gilman, M. Sc., P. Geo, P. Eng

NOTES:
1. Blue dot [•] signifies the drillhole collar (trend/plunge).
* All Stereonets are lower hemisphere equal angle projections
** Plots show poles to discontinuity set planes

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Figure 1 - VL-GT-19-01 - Orientated Core Structural Data

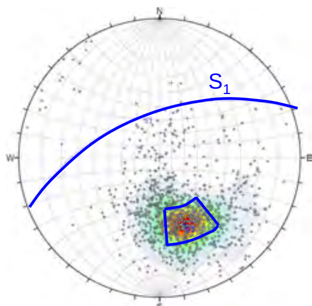


Figure 2 - VL-GT-19-02 - Orientated Core Structural Data

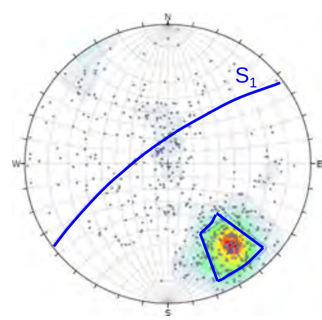


Figure 3 - VL-GT-19-03 - Orientated Core Structural Data

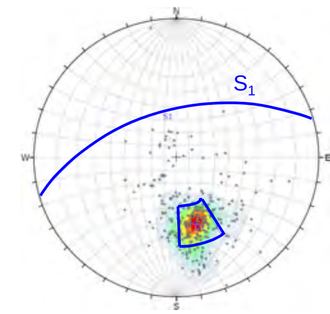


Figure 4 - VL-GT-19-04 - Orientated Core Structural Data

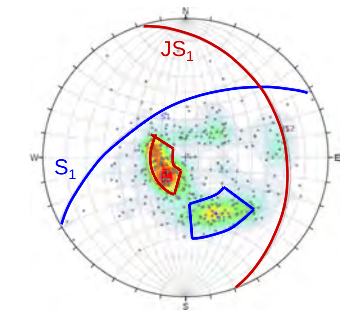


Figure 5 - VL-GT-19-01 - Televiewer Structural Data

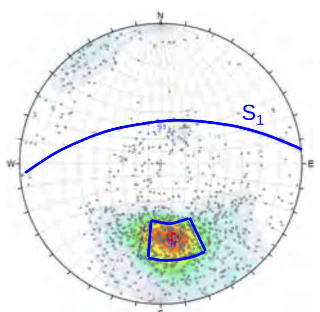


Figure 6 - VL-GT-19-02 - Televiewer Structural Data

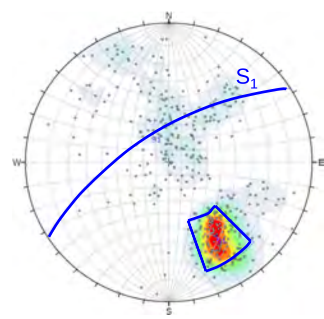


Figure 7 - VL-GT-19-03 - Televiewer Structural Data

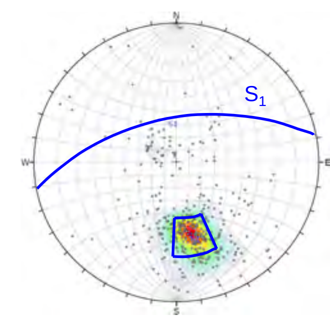


Figure 8 - VL-GT-19-04 - Televiewer Structural Data

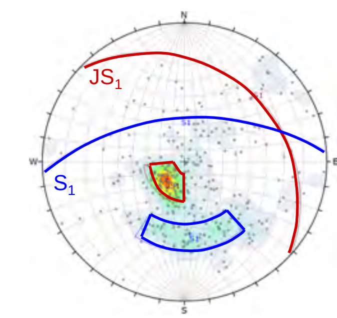


Figure 9 - VL-GT-19-01 - Structural Data - Combined[1.]

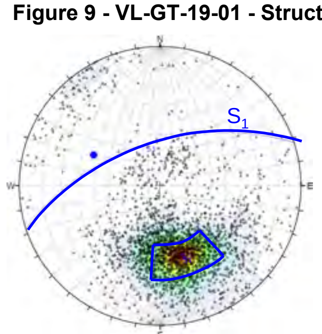


Figure 10 - VL-GT-19-02 - Structural Data - Combined[1.]

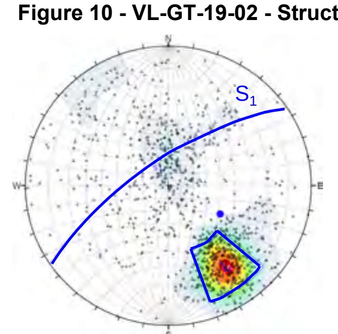


Figure 11 - VL-GT-19-03 - Structural Data -Combined[1.]

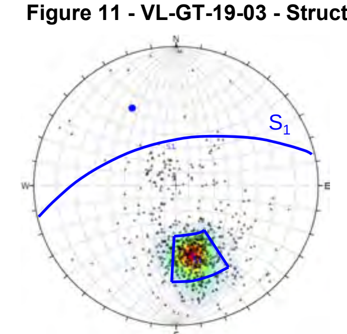


Figure 12 - VL-GT-19-04 - Structural Data - Combined[1.]

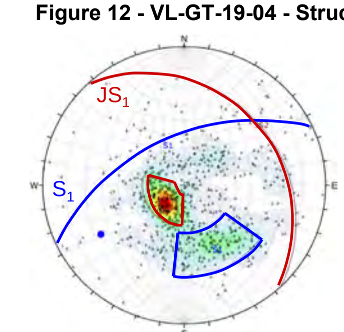


Table 1 - Structural Analysis Summary

Drillhole	Discontinuity Set	Orientated Core		Televiewer Data		Combined Data	
		Strike	Dip	Strike	Dip	Strike	Dip
VL-GT-19-01	S_1	249º	54º	265º	57º	252º	55º
VL-GT-19-02	S_1	234º	72º	238º	64º	236º	68º
VL-GT-19-03	S_1	254º	52º	259º	55º	257º	54º
VL-GT-19-04	S_1	242º	49º	266º	55º	245º	51º
VL-GT-19-04	JS_1	340º	19º	312º	21º	317º	21º

Table 2 - Leprechaun Deposit - Line Mapping - Summary

Discontinuity Set	Average Strike	Average Dip
S_1	243º	64º

Table 3 - Leprechaun Deposit - Structural Analysis Summary

Discontinuity Set	Average Strike	Average Dip
S_1	250º	56º
JS_1	303º	24º

Figure 13 - Leprechaun Deposit - All Data - Combined

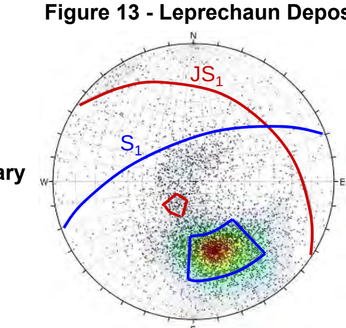
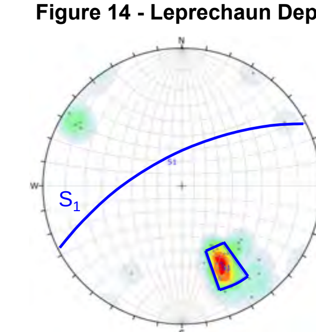


Figure 14 - Leprechaun Deposit - Line Mapping Data



VALENTINE GOLD PROJECT PRE-FEASIBILITY GEOTECHNICAL INVESTIGATION - MARATHON DEPOSIT

PROJECT:	**Valentine Gold Project - Pre-Feasibility Geotechnical Investigation**
DATE:	**02-Oct-19**
PROJECT No:	**19-0015-H**
SUBJECT:	**Geotechnical Domains - Marathon Deposit**
Prepared by:	**Andrew Guest, P. Eng**
Checked by:	**Tony L. Gilman, M. Sc., P. Geo, P. Eng**

LEGEND

- QEP - Quartz-Eye Porphyry Domain
- CG - Conglomerate Domain
- MFI - Mafic Intrusive Domain

NOTES:

1. Number of data points from Terrane 2019 Geotechnical Investigation and 2019 Marathon geotechnical logging of exploration drillholes.
2. Design range based on the mean value +/- 1 standard deviation.
3. Data from Terrane 2019 PFS Geotechnical Investigation.
4. Depth is approximate, depth based on vertical depth from collar.

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Figure 1 - RMR_{76} Distribution - QEP Domain[1.]

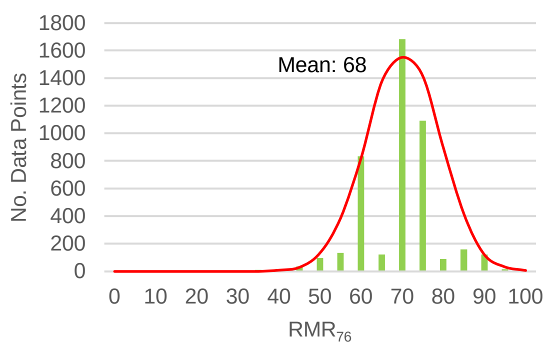


Figure 2 - RMR_{76} Distribution - CG Domain

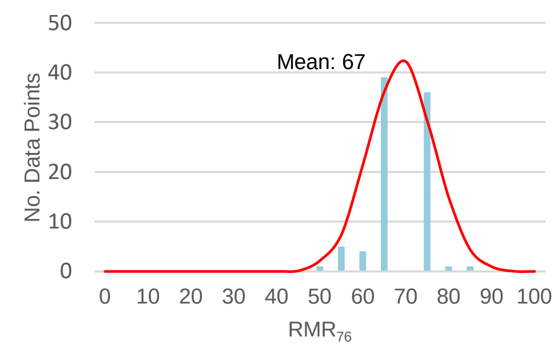


Figure 3 - RMR_{76} Distribution - MFI Domain

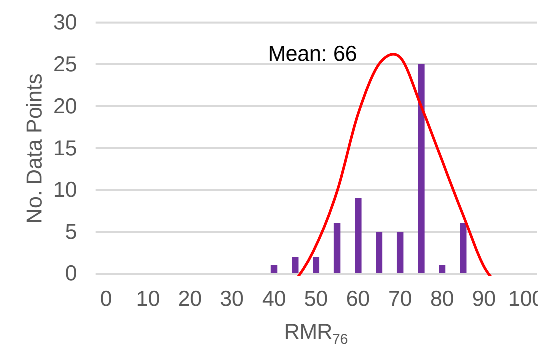


Figure 4 - RMR_{76} - Percent Less Than - QEP Domain

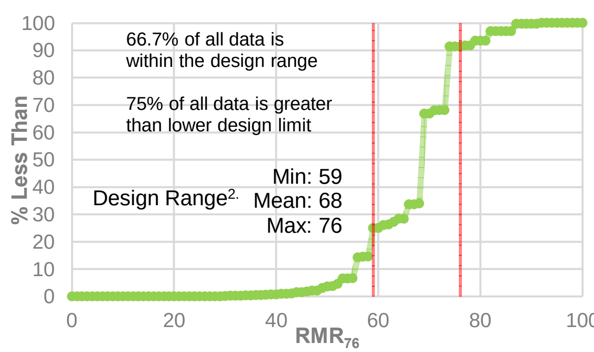


Figure 5 - RMR_{76} - Percent Less Than - CG Domain

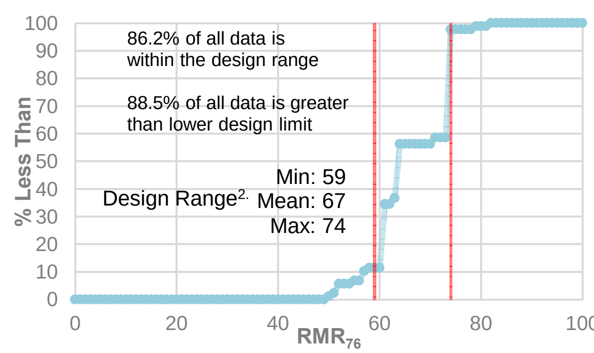


Figure 6 - RMR_{76} - Percent Less Than - MFI Domain

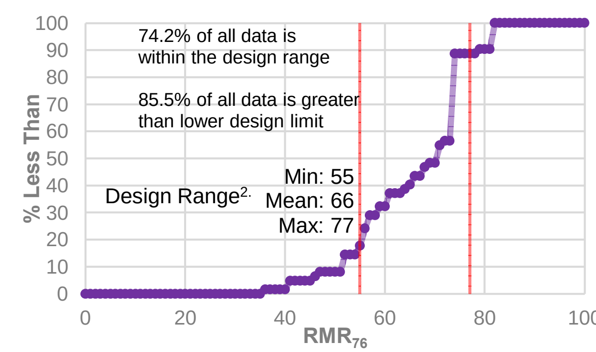


Figure 7 - RQD% vs. Depth[3.]

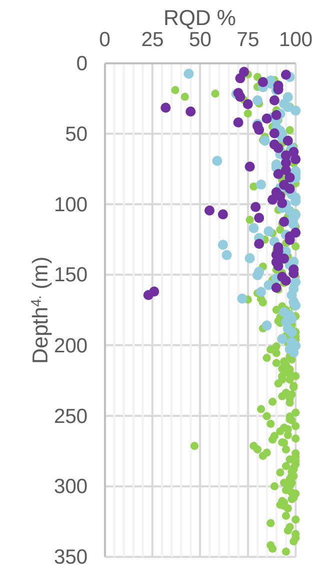


Figure 8 - Fracture Frequency vs. Depth[3.]

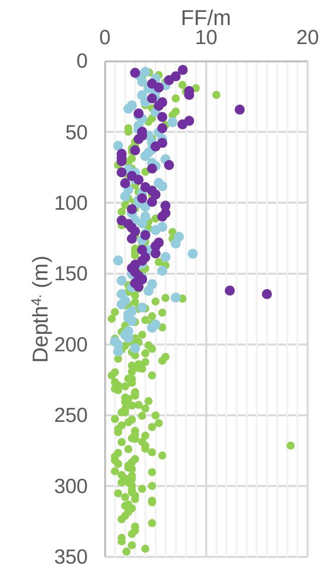


Figure 9 - RMR_{76} vs. Depth[3.]

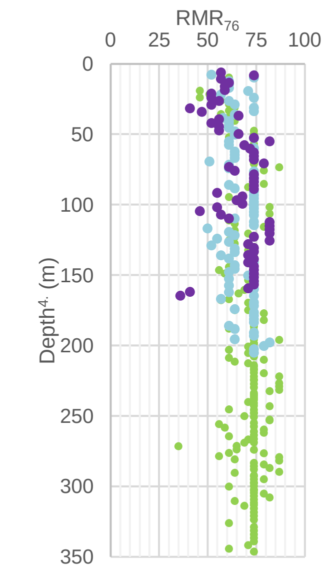
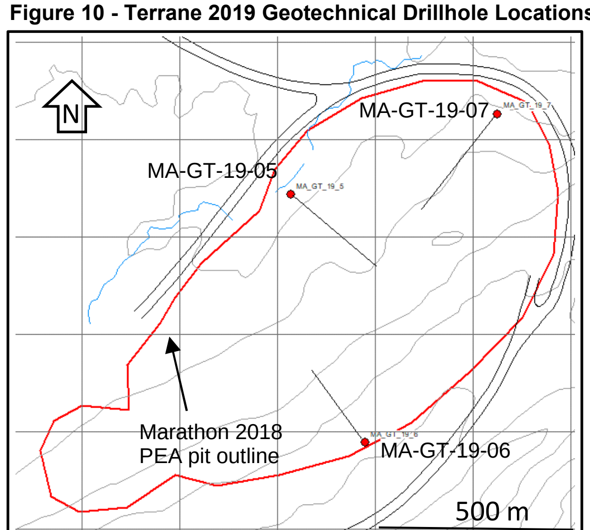


Table 1 - Terrane 2019 Geotechnical Investigation Rock Mass Characteristics Summary Per Domain

Drillhole ID	Domain	RMR_{76} Avg.	FF/m Avg.	RQD% Avg.
MA-GT-19-05	MFI	66	5	85
MA-GT-19-05	QEP	73	3	95
MA-GT-19-06	CG	67	4	91
MA-GT-19-07	QEP	70	4	92

Figure 10 - Terrane 2019 Geotechnical Drillhole Locations



Summary

Geotechnical logging of the rock mass conditions at the Marathon deposit indicates that there are three geotechnical domains. The Quartz-Eye Porphyry (QEP), Conglomerate (CG), and Mafic Intrusive (MFI) domain.

The QEP domain is characterized by Quartz-eye porphyry intruded by mafic dykes. The average RMR_{76} is 68 (Good Rock) with a standard deviation of 9.

The CG domain is characterized by Conglomerate intruded by mafic dykes. The average RMR_{76} is 67 (Good Rock) with a standard deviation of 7.

The MFI domain is characterized by Mafic Intrusive units. The average RMR_{76} is 66 (Good Rock) with a standard deviation of 11.

Figures 1 through 3 show the number of RMR_{76} values logged by core run (3 m intervals). This data includes geotechnical logging completed by Marathon personnel in 2019 exploration drillholes. The data fits a "normal distribution" where 95% of the data falls within 2 standard deviations of the mean. Figures 4 through 6 show the RMR_{76} values plotted as "percent less than curves" and show the design range for each domain. The design range is based on the mean values within each domain +/- one standard deviation.

Figures 7 through 9 display how the rock properties, within each domain, vary with depth. Generally, the rock quality improves slightly with depth. Table 1 summarizes the Terrane 2019 PFS Geotechnical Investigation results by domain for each drillhole in the Marathon deposit.

VALENTINE GOLD PROJECT PRE-FEASIBILITY GEOTECHNICAL INVESTIGATION - MARATHON DEPOSIT

PROJECT:	Valentine Gold Project - Pre-Feasibility Geotechnical Investigation
DATE:	02-Oct-19
PROJECT No:	19-0015-H
SUBJECT:	Marathon Deposit - Rock Strength Characteristics
Prepared by:	Andrew Guest, P. Eng.
Checked by:	Tony L. Gilman, M. Sc., P. Geo, P. Eng.

LEGEND
MFI - Mafic Intrusive Domain
CG - Conglomerate Domain
QEP - Quartz-eye Porphyry

NOTES
1. Failure angle relative to loading direction (i.e. vertical = 0°).
2. UCS estimated by using 4.52927e^0.05609RL (Wang et al., 2016).
3. UCS estimated by using IRS (Brown,1981).
4. The laboratory results for both Leprechaun and Marathon are included in the summary of the Conglomerate domain.
5. Outlier, not included in average.

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Table 1 - Intact Rock Strength Summary - Marathon Deposit

STRENGTH TEST	VALIDITY COMMENTS	DOMAIN	AVERAGE ROCK STRENGTH (MPa)
Laboratory Uniaxial Compressive Strength (UCS)	The UCS results showed the majority of the samples failed along internal defects (i.e. foliation). UCS results can vary greatly depending on the angle of foliation relative to the loading direction. The results of UCS laboratory testing are interpreted to represent the lower bound of the intact rock strength for design.	MFI	62
		CG	54
		QEP	89
Rebound Hammer (Schmidt Hammer)	Rebound hammer index testing was conducted on each core logging run (~3 m spacing). Using equations by Wang et al. (2016) the UCS strength has been estimated from rebound hammer results. The results of the rebound hammer index testing is interpreted to represent the mean intact rock strength for design.	MFI	120
		CG	113
		QEP	145
Field Intact Rock Strength, IRS (Brown, 1981)	IRS was evaluated on each core logging run (~ 3 m spacing). IRS is a semi - quantitative test which is used in the RMR_{76} rockmass classification and corresponds to a range of UCS values. This estimate of IRS broadly agrees with the rebound hammer results.	MFI	R5
		CG	R5
		QEP	R5

Table 2 - Summary of Laboratory UCS Testing

Deposit	Domain	UCS (MPa)	Failure Angle[1.]	Young's Modulus (GPa)	Poisson's Ratio	Comments
Marathon	MFI	62.4	25°	72.8	0.16	Shear along foliation
	CG	56.2	35°	54.7	0.12	Shear along foliation
	QEP	57.1	45°	53.6	0.07	Shear along foliation
	QEP	66.1	30°	56.4	0.07	Shear along foliation
	QEP	113.2	10°	67.0	0.1	Axial splitting
	QEP	120.1	55°	65.0	0.09	Shear along foliation at bottom of sample
Leprechaun[4.]	CG	63.4	40°	42.5	0.11	Shear along foliation at bottom of sample
	CG	41.9	35°	46.9	0.05	Shear along foliation

Table 3 - Summary of Field Rebound Hammer Tests[2.]

Deposit	Domain	# of Tests	Average MPa	Min MPa	Max MPa
Marathon	MFI	62	120	24	205
	CG	84	113	43	147
	QEP	201	145	86	196

Table 4 - Summary of Field Intact Rock Strength (Brown, 1981)[3.]

Deposit	Domain	# of Tests	IRS		
			R5 %	R4 %	R3 %
Marathon	MFI	62	73	27	0
	CG	84	98	2	0
	QEP	201	95	5	0

Table 5 - Summary of Laboratory Brazilian Tensile Strength Testing

Deposit	Domain	σ_t (MPa)
Marathon	QEP	15.8
	CG	8.8
	QEP	20.1
	QEP	11.2

Table 6 - Summary of Laboratory Discontinuity Direct Shear Strength Testing

Deposit	Domain	Friction Angle
Marathon	CG	23.3°[5.]
	CG	25.5°
	CG	33.8°
	CG	36.3°
	QEP	32.6°
Leprechaun[4.]	CG	34.7°

Table 7 - Summary of Laboratory Triaxial Compressive Strength Testing

Deposit	Domain	σ_1 (MPa)	σ_3 (MPa)	Young's Modulus (GPa)	Poisson's Ratio
Marathon	MFI	204.8	3.0	81.6	0.17
	QEP	182.5	3.0	70.58	0.12
	QEP	164.8	7.0	76.34	0.09
	CG	56.1	3.0	59.69	0.12
Leprechaun[4.]	CG	134.3	7.0	76.85	0.10

Table 8 - Summary of UCS Values Used for Pit Slope Design

Deposit	Domain	UCS_{design}	UCS_{min}
Marathon	MFI	120	62
	CG	113	54
	QEP	145	89

VALENTINE GOLD PROJECT PRE-FEASIBILITY GEOTECHNICAL INVESTIGATION - LEPRECHAUN DEPOSIT

PROJECT:	**Valentine Gold Project - Pre-Feasibility Geotechnical Investigation**
DATE:	**22-Oct-19**
PROJECT No:	**19-0015-H**
SUBJECT:	**Geotechnical Domains - Leprechaun Deposit**
Prepared by:	**Andrew Guest, P. Eng**
Checked by:	**Tony L. Gilman, M. Sc., P. Geo, P. Eng**

LEGEND

TRJ - Trondhjemite Domain

CG - Conglomerate Domain

NOTES:
1. Number of data points from Terrane 2019 PFS Geotechnical Investigation and 2019 Marathon geotechnical logging of exploration drillholes.
2. Design range based on the mean value +/- 1 standard deviation.
3. Data from Terrane 2019 PFS Geotechnical Investigation.
4. Depth is approximate, depth based on vertical depth from collar.

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Figure 1 - RMR_{76} Distribution - TRJ Domain[1.]

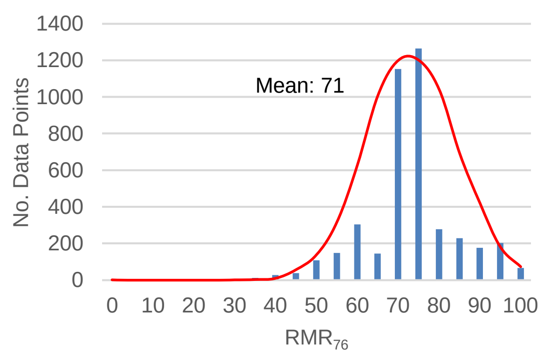

Figure 2 - RMR_{76} Distribution - CG Domain

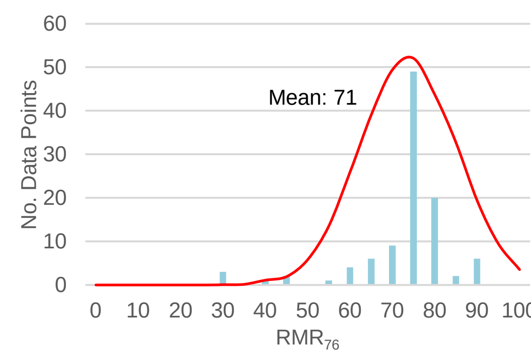

Figure 3 - RMR_{76} - Percent Less Than - TRJ Domain

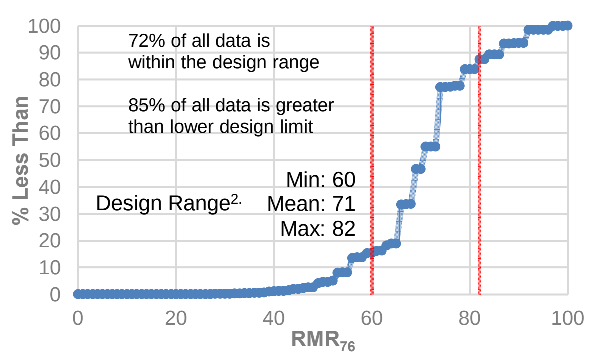

Figure 4 - RMR_{76} - Percent Less Than - CG Domain

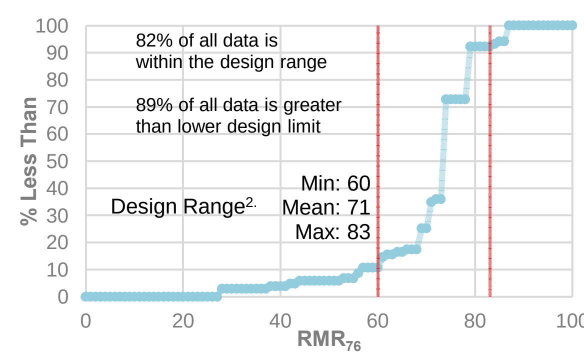

Figure 5 - RQD% vs. Depth[3.]

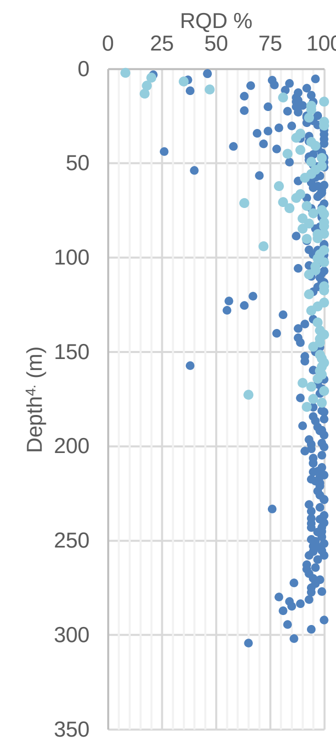

Figure 6 - Fracture Frequency vs. Depth[3.]

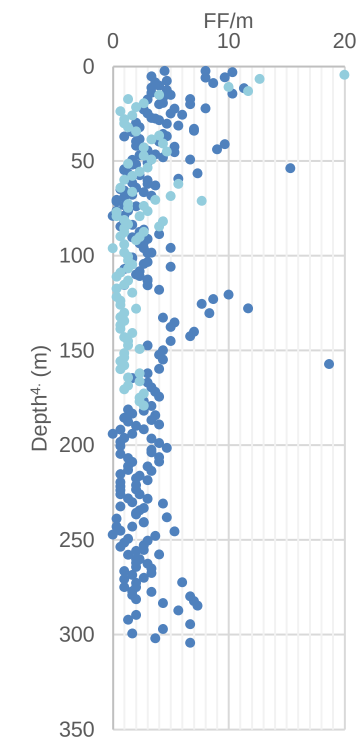

Figure 7 - RMR_{76} vs. Depth[3.]

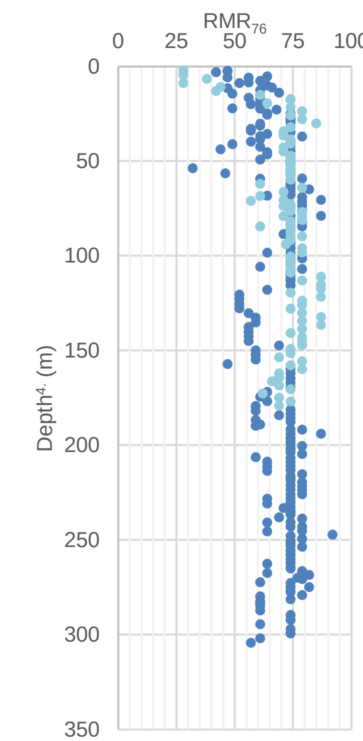

Figure 8 - Terrane 2019 Geotechnical Drillhole Locations

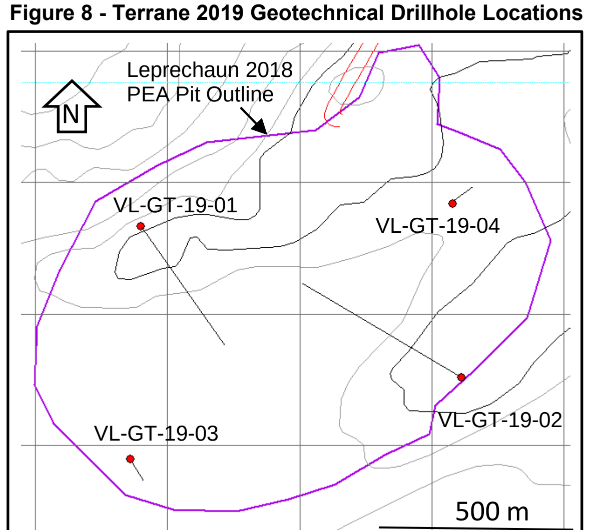

Table 1 - Terrane 2019 Geotechnical Borehole Investigation

Rock Mass Characteristics Summary Per Domain

Drillhole ID	Domain	RMR_{76} Average	FF/m Average	RQD% Average
VL-GT-19-01	TRJ	68	4	93
VL-GT-19-02	TRJ	76	1	98
VL-GT-19-02	CG	72	3	91
VL-GT-19-03	TRJ	62	5	81
VL-GT-19-03	CG	70	4	88
VL-GT-19-04	TRJ	67	4	87

Summary

Geotechnical logging of the rock mass conditions at the Leprechaun deposit indicates that there are two geotechnical domains. The Trondhjemite (TRJ) and Conglomerate (CG) domain.

The TRJ domain is characterized by Trondhjemite intruded by mafic dykes. The average RMR_{76} is 71 (Good Rock) with a standard deviation of 11.

The CG domain is characterized by Conglomerate intruded by mafic dykes. The average RMR76 is 71 (good Rock) with a standard deviation of 11.

Figures 1 & 2 show the number of RMR_{76} values logged by core run (3 m intervals). This data includes geotechnical logging completed by Marathon personnel in 2019 exploration drillholes. The data fits a "normal distribution" where 95% of the data falls within 2 standard deviations of the mean. Figures 3 & 4 show the RMR_{76} values plotted as "percent less than curves" and show the design range for each domain. The design range is based on the mean values within each domain +/- one standard deviation.

Figures 5 through 7 display how the rock properties, within each domain, vary with depth. Generally, the rock quality improves slightly with depth. Table 1 summarizes the Terrane 2019 PFS Geotechnical Investigation results by domain for each drillhole within the Leprechaun deposit.

VALENTINE GOLD PROJECT PRE-FEASIBILITY GEOTECHNICAL INVESTIGATION - LEPRECHAUN DEPOSIT

PROJECT: Valentine Gold Project - Pre-Feasibility Geotechnical Investigation
DATE: 02-Oct-19
PROJECT No: 19-0015-H
SUBJECT: Leprechaun Deposit - Rock Strength Characteristics
Prepared by: Andrew Guest, P. Eng.
Checked by: Tony L. Gilman, M. Sc., P. Geo, P. Eng.

LEGEND
TRJ - Trondhjemite Domain
CG - Conglomerate Domain

NOTES
1. Failure angle relative to loading direction (i.e. vertical = 0°).
2. Outlier, not included in average.
3. UCS estimated by using 4.52927e^0.05609RL (Wang et al., 2016).
4. UCS estimated by using IRS (Brown,1981).
5. The laboratory results for both Leprechaun and Marathon are included in the summary of the Conglomerate domain.

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Table 1 - Intact Rock Strength Summary - Leprechaun Deposit

STRENGTH TEST	VALIDITY COMMENTS	DOMAIN	AVERAGE ROCK STRENGTH (MPa)
Laboratory Uniaxial Compressive Strength (UCS)	The UCS results showed the majority of the samples failed along internal defects (i.e. foliation). UCS results can vary greatly depending on the angle of foliation relative to the loading direction. The results of UCS laboratory testing are interpreted to represent the lower bound of the intact rock strength for design.	TRJ	72
		CG	54
Rebound Hammer (Schmidt Hammer)	Rebound hammer index testing was conducted on each core logging run (~3 m spacing). Using equations by Wang et al. (2016) the UCS strength has been estimated from rebound hammer results. The results of the rebound hammer index testing is interpreted to represent the mean intact rock strength for design.	TRJ	112
		CG	126
Field Intact Rock Strength, IRS (Brown, 1981)	IRS was evaluated on each core logging run (~ 3 m spacing). IRS is a semi - quantitative test which is used in the RMR76 rockmass classification and corresponds to a range of UCS values. This estimate of IRS broadly agrees with the rebound hammer results.	TRJ	R5
		CG	R5

Table 2 - Summary of Laboratory UCS Testing

Deposit	Domain	UCS (MPa)	Failure Angle[1]	Young's Modulus (GPa)	Poisson's Ratio	Comments
Leprechaun	TRJ	76.4	40°	52.2	0.08	Shear along foliation
	TRJ	68.6	25°	59.1	0.07	Shear along foliation
	TRJ	96.1	10°	59.3	0.09	Axial splitting
	TRJ	78.2	40°	41.2	0.12	Shear along foliation
	TRJ	301.6[2]	-	83.1	0.16	Violent failure
	TRJ	40.2	50°	41.1	0.04	Shear along foliation
	CG	63.4	40°	42.5	0.11	Shear along foliation
	CG	41.9	35°	46.9	0.05	Shear along foliation
Marathon[5]	CG	56.2	35°	54.7	0.12	Shear along foliation

Table 3 - Summary of Field Rebound Hammer Tests[3]

Deposit	Domain	# of Tests	Average MPa	Min MPa	Max MPa
Leprechaun	TRJ	238	112	40	164
	CG	90	126	78	151

Table 4 - Summary of Field Intact Rock Strength (Brown, 1981)[4]

Deposit	Domain	# of Tests	IRS R5 %	IRS R4 %	IRS R3 %
Leprechaun	TRJ	239	83	16	<1
	CG	92	85	10	5

Table 5 - Summary of Laboratory Brazilian Tensile Strength Testing

Deposit	Domain	σ_t (MPa)
Leprechaun	TRJ	13.0
	TRJ	15.7
	TRJ	18.0
	TRJ	11.6
Marathon[5]	CG	8.8

Table 6 - Summary of Laboratory Discontinuity Direct Shear Strength Testing

Deposit	Domain	Friction Angle
Leprechaun	TRJ	32.3°
	TRJ	27.8°
	TRJ	32.2°
	CG	34.7°
Marathon[5]	CG	23.3°[2]
	CG	25.5°
	CG	33.8°
	CG	36.3°

Table 7 - Summary of Laboratory Triaxial Compressive Strength Testing

Deposit	Domain	σ_1 (MPa)	σ_3 (MPa)	Young's Modulus (GPa)	Poisson's Ratio
Leprechaun	CG	134.3	7.0	76.85	0.10
	TRJ	46.9	3.0	35.16	0.15
	TRJ	198.0	7.0	68.29	0.17
	TRJ	322.3	7.0	89.86	0.22
Marathon[5]	CG	56.1	3.0	59.69	0.12

Table 8 - Summary of UCS Values Used for Pit Slope Design

Deposit	Domain	UCS_{design}	UCS_{min}
Leprechaun	TRJ	112	72
	CG	126	54

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DRAFT

Hydraulic Conductivity Testing and Estimation of Pit Inflows

Pre-feasibility Geotechnical Open Pit Slope Recommendations
Valentine Gold Project,
Central Newfoundland, NL

November 12, 2019

GEMTEC Consulting Engineers and Scientists Ltd.

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APPENDIX A Packer Test Analysis Reports

1.0 INTRODUCTION

GEMTEC Consulting Engineers and Scientists Limited (GEMTEC) was retained for Terrane Geoscience Incorporated (Terrane) to provide hydrogeological services in support of a Pre-Feasibility (PFS) Geotechnical Open Pit Slope Recommendations program for the Valentine Gold Project located in Central Newfoundland. As part of this program, GEMTEC worked with Terrane to select packer test intervals within the geotechnical boreholes completed at the Leprechaun and Marathon pits, and carried out analysis of packer test data to provide general estimates of hydraulic conductivity for the rock mass in the areas of each pit that was used for preliminary calculations of pit inflows.

GEMTEC did not supervise the packer testing, and its analysis is based solely on the data and information provided by Terrane. This report contains all of the findings, results, conclusions, and recommendations for the hydrogeological program. Note background information relating to the location of each pit, and its physical setting, surficial, bedrock and structural geology, and pit geometry is provided in Terrane's overall report, of which this report forms an appendix.

2.0 HYDROGEOLOGICAL SETTING OF THE PIT AREAS

Preliminary hydrogeological studies were conducted by Stantec Consulting Limited in 2017 and 2019 to characterize baseline groundwater conditions at the site in support of environmental permitting and mine development. The initial hydrogeology program completed in November 2017 is described in Stantec (2017b), and included a desktop review of existing topographic, geological and hydrogeological data combined with a field program comprising hydraulic conductivity testing, water quality sampling, and groundwater level monitoring in six selected historical exploration boreholes within the Leprechaun and Marathon deposit areas. A follow-up field program was completed by Stantec in November 2018 to retrieve long-term monitoring data from water level loggers installed in the six 2017 field program boreholes, and also included site-wide groundwater level monitoring in 191 historical exploration boreholes. Results of the 2019 program are provided in Stantec (2019). An additional baseline hydrogeological study is currently in progress for the site, but the results of this study were not available at the time of writing.

The two pit areas are inferred to be underlain by an unconfined to semi-confining aquifer system contained within the intrusive and sedimentary bedrock units that characterize the deposits. Due to the relatively low matrix porosity, the majority of groundwater flow in the bedrock is expected to occur within fractures, faults, joints, and other structural apertures; and the bedrocks' flow characteristics (permeability) will depend on the width, spacing, and interconnectivity of these apertures. The direction of shallow horizontal groundwater flow in the two pit areas is inferred to follow local topography and surface water run-off, which is generally to the northwest towards Valentine Lake in the Marathon pit area, and to the southeast towards Victoria Lake in the Leprechaun Pit area. Locally, a component of drainage in the Marathon pit area is also expected to travel northeast to Victoria River, which ultimately drains into Red Indian Lake to the north.

Groundwater in both pit areas is expected to be mainly derived from local recharge and runoff; while at moderate depths groundwater may be influenced by recharge and lateral inflow from up-gradient areas along the crest of the prominent northeast-trending ridge that runs through the Valentine Gold project area. Surrounding surface water features including Valentine Lake and Victoria River in the Marathon Pit area, and Victoria Lake in the Leprechaun Pit area are expected to be areas of groundwater discharge. Groundwater levels are generally shallow in the two pit areas (i.e., less than 10 m below ground surface) with localized artesian conditions, and are typically a subdued reflection of the topography. Results of hydraulic testing completed by Stantec (2017b) for selected historical exploration boreholes in the two pit areas indicated average hydraulic conductivity values of 3.4×10^{-8} m/s and 7.8×10^{-8} m/s, respectively, for the rock masses of the Leprechaun and Marathon pits.

3.0 PACKER HYDRAULIC CONDUCTIVITY TESTING

In-situ packer hydraulic conductivity testing was conducted in selected geotechnical boreholes in the two Project pit areas by Terrane Geoscience during the geotechnical borehole drilling program. The objective of the packer testing was to provide general estimates of hydraulic conductivity for the rock mass in the areas of each proposed pit that could be used for preliminary calculations of pit inflows. This packer testing was not extensive and included both intact rock and fractured rock zones.

3.1 Methods

The geologic models for the Leprechaun and Marathon ore deposits indicated several predominant bedrock units within the pits, and the objective of the packer testing was to estimate the bulk hydraulic conductivity of these various bedrock units, and to determine if there was any variation associated with depth or the presence of various structural features (faults, fractures, shear zones). Five (5) geotechnical boreholes were packer tested as part of the current program, including the following:

- three (3) boreholes (VL-GT-19-01, VL-GT-19-02, and VL-GT-19-03) located along the proposed pit shell of the Leprechaun pit; and,
- two (2) boreholes (MA-GT-19-05, and MA-GT-19-06) located along the proposed pit shell of the Marathon pit.

The packer tests were conducted using a Standard Wireline Packer System (SWiPS) manufactured by Inflatable Packers International (IPI). The tests were performed using the constant head (Lugeon) packer injection test method and utilized a single packer inserted through the HQ drill rods to test selected intervals as the hole was advanced.

The test was conducted as follows:

- When a borehole was advanced to the bottom of a chosen test interval and the hole was flushed with clean water through the drill rod until the return water was clear.
- The drill rods were then withdrawn and a single-element packer assembly was lowered inside the drill rods to the top of the test interval with the wireline. The packer bladder was then inflated using water to isolate the test interval; the bottom of which was bounded by the bottom of the drilled section of the borehole.
- Once a successful seal was established, water was pumped into the isolated test interval through the injection pipe to achieve a constant differential head and inflow rate. A total of three ascending and two descending water pressure steps were applied for each interval with regulated constant head.
- For each test step, the water injection rate was observed until the water injection rate had stabilized (up to 10 minutes). During this observation period, the injected quantity of water was observed and recorded every minute, and the stabilized flow rate was used to determine the bulk hydraulic conductivity of the rock mass over the tested interval.

A total of 10 packer tests were performed within these boreholes at selected intervals that covered the full planned depths of pit development; including six tests carried out in the Leprechaun pit at downhole depths ranging from 12 to 374 meters below ground surface (mbgs), and four tests in the Marathon pit at downhole depths ranging from 30 to 296 mbgs. The tests for the Leprechaun pit were all completed within the trondhjemite (TRJ) bedrock unit, which makes up the bulk of the pit's rock mass; while the tests for the Marathon Pit were limited to one or two tests for each of the three primary bedrock units making up the pit's rock mass (quartz-eye porphyry (QEP), conglomerate (CG) and mafic dyke (MD).

In addition, the Valentine Lake thrust fault was tested in packer test PT2 in borehole VL-GT-19-03 (Leprechaun pit). An additional test location was selected to test the Valentine Lake thrust fault in borehole MA-GT-19-06 (Marathon pit); however a mechanical issue with the drill rig resulted in termination of this borehole before reaching the target downhole depth of approximately 320 m.

With the exception of packer test PT2 in borehole MA-GT-19-06 (Marathon pit), which had a test interval length of approximately 71 m, all other packer tests were performed over intervals ranging from 17 to 32 m long. Table 1 summarizes packer test information and results for this program. Packer test data are presented on the analysis reports in Appendix A.

3.2 Packer Test Results

The hydraulic conductivity for each test interval was determined based on the analysis of the packer test data using the software AquiferTest® Version 8 (Waterloo Hydrogeologic, Waterloo, ON). Overall results are presented below. Results for each test interval are presented on the analysis reports in Appendix A.

The calculated hydraulic conductivity values for each test interval in the two pit areas are provided in Table 1, and a plot of hydraulic conductivity versus depth separated by rock type for each pit is presented in Figure 1.

For the Leprauchan pit, where all tests were completed within the trondhjemite bedrock unit, the hydraulic conductivity values ranged through four orders of magnitude from 5.79 x10^{-10} to 1.69 x 10^{-6} m/s, with a geometric mean of 5.88 x 10^{-8} m/s. The highest hydralic conductivity value (1.69 x 10^{-6} m/s) was measured in packer test PT2 in borehole VL-GT-19-03, which included a faulted interval from downhole depth 44 to 50 m, (possibly representing the Valentine Lake fault). The relatively high hydraulic conductivity measured in this test interval is attributed to increased permeability due to fracturing associated with this structure. There was no apparent variation related to depth evident in the Leprechaun pit test results.

For the Marathon pit, a similar broad range in hydraulic conductivity values through four orders of magnitude was measured from 3.9 x10^{-10} to 6.18 x 10^{-6} m/s. The variability for these results may be a function of bedrock type, as well as structure. A single hydraulic conductivity value was determined for the mafic dyke (8.73 x 10^{-7} m/s) and the quartz-eye porphyry (3.9 x10^{-10} m/s). Two tests were completed in the conglomerate, both within borehole MA-GT-19-06, including a shallow test at downhole depth 32 m, and a deeper test at downhole depth 180 m. The shallow test returned a hydraulic conductivity of 1.93 x 10^{-6} m/s, and the deeper test returned a value of 6.18 x10^{-9} m/s. All four test results for the Marathon pit show an apparent linear trend in decreasing hydraulic conductivity with depth, with an approximately one order of magnitude decrease in hydraulic conductivity for each 100 m increase in depth. While it is expected that hydraulic conductivity will decrease with depth as fracture apertures collapse due to lithostatic load; the sparsity of data and variable rock types does not allow this relationship to be clearly demonstrated in the Marathon pit as part of this program.

Table 1 Summary of Packer Testing

Location of Borehole	Borehole ID	Packer Test (Lugeon)						Lithology	Hydraulic Conductivity (K) (m/s)		Notes
		Test ID	Tested Zone (m)[1]								
			From	To	Interval Length	Mid-point (tested zone)			Packer Test Average	Geomean	
Leprechaun Open Pit	VL-GT-19-01	PT1	30.3	50.0	19.7	40.2		Trondhjemite (TRJ)	2.51E-08	5.88E-08	
		PT2	150.3	170.0	19.7	160.2		Trondhjemite (TRJ)	5.79E-10		
		PT3	348.3	374.0	25.7	361.2		Trondhjemite (TRJ)	4.14E-08		Fault identified from 350 - 353 m; associated with good rock quality
	VL-GT-19-02	PT1	354.3	374.0	19.7	364.2		Trondhjemite (TRJ), QTP	1.73E-07		
	VL-GT-19-03	PT1	12.3	32.0	19.7	22.2		Trondhjemite (TRJ), QTP	2.36E-07		
		PT2	45.3	77.0	31.7	61.2		Trondhjemite (TRJ); Mafic Dyke (MD)	1.69E-06		Fault identified from 44 - 50 m; associated with fair - poor rock quality
Marathon Open Pit	MA-GT-19-05	PT1	45.3	62	16.7	53.7		Mafic Dyke (MD)	8.73E-07	4.49E-08	Flowing artesian conditions; static water level not determined
		PT2	273.3	296	22.7	284.7		Quartz Eye Porphyry (QE-POR)	3.90E-10		Flowing artesian conditions; static water level not determined
	MA-GT-19-06	PT1	30.3	50	19.7	40.2		Conglomerate (CG)	1.93E-06		
		PT2	189.3	260	70.7	224.7		Conglomerate (CG); Mafic Dyke (MD)	6.18E-09		Fault identified from 233 - 236 m; associated with good rock quality

Notes: 1. Depth measurements are referenced with respect to ground surface, and are inclined borehole depths.

In general, the hydraulic conductivity values determined during this program were within the typical range of values in the literature for similar rock types (e.g., Freeze & Cherry, 1979).

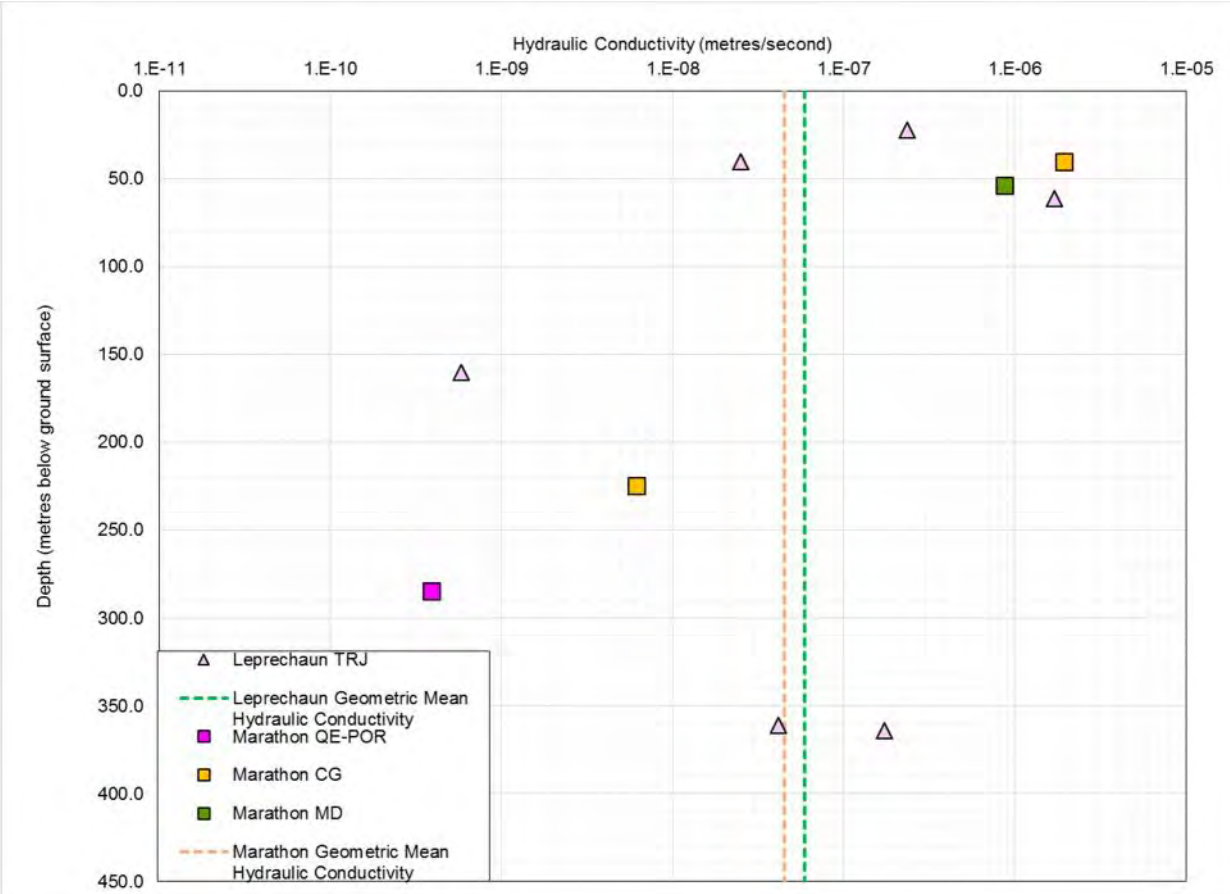


Figure 1 Hydraulic Conductivity vs Depth by Bedrock Unit and Pit

Given the limited scope of this program, particularly for the Marathon pit, there were not enough test data to determine separate hydraulic conductivity estimates based on bedrock type and depth. Because of this, the geometric mean of the packer test results for each pit were used to represent the average hydraulic conductivity of the bulk rock mass for estimation of groundwater inflow rates. The geometric mean hydraulic conductivity values used for estimation of groundwater pit inflow rates for the Project pits are provided in Table 1 (5.88 x 10^{-8} m/s for the Leprechaun pit, and 4.49 x 10^{-8} m/s for the Marathon pit). These average hydraulic conductivity values are similar to the average hydraulic conductivity estimates determined for the bulk rock mass of the two pits provided by Stantec (3.4 x 10^{-8} m/s for the Leprechaun pit, and 7.8 x 10^{-8} m/s for the Marathon pit) (Stantec, 2017b), and are considered to be reasonable for the purposes of preliminary estimates of pit inflows presented here. It should be noted however that various faults, fractures, and shear zones were identified in the two pits as part of the geotechnical program that weren't tested and are not yet well characterized. Such structures may have substantially higher localized permeability than the surrounding rock mass that could lead to higher pit inflows.

4.0 ESTIMATION OF PIT INFLOWS

The calculations used to determine the estimated total inflows to each of the pits are provided below and include two primary sources: 1) direct precipitation into the future pits, and 2) groundwater inflow from bedrock. The inflow estimate associated with direct precipitation was determined using a simple calculation based on metrological data for the site and pit geometry. The estimate of groundwater inflow was determined using an analytical solution that assumes steady-state conditions and full development down to the final base elevation.

These calculations are considered preliminary, and are based on a number of assumptions, as well as a limited hydraulic conductivity data set, and as such are considered to provide only an order of magnitude estimate of pit inflows.

4.1 Inflow from Direct Precipitation

Inflow from direct precipitation was determined using the long-term average annual rainfall total for the Project area (1,236 mm/year), applied over the area of the pit footprint (6.6 x 10^5 m^2 for the Leprechaun pit, and 8.4 x 10^5 m^2 for the Marathon pit). A direct precipitation pit inflow of approximately 2,236 m^2/day was calculated for the Leprechaun pit, and 2,839 m^2/day was calculated for the Marathon pit.

Note that evaporation and transpiration were not considered in these estimates as it is assumed that, during normal operations, water would be pumped out of the pits as soon as it accumulates thus reducing the time for evaporation, and that exposed bedrock of the pit would be devoid of vegetation so transpiration within the active extraction area would be negligible.

4.2 Groundwater Inflow Estimates

4.2.1 Analytical Solution Method

The analytical solution developed by Marinelli and Niccoli (2000) was used to estimate the steady-state radius of hydraulic influence of the pits and groundwater inflow rates. The Marinelli and Niccoli solution presents separate equations to calculate groundwater inflow from the pit walls (Zone 1) and from the pit bottom (Zone 2) separately, based on the simplified conceptual model presented in Figure 2.

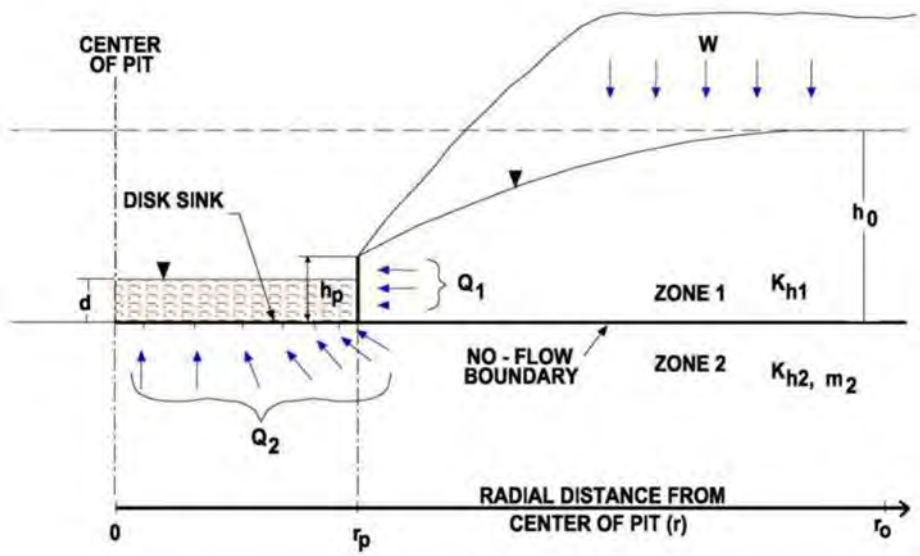


Figure 2 Conceptual Model of Groundwater Pit Inflow (after Marinelli and Niccoli, 2000)

For Zone 1, the analytical solution considers steady-state, unconfined, horizontal, radial flow and assumes that:

- pit walls are approximated as a circular cylinder;
- groundwater flow is horizontal;
- the static (pre-mining) water level is horizontal;
- uniform distributed recharge occurs across the site;
- all recharge within the radius of influence of the pit is captured by the excavation; and,
- groundwater flow toward the pit is axially symmetric.

Groundwater inflows were calculated for Zone 1 using the following equations:

$$h_0 = \sqrt{h_p^2 + \frac{W}{k_{h1}}\left[r_0^2 ln\left(\frac{r_0}{r_p}\right) - \left(\frac{r_0^2 - r_p^2}{2}\right)\right]}$$

where:

h_0 =	*initial (pre-mining) saturated thickness above the base of Zone 1 (m).*
h_p =	*saturated thickness above the base of Zone 1 at r_p (m);*
W =	*distributed groundwater recharge (m/s);*
K_{h1} =	*horizontal hydraulic conductivity of materials within Zone 1 (m/s);*
r_0 =	*radius of influence (m); and,*
r_p =	*effective pit radius (m).*

Given input values of W, K_{h1}, r_p, h_p and h_0, the radius of influence (r_0) is determined from the above equation through iteration. Once r_0 is determined, the pit inflow rate (Q_1) from Zone 1 is calculated as follows:

$$Q_1 = W \ (r_0^2 - r_p^2)$$

For Zone 2, the analytical solution considers steady-state flow to one side of a circular disk sink of constant and uniform drawdown that represents the bottom of the pit. This solution assumes

- The hydraulic head is initially uniform
- The disk sink has a constant hydraulic head equivalent to the elevation the water level in the pit
- Flow to the disk sink is three-dimensional and axially symmetrical
- Materials are anisotropic and the principal directions are horizontal and vertical for hydraulic conductivity

Groundwater inflows were calculated for Zone 2 using the following equations:

$$Q_2 = 4r_p \left(\frac{K_{h2}}{m_2}\right) (h_0 - d)$$

where:

K_{h2} = *horizontal hydraulic conductivity of Zone 2 (m/s);*

K_{v2} = *vertical hydraulic conductivity of Zone 2 (m/s);*

d = *ponded water depth (m); and,*

$$m_2 = \sqrt{\frac{K_{h2}}{K_{v2}}}$$

4.2.2 Groundwater Parameters

The parameters used in the calculation of groundwater pit inflows using the Marinelli and Niccoli solution are presented in Table 2, and are further discussed below.

Groundwater Recharge Flux (W)

The groundwater recharge flux (W) is taken as approximately 1.96 x 10^{-9} m/s, based on an estimate of annual precipitation for the Valentine Lake Project of 1,236 mm/year (provided in Stantec, 2017a) and conservatively assuming a groundwater recharge rate of 5%. We consider this recharge rate to be conservative since that the pits are situation on a topographic ridge (where, in general, runoff would be enhanced and groundwater recharge diminished), and since the dominant bedrock types are fractured igneous rocks with limited secondary porosity.

Effective Pit Radius (r_p) and radius of influence (r_o)

The effective pit radius (r_p) was estimated assuming a saturated thickness of pit wall (h_p) equal to one-third of the total pit depth, and the planned slope of the pit walls. The effective pit radius was estimated for the pits as follows:

- Leprechaun pit – 208 m
- Marathon pit – 256 m

The radius of influence (r_o) is determined iteratively using the first equation above in the Marinelli and Niccoli solution. The radius of influence was calculated for the pits as follows:

- Leprechaun pit – 1,382 m
- Marathon pit – 1,486 m

These calculated values suggest that the radius of influence for the Leprechaun pit extends out to Victoria Lake; while the radius of influence for the Marathon pit extends out to Victoria River. Valentine Lake does not appear to be within the radius of influence of either of the pits.

Saturated Aquifer Thickness (h_p and h_0)

The average original ground surface elevation for the Leprechaun pit is estimated to be 390 m above sea level (masl), and the proposed pit bottom is 68 masl. Water level measurements in the pit geotechnical boreholes indicate water levels ranging from 3.4 to 6.4 metres below ground surface (mbgs), and an average water level depth of 4.5 mbgs. Based on these estimates, the saturated aquifer thickness (h_0) for Zone 1 is taken as 317.5 m for the Leprechaun pit.

The average original ground surface elevation for the Marathon pit is estimated to be 339 m above sea level, and the proposed pit bottom is -34 masl. Water level measurements in the pit geotechnical boreholes indicate water levels ranging from an assumed 0.5 meters above ground surface (artesian; water level not measured) to 3.5 mbgs, and an average water level depth of 1.5 mbgs. Based on these estimates, the saturated aquifer thickness (h_0) for Zone 1 is taken as 371.5 m for the Marathon pit.

The maximum saturated aquifer thickness at the pit wall (hp) was assumed to be one-third of the saturated aquifer thickness (h_0) for Zone 1, equal to 105.8 m for the Leprechaun pit and 124 m for the Marathon pit. The pond depth or depth of ponded water at the base (d) was set to 0 m (pit fully dewatered) for both pits.

Hydraulic Conductivity (K_{h1}, K_{h2}, K_{v2})

To apply the analytical solution to the pits, it was considered that Zone 1 and Zone 2 are represented by the properties of the bedrock units encountered in the pit geotechnical boreholes, and the geometric mean of hydraulic conductivity values obtained from packer testing provide a reasonable estimation of the bulk rock mass within both Zone 1 and Zone 2 in each pit (i.e., K_{h1}, K_{h2}) = 5.88 x10^{-8} m/s for the Leprechaun pit, and 4.49 x10^{-8} m/s for the Marathon pit.

Further, the limited data set does not allow for a clear trend to be observed with respect to variation in hydraulic conductively values associated with depth. Therefore, it is assumed that K_{v2} = K_{h2}, and m_2 = 1 for both pits.

Table 2 Summary of Analytical Solution Parameter Values

Parameter	Leprechaun Pit	Marathon Pit
Effective Pit Radius - r_p (m)	208	256
Saturated Aquifer Thickness - h_0 (m)	317.5	371.5
Groundwater Recharge Flux - W (m/s)	1.96 x 10^{-9}	1.96 x 10^{-9}
K - Hydraulic Conductivity (m/s)		
- horizontal (K_{h1} = K_{h2})	5.88 x 10^{-8}	4.49 x 10^{-8}
- vertical (K_{v2} = K_{h2})	5.88 x 10^{-8}	4.49 x 10^{-8}
r_0 - radius of influence (m)	1,382	1,486

4.2.3 Groundwater Inflow Results

The groundwater inflow estimates for each pit are summarized in Table 3. The total groundwater inflow calculated for the Leprechaun pit is approximately 2,331 m^3/day with approximately 992 m^3/day originating from Zone 1 (pit walls) and 1,339 m^3/day originating from Zone 2 (pit bottom). The total groundwater inflow calculated for the Marathon pit is approximately 2,615 m^3/day with approximately 1,139 m^3/day originating from Zone 1 (pit walls) and 1,476 m^3/day originating from Zone 2 (pit bottom). For both pits, the calculated groundwater inflows were larger through the bottom of the pit compared with inflows from the side of the pit.

4.3 Total Pit Inflow Results

The estimated total daily inflow rates to the Leprechaun and Marathon pits are summarized in Table 3, and includes the estimates of inflow associated with direct prescription and groundwater seepage from bedrock.

By combining the estimated inflow from direct precipitation of 2,236 m^3/day for the Leprechaun pit, and 2,839 m^3/day for the Marathon Pit, to the groundwater inflow estimates provided above, a total average daily inflow rate of 4,568 m^3/day (3,172 L/min or 838 USgal/min) is calculated for the final configuration of the Leprechaun pit, and 5,454 m^3/day (3,788 L/min or 1,000 USgal/min) is calculated for the final pit configuration of the Marathon pit. These estimated total daily inflow rates are considered to be manageable with conventional dewatering equipment

Table 3 Total Pit Inflow Results

Pit	Pit Area (m^2)	Direct Precipitation Inflow (m^3/day)	Groundwater Inflow (m^3/day)	Total Pit Inflow (m^3/day)
Leprechaun	6.6 x 10^5	2,236	2,331 (Zone 1 – 992; Zone 2 – 1,339)	4,568
Marathon	8.4 x 10^5	2,839	2,615 (Zone 1 – 1,139; Zone 2 – 1,476)	5,454

The total pit inflow estimates presented above suggest that inflows from direct precipitation and groundwater will provide near equivalent contributions to the total inflow amounts estimated for each pit. Further, a slightly higher total inflow rate is estimated for the Marathon pit. This higher estimated inflow rate for the Marathon pit is solely attributed to its larger geometry, since its bulk rock mass hydraulic conductivity was slightly less than that determined for the Leprechaun pit.

The calculated inflow estimates presented above represent long-term, average rates under a steady-state, full pit development scenario. It is expected that initial inflow rates into the pits will be higher than that estimated as the rock mass dewaters under higher horizontal hydraulic gradients, and overtime as the pit is developed the flow rates will relax and end–up at the steady-state inflow rates presented here.

Further, there are a number of assumptions under the Marinelli and Niccoli solution used to estimate groundwater inflows that are not met for the two pits that may result in actual inflow rates different from that presented here, including the following:

- The Marinelli and Niccoli solution assumes a horizontal water table, and uniformly distributed recharge within the radius of influence that is fully captured by the pit. The two pits are in fact situated within areas of moderately to strongly-sloping topography, which is expected to result in a non-uniform, and possibly reduced radius of influence particularly in up-gradient areas to the southeast of both pits. A reduction in the radius of influence associated with the sloping terrain in the vicinity of the pits would tend to result in lower groundwater recharge flux and consequently lower groundwater inflow rates than that estimated.
- The Marinelli and Niccoli solution only considers recharge due to infiltration from precipitation. It is possible that inflow rates higher than that calculated may occur as the radius of influence reaches out to various surface water features within and surrounding the pit footprints and these water bodies become additional sources of recharge not accounted for in the Marinelli and Niccoli solution. In particular the calculated radius of

influence for the Leprechaun pit appears to extend out to Victoria Lake, and the calculated radius of influence for the Marathon pit appears to extend out to Victoria River. Depending on the hydraulic connectively of these two pits with these surface water bodies through various structural features (i.e., faults, fractures and shear zones), it is possible that these could provide significant sources of recharge and result in higher pit inflow rates than that estimated.

- The final depths of the two pits are not equal – the Leprechaun Pit bottom is planned for 68 masl; while the Marathon Pit bottom is planned for 34 m below sea level. This will produce a 102 m difference in elevation over an approximate 2 km separation. This will create a moderate horizontal hydraulic gradient from the Leprechaun Pit toward the Marathon Pit of about 0.05 m/m. This component of hydraulic connection between the pits is not accounted for in the preliminary analytical solution presented here.

While the physical and hydrogeological settings of the two pits are not in strict agreement with the assumptions associated with the Marinelli and Niccoli solution, as discussed above, the provided inflow estimates are considered reasonable to provide preliminary, order of magnitude estimates of pit inflows that can be used for early stages of mine planning. A more refined estimate of pit inflows for use in design of the pit dewatering systems should include a more comprehensive assessment of hydraulic conductivity for each bedrock type, account for permeable structural features, and consider the potential recharge contributions from nearby surface water bodies.

5.0 CLOSURE

This report has been prepared for the sole benefit of our client, Terrane Geoscience Incorporated. The report may not be relied upon by any other person or entity without the express written consent of GEMTEC Consulting Engineers and Scientist Limited and our client, Terrane Geoscience Incorporated.

Any use that a third party makes of this report, or any reliance or decisions made based on it, is the responsibility of such third parties. GEMTEC Consulting Engineers and Scientist Limited accepts no responsibility for damages, if any, suffered by any third party as a result of decisions made or actions based on this report.

The conclusions presented represent the best technical judgment of GEMTEC Consulting Engineers and Scientist Limited based on current engineering and scientific practices and environmental standards at the time the work was performed. The conclusions are based on the site conditions encountered at the time the work was performed at the testing locations, and can only be extrapolated to an undefined limited area around these locations.

Should additional information become available, GEMTEC Consulting Engineers and Scientist Limited requests that this information be brought to our attention so that we may re-assess the conclusions presented herein.

We trust this report provides sufficient information for your present purposes. If you have any questions concerning this report, please do not hesitate to contact the undersigned.

Respectfully Submitted,

GEMTEC Consulting Engineers and Scientist Limited

Carolyn Anstey-Moore, M.Sc., M.A.Sc., P.Geo.

6.0 REFERENCES

Freeze, R.A. & Cherry, J.A. (1979). *Groundwater*. Prentice Hall: Englewood Cliffs, NJ.

Marinelli, F. & Niccoli, W.L. (2000). *Simple Analytical Equations for Estimating Groundwater Inflow to a Mine Pit*. Ground Water, Vol 2, No 3 (March – April, 2000), pages 311 – 314.

Stantec Consulting Limited (Stantec) (2017a). *Valentine Lake Project: Baseline Hydrology and Surface Water Quality Monitoring Program*. Prepared for Marathon Gold Corporation, December 1, 2017.

Stantec (2017b). *Valentine Lake Project: Preliminary Baseline Hydrogeology Assessment*. Prepared for Marathon Gold Corporation, December 15, 2017.

Stantec (2019). Valentine Lake Project: Preliminary Baseline Hydrogeology Assessment, Water Level Data. Prepared for Marathon Gold Corporation, January 18, 2019.

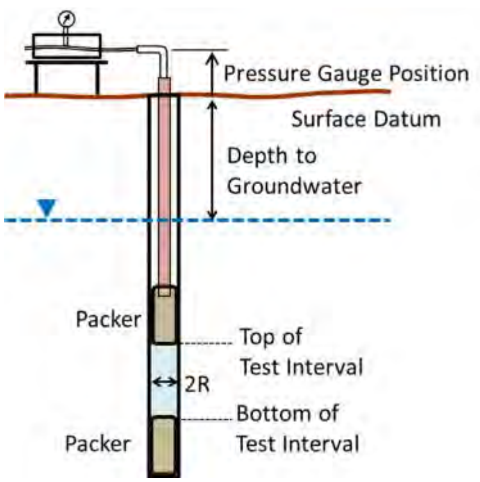
APPENDIX A

Packer Test Analysis Reports

GEMTEC Consulting Engineers and Scientists	**Lugeon Test Analysis Report**
	Project: Valentine Lake Pre-Feasibility Geotechnical Investigation
	Number: 80047.02
	Client: Terrane Geoscience Inc.

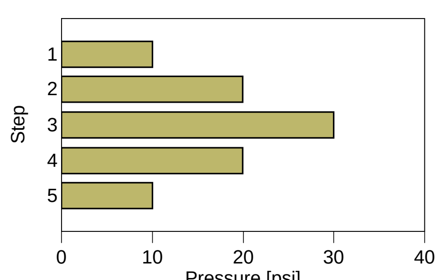
Location: Valentine Lake, NL	Lugeon Test: PT1	Tested bore: VL-GT-19-01
Test Conducted by: Terrane Geoscience Inc.		Test Date: 8/11/2019
Analysis Performed by: C. Anstey-Moore		Analysis Date: 10/21/2019

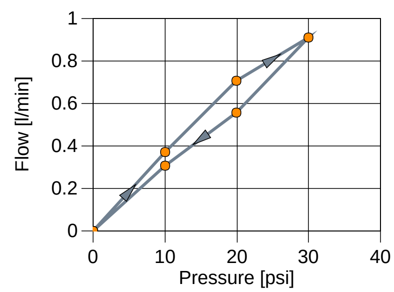
Lithology: Fine to medium-grained, grey to green Trondhjemite.

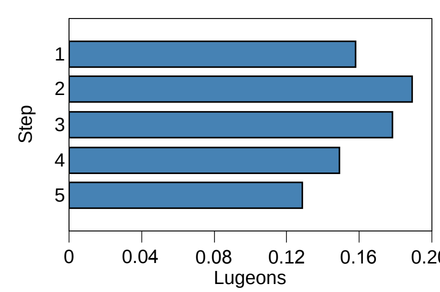


Top of Test Interval: 30.30 m
Bottom of Test Interval: 50.00 m
Length of Test Interval: 19.70 m
Gauge Position: 1.80 m
Depth to Groundwater: 3.50 m
Radius of Test Section: 0.05 m
Dip of bore: 35.19° from vertical
Vertical Top of Test Interval: 24.76 m
Vertical Bottom of Test Interval: 40.86 m

Step	Pressure [psi]	Flow Meter Readings [l]										Average Flow Rate [l/min]	Hydraulic Conductivity		
		1	2	3	4	5	6	7	8	9	10		[m/s]	[m/d]	Lugeon
1	10	5856.50	5856.93	5857.33	5857.72	5858.09	5858.45	5858.82	5859.17	5859.51	5859.85	0.37	2.45×10^{-8}	0.0021	0.16
2	20	5860.50	5861.29	5862.04	5862.76	5863.45	5864.15	5864.84	5865.50	5866.16		0.71	2.96×10^{-8}	0.0026	0.19
3	30	5866.95	5867.92	5868.85	5869.76	5870.65	5871.53	5872.41				0.91	2.80×10^{-8}	0.0024	0.18
4	20	5873.00	5873.55	5874.11	5874.67	5875.23						0.56	2.33×10^{-8}	0.0020	0.15
5	10	5875.30	5875.55	5875.84	5876.13	5876.45	5876.76	5877.09	5877.42	5877.75		0.31	2.01×10^{-8}	0.0017	0.13
												Average	2.51×10^{-8}	0.0022	0.16







Laminar
Lugeon: 0.16
Hydraulic Conductivity: 2.51E-8 m/s
Hydraulic Conductivity: 0.0022 m/d

Performed using a single packer test assembly. The packer test was performed as the borehole was advanced, with the bottom of the test interval bounded by the bottom of the drilled section of the borehole.

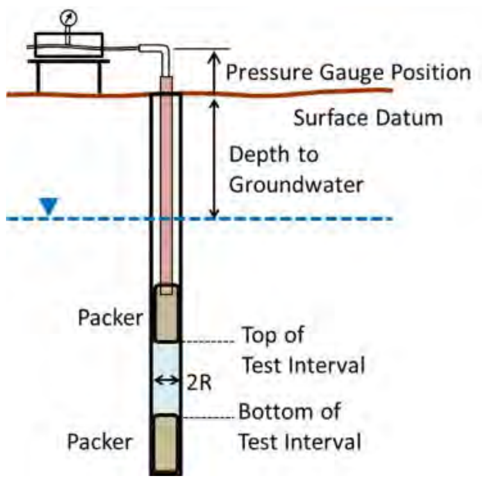
Note – although the Lugeon and Flow vs Pressure patterns suggest non-laminar flow, the arithmetic mean (average) of the hydraulic conductivity (K) values determined for each step is used to calculate the representative K value for the tested interval.

Lugeon Test Analysis Report

Project:	Valentine Lake Pre-Feasibility Geotechnical Investigation
Number:	80047.02
Client:	Terrane Geoscience Inc.

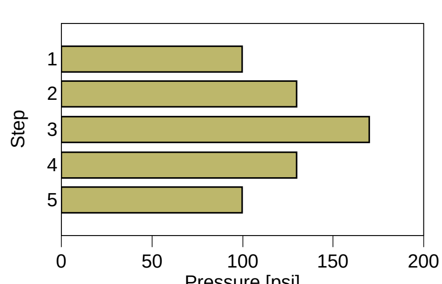
Location: Valentine Lake, NL	Lugeon Test: PT2	Tested bore: VL-GT-19-01
Test Conducted by: Terrane Geoscience Inc.		Test Date: 8/13/2019
Analysis Performed by: C. Anstey-Moore		Analysis Date: 10/22/2019

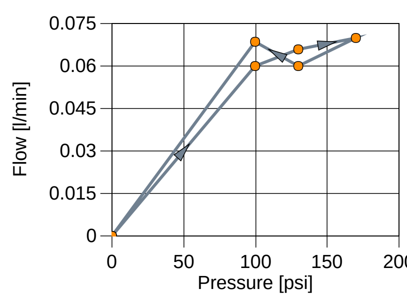
Lithology: Fine to medium-grained, grey to green Trondhjemite.

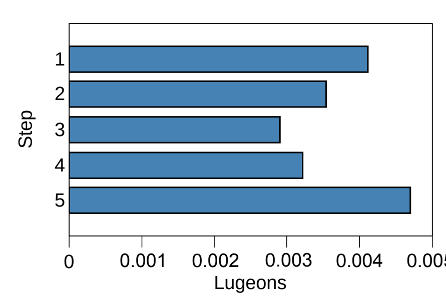


Top of Test Interval: 150.30 m
Bottom of Test Interval: 170.00 m
Length of Test Interval: 19.70 m
Gauge Position: 1.70 m
Depth to Groundwater: 3.50 m
Radius of Test Section: 0.05 m
Dip of bore: 35.19° from vertical
Vertical Top of Test Interval: 122.83 m
Vertical Bottom of Test Interval: 138.93 m

Step	Pressure [psi]	Flow Meter Readings [l]								Average Flow Rate [l/min]	Hydraulic Conductivity		
		1	2	3	4	5	6	7	8		[m/s]	[m/d]	Lugeon
1	100	5884.33	5884.39	5884.45	5884.51					0.06	6.44×10^{-10}	0.000056	0.0041
2	130	5884.70	5884.78	5884.85	5884.91	5884.97	5885.03			0.07	5.54×10^{-10}	0.000048	0.0035
3	170	5885.25	5885.33	5885.40	5885.47	5885.53	5885.60	5885.67	5885.74	0.07	4.55×10^{-10}	0.000039	0.0029
4	130	5885.65	5885.71	5885.77	5885.83					0.06	5.04×10^{-10}	0.000044	0.0032
5	100	5885.75	5885.81	5885.88	5885.95	5886.01	5886.09	5886.16	5886.23	0.07	7.36×10^{-10}	0.000064	0.0047
										Average	5.79×10^{-10}	0.000050	0.0037







Laminar
Lugeon: 0.0037
Hydraulic Conductivity: 5.79E-10 m/s
Hydraulic Conductivity: 0.000050 m/d

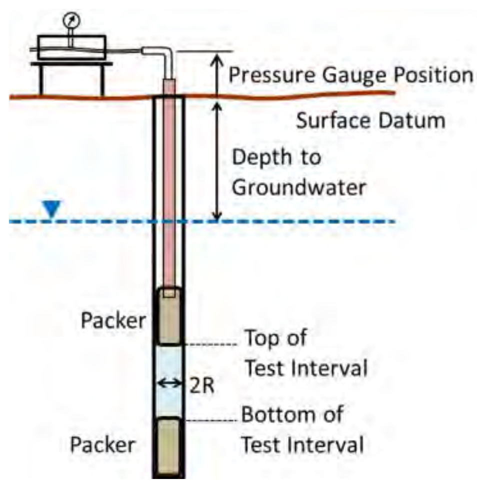
Performed using a single packer test assembly. The packer test was performed as the borehole was advanced, with the bottom of the test interval bounded by the bottom of the drilled section of the borehole.

Note – although the Lugeon and Flow vs Pressure patterns suggest non-laminar flow, the arithmetic mean (average) of the hydraulic conductivity (K) values determined for each step is used to calculate the representative K value for the tested interval.

CONSULTING ENGINEERS AND SCIENTISTS	**Lugeon Test Analysis Report**
	Project: Valentine Lake Pre-Feasibility Geotechnical Investigation
	Number: 80047.02
	Client: Terrane Geoscience Inc.

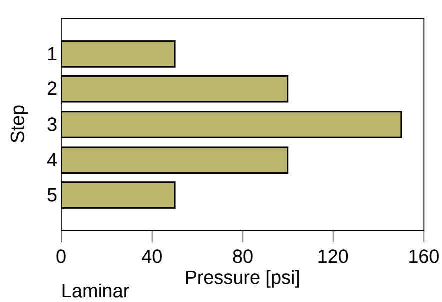
Location: Valentine Lake, NL	Lugeon Test: PT3	Tested bore: VL-GT-19-01
Test Conducted by: Terrane Geoscience Inc.		Test Date: 8/16/2019
Analysis Performed by: C. Anstey-Moore		Analysis Date: 10/22/2019

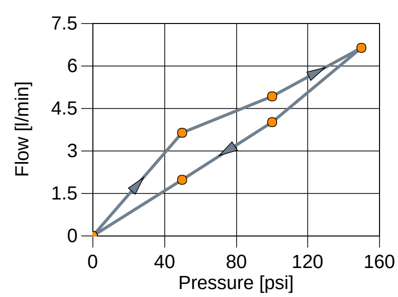
Lithology: Medium-grained, peach Trondhjemite. Fault structure identified from 350 - 353 m; associated with good rock quality.

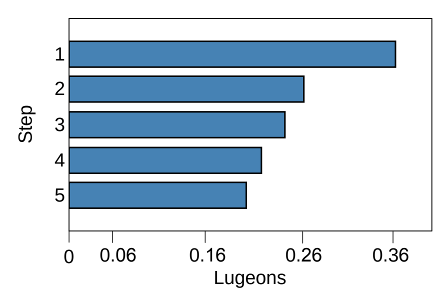


Top of Test Interval: 348.30 m
Bottom of Test Interval: 374.00 m
Length of Test Interval: 25.70 m
Gauge Position: 1.50 m
Depth to Groundwater: 3.50 m
Radius of Test Section: 0.05 m
Dip of bore: 35.19° from vertical
Vertical Top of Test Interval: 284.65 m
Vertical Bottom of Test Interval: 305.65 m

Step	Pressure [psi]	Flow Meter Readings [l]											Average Flow Rate [l/min]	Hydraulic Conductivity		
		1	2	3	4	5	6	7	8	9	10	11		[m/s]	[m/d]	Lugeon
1	50	5927.50	5931.46	5935.44	5939.33	5943.13	5946.85	5950.48	5954.00	5957.47	5960.72	5963.90	3.64	5.88×10^{-8}	0.0051	0.36
2	100	5967.00	5972.47	5977.67	5982.76	5987.72	5992.64	5997.43	6002.20	6006.91	6011.55	6016.16	4.92	4.24×10^{-8}	0.0037	0.26
3	150	6021.50	6028.48	6035.42	6041.94	6048.50	6055.35	6061.98	6068.50	6074.96	6081.35	6087.84	6.63	3.90×10^{-8}	0.0034	0.24
4	100	6091.00	6095.05	6099.10	6103.10	6107.10	6111.10						4.02	3.46×10^{-8}	0.0030	0.21
5	50	6112.20	6113.99	6115.90	6117.85	6119.85	6121.88	6123.90	6125.94	6127.98	6130.02		1.98	3.20×10^{-8}	0.0028	0.20
													Average	4.14×10^{-8}	0.0036	0.25







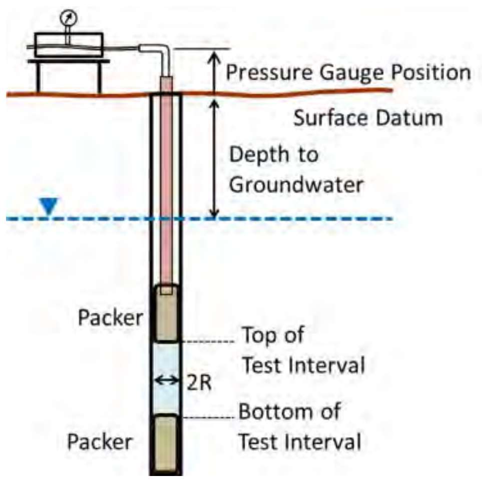
Laminar
Lugeon: 0.25
Hydraulic Conductivity: 4.14E-8 m/s
Hydraulic Conductivity: 0.0036 m/d

Performed using a single packer test assembly. The packer test was performed as the borehole was advanced, with the bottom of the test interval bounded by the bottom of the drilled section of the borehole.

Note – although the Lugeon and Flow vs Pressure patterns suggest non-laminar flow, the arithmetic mean (average) of the hydraulic conductivity (K) values determined for each step is used to calculate the representative K value for the tested interval.

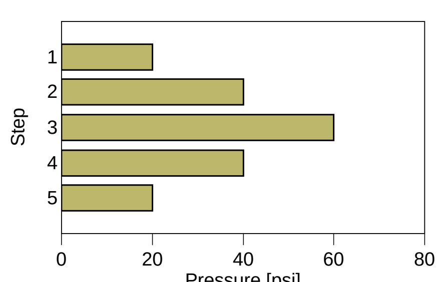
GEMTEC Consulting Engineers and Scientists	**Lugeon Test Analysis Report**
	Project: Valentine Lake Pre-Feasibility Geotechnical Investigation
	Number: 80047.02
	Client: Terrane Geoscience Inc.

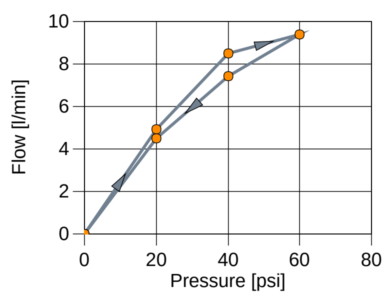
Location: Valentine Lake, NL	Lugeon Test: PT1	Tested bore: VL-GT-19-02
Test Conducted by: Terrane Geoscience Inc.		Test Date: 8/7/2019
Analysis Performed by: C. Anstey-Moore		Analysis Date: 10/22/2019
Lithology: Fine to medium-grained, beige to grey-green Trondhjemite with minor QTP veins.		

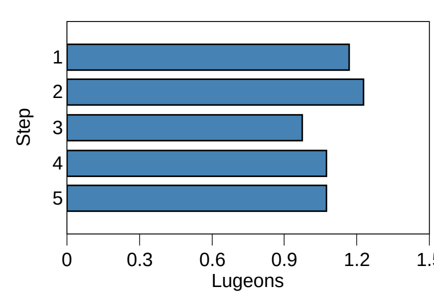


Top of Test Interval: 354.30 m
Bottom of Test Interval: 374.00 m
Length of Test Interval: 19.70 m
Gauge Position: 1.30 m
Depth to Groundwater: 6.40 m
Radius of Test Section: 0.05 m
Dip of bore: 44.77° from vertical
Vertical Top of Test Interval: 251.53 m
Vertical Bottom of Test Interval: 265.52 m

Step	Pressure [psi]	Flow Meter Readings [l]											Average Flow Rate [l/min]	Hydraulic Conductivity		
		1	2	3	4	5	6	7	8	9	10	11		[m/s]	[m/d]	Lugeon
1	20	2572.00	2577.30	2582.28	2587.28	2592.21	2597.10	2601.95	2606.78	2611.58	2616.36	2621.11	4.91	1.83×10^{-7}	0.016	1.17
2	40	2661.00	2671.37	2680.65	2689.40	2697.90	2706.18	2714.37	2722.38	2730.30	2738.19	2746.05	8.51	1.92×10^{-7}	0.017	1.23
3	60	2777.00	2786.87	2796.55	2806.01	2815.45	2824.87	2834.20	2843.50	2852.75	2861.90	2871.00	9.40	1.53×10^{-7}	0.013	0.98
4	40	2866.00	2874.27	2881.85	2888.97	2896.40	2903.65	2910.81	2918.25	2925.73	2933.15	2940.44	7.44	1.68×10^{-7}	0.015	1.08
5	20	2933.50	2937.83	2942.19	2946.62	2951.09	2955.58	2960.15	2964.74	2969.35	2973.97	2978.60	4.51	1.68×10^{-7}	0.015	1.07
													Average	1.73×10^{-7}	0.015	1.10







Laminar
Lugeon: 1.10
Hydraulic Conductivity: 1.73E-7 m/s
Hydraulic Conductivity: 0.015 m/d

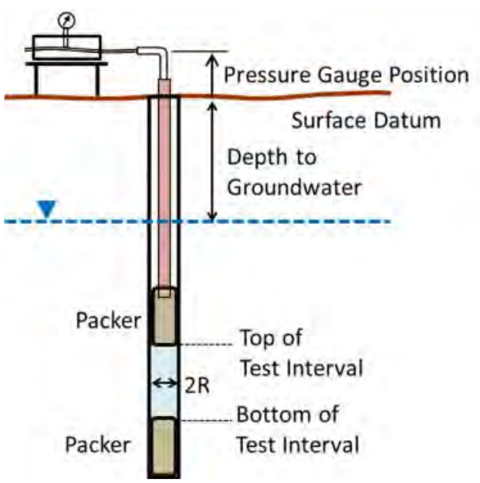
Performed using a single packer test assembly. The packer test was performed as the borehole was advanced, with the bottom of the test interval bounded by the bottom of the drilled section of the borehole.

Note – although the Lugeon and Flow vs Pressure patterns suggest non-laminar flow, the arithmetic mean (average) of the hydraulic conductivity (K) values determined for each step is used to calculate the representative K value for the tested interval.

GEMTEC
CONSULTING ENGINEERS AND SCIENTISTS

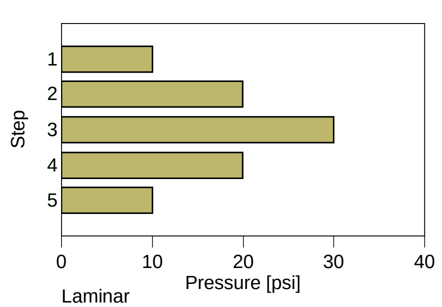
Lugeon Test Analysis Report
Project: Valentine Lake Pre-Feasibility Geotechnical Investigation
Number: 80047.02
Client: Terrane Geoscience Inc.

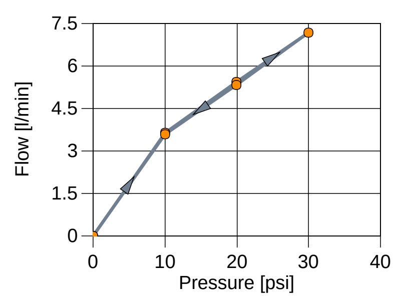
Location: Valentine Lake, NL	Lugeon Test: PT1	Tested bore: VL-GT-19-03
Test Conducted by: Terrane Geoscience Inc.		Test Date: 8/8/2019
Analysis Performed by: C. Anstey-Moore		Analysis Date: 10/22/2019
Lithology: Fine to medium-grained, peach Trondhjemite with minor QTP veins.		

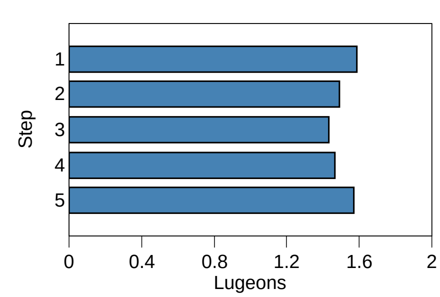


Top of Test Interval: 12.30 m
Bottom of Test Interval: 32.00 m
Length of Test Interval: 19.70 m
Gauge Position: 1.30 m
Depth to Groundwater: 3.50 m
Radius of Test Section: 0.05 m
Dip of bore: 25.07° from vertical
Vertical Top of Test Interval: 11.14 m
Vertical Bottom of Test Interval: 28.99 m

Step	Pressure [psi]	Flow Meter Readings [l]											Average Flow Rate [l/min]	Hydraulic Conductivity		
		1	2	3	4	5	6	7	8	9	10	11		[m/s]	[m/d]	Lugeon
1	10	3062.00	3065.77	3069.50	3073.11	3076.78	3080.38	3084.00	3087.61	3091.18	3094.74	3098.30	3.63	2.49×10^{-7}	0.021	1.59
2	20	3103.00	3108.40	3113.90	3119.37	3124.86	3130.31	3135.75	3141.12	3146.48	3151.85		5.43	2.33×10^{-7}	0.020	1.49
3	30	3160.00	3167.40	3174.62	3181.81	3188.98	3196.20	3203.40	3210.52	3217.62	3224.65	3231.70	7.17	2.25×10^{-7}	0.019	1.43
4	20	3237.50	3242.95	3248.30	3253.60	3258.90	3264.20						5.34	2.30×10^{-7}	0.020	1.47
5	10	3269.00	3272.65	3276.30	3279.90	3283.50	3287.09	3290.66	3294.20	3297.75	3301.29		3.59	2.46×10^{-7}	0.021	1.57
													Average	2.36×10^{-7}	0.020	1.51







Laminar
Lugeon: 1.51
Hydraulic Conductivity: 2.36E-7 m/s
Hydraulic Conductivity: 0.020 m/d

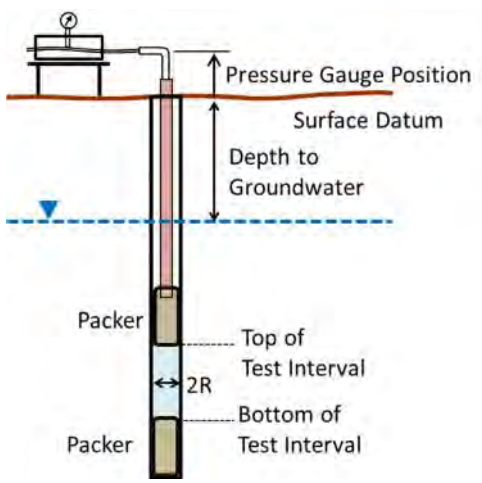
Performed using a single packer test assembly. The packer test was performed as the borehole was advanced, with the bottom of the test interval bounded by the bottom of the drilled section of the borehole.

Note – the Lugeon and Flow vs Pressure patterns suggest laminar flow conditions, and the arithmetic mean (average) of the hydraulic conductivity (K) values determined for each step is used to calculate the representative K value for the tested interval.

GEMTEC Consulting Engineers and Scientists	**Lugeon Test Analysis Report**
	Project: Valentine Lake Pre-Feasibility Geotechnical Investigation
	Number: 80047.02
	Client: Terrane Geoscience Inc.

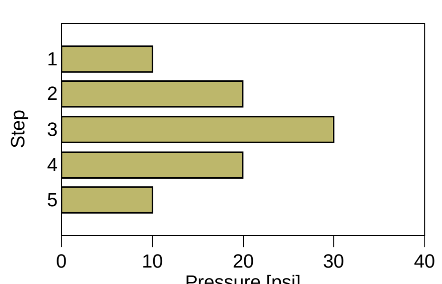
Location: Valentine Lake, NL	Lugeon Test: PT2	Tested bore: VL-GT-19-03
Test Conducted by: Terrane Geoscience Inc.		Test Date: 8/9/2019
Analysis Performed by: C. Anstey-Moore		Analysis Date: 10/22/2019

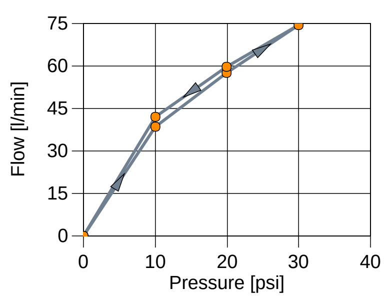
Lithology: Green-grey Mafic Dyke, with Trondhjemite and minor QTP veins. Fault structure identified from 44 - 50 m; associated with fair - poor rock quality.

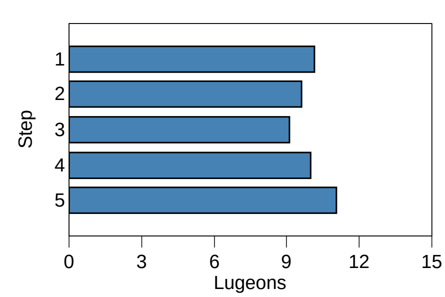


Top of Test Interval: 45.30 m
Bottom of Test Interval: 77.00 m
Length of Test Interval: 31.70 m
Gauge Position: 1.70 m
Depth to Groundwater: 3.50 m
Radius of Test Section: 0.05 m
Dip of bore: 25.07° from vertical
Vertical Top of Test Interval: 41.03 m
Vertical Bottom of Test Interval: 69.75 m

Step	Pressure [psi]	Flow Meter Readings [l]							Average Flow Rate [l/min]	Hydraulic Conductivity		
		1	2	3	4	5	6	7		[m/s]	[m/d]	Lugeon
1	10	3773.00	3812.00	3850.80	3889.00	3927.80	3966.00		38.60	1.72×10^{-6}	0.148	10.15
2	20	4114.00	4171.00	4229.00	4286.50	4344.00			57.50	1.62×10^{-6}	0.140	9.60
3	30	4560.00	4634.00	4708.00	4782.50	4857.00	4932.00	5007.00	74.50	1.54×10^{-6}	0.133	9.11
4	20	5206.00	5266.50	5326.00	5386.00	5445.50	5505.00		59.80	1.69×10^{-6}	0.146	9.99
5	10	5692.50	5734.50	5776.50	5818.50				42.00	1.87×10^{-6}	0.161	11.05
									Average	1.69×10^{-6}	0.146	9.98







Laminar
Lugeon: 9.98
Hydraulic Conductivity: 1.69E-6 m/s
Hydraulic Conductivity: 0.146 m/d

Performed using a single packer test assembly. The packer test was performed as the borehole was advanced, with the bottom of the test interval bounded by the bottom of the drilled section of the borehole.

Note – the Lugeon and Flow vs Pressure patterns suggest laminar flow conditions, and the arithmetic mean (average) of the hydraulic conductivity (K) values determined for each step is used to calculate the representative K value for the tested interval.

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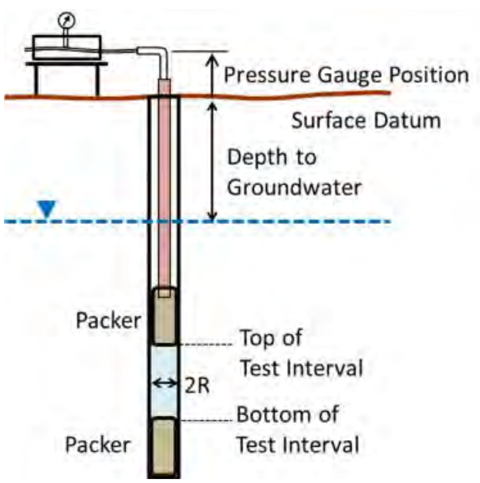
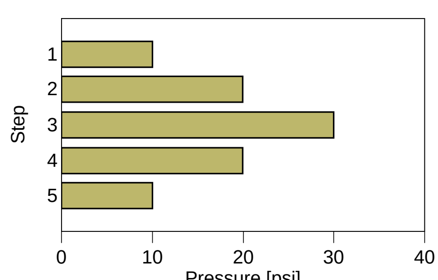
Lugeon Test Analysis Report
Project: Valentine Lake Pre-Feasibility Geotechnical Investigation
Number: 80047.02
Client: Terrane Geoscience Inc.

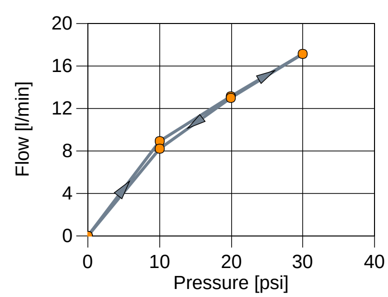
Location: Valentine Lake, NL	Lugeon Test: PT1	Tested bore: MA-GT-19-05
Test Conducted by: Terrane Geoscience Inc.		Test Date: 7/18/2019
Analysis Performed by: C. Anstey-Moore		Analysis Date: 10/22/2019
Lithology: Medium-grained, dark green Mafic Dyke.		

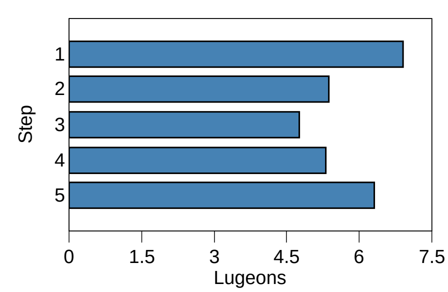


Top of Test Interval: 45.30 m
Bottom of Test Interval: 62.00 m
Length of Test Interval: 16.70 m
Gauge Position: 1.40 m
Depth to Groundwater: -0.50 m
Radius of Test Section: 0.05 m
Dip of bore: 29.75° from vertical
Vertical Top of Test Interval: 39.33 m
Vertical Bottom of Test Interval: 53.83 m

Step	Pressure [psi]	Flow Meter Readings [l]											Average Flow Rate [l/min]	Hydraulic Conductivity		
		1	2	3	4	5	6	7	8	9	10	11		[m/s]	[m/d]	Lugeon
1	10	346.00	355.21	364.31	373.36	382.38	391.38	400.29	409.16	418.07	426.87	435.63	8.96	1.05×10^{-6}	0.091	6.90
2	20	495.00	508.61	521.72	534.90	548.00	561.11	574.22	587.22	600.22			13.15	8.18×10^{-7}	0.071	5.37
3	30	633.00	650.35	667.50	684.55	701.65	718.68	735.70					17.12	7.24×10^{-7}	0.063	4.75
4	20	770.00	783.30	796.28	809.21	822.15	835.09	848.00					13.00	8.08×10^{-7}	0.070	5.31
5	10	863.00	871.23	879.39	887.55	895.79	903.98	912.19	920.40				8.20	9.62×10^{-7}	0.083	6.31
													Average	8.73×10^{-7}	0.075	5.73







Laminar
Lugeon: 5.73
Hydraulic Conductivity: 8.73E-7 m/s
Hydraulic Conductivity: 0.075 m/d

Performed using a single packer test assembly. The packer test was performed as the borehole was advanced, with the bottom of the test interval bounded by the bottom of the drilled section of the borehole.

Note – although the Lugeon and Flow vs Pressure patterns suggest non-laminar flow, the arithmetic mean (average) of the hydraulic conductivity (K) values determined for each step is used to calculate the representative K value for the tested interval.

Flowing artesian conditions; static water level not determined. Estimated -0.5 m below ground surface for purposes of analysis.

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Lugeon Test Analysis Report

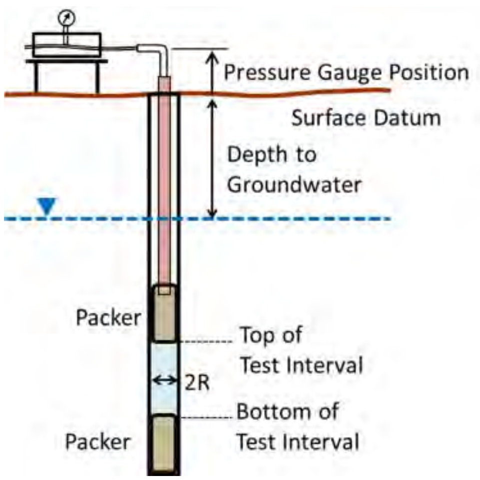
Project: Valentine Lake Pre-Feasibility Geotechnical Investigation

Number: 80047.02

Client: Terrane Geoscience Inc.

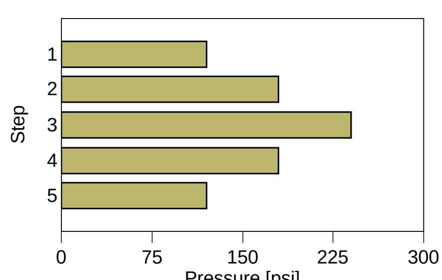
Location: Valentine Lake, NL	Lugeon Test: PT2	Tested bore: MA-GT-19-05
Test Conducted by: Terrane Geoscience Inc.		Test Date: 7/21/2019
Analysis Performed by: C. Anstey-Moore		Analysis Date: 10/22/2019

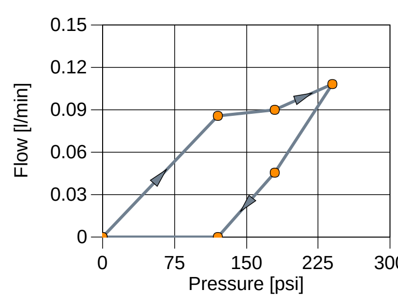
Lithology: Coarse-grained, pale blue Quartz Eye Porphyry.

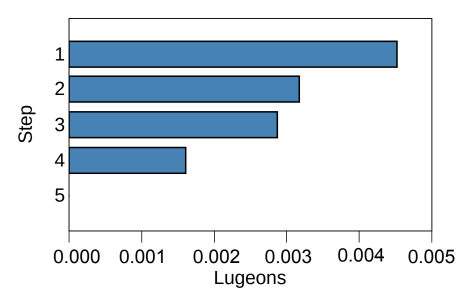


Top of Test Interval: 273.30 m
Bottom of Test Interval: 296.00 m
Length of Test Interval: 22.70 m
Gauge Position: 1.40 m
Depth to Groundwater: -0.50 m
Radius of Test Section: 0.05 m
Dip of bore: 29.75° from vertical
Vertical Top of Test Interval: 237.28 m
Vertical Bottom of Test Interval: 256.99 m

Step	Pressure [psi]	Flow Meter Readings [l]								Average Flow Rate [l/min]	Hydraulic Conductivity		
		1	2	3	4	5	6	7	8		[m/s]	[m/d]	Lugeon
1	120	946.60	946.73	946.82	946.91	946.99	947.06	947.13	947.20	0.09	7.24×10^{-10}	0.000063	0.0045
2	180	947.90	948.01	948.11	948.20	948.29	948.37	948.45	948.53	0.09	5.09×10^{-10}	0.000044	0.0032
3	240	949.20	949.34	949.45	949.55	949.66	949.75	949.85		0.11	4.60×10^{-10}	0.000040	0.0029
4	180	949.47	949.49	949.55	949.59	949.63	949.68	949.74	949.79	0.05	2.58×10^{-10}	0.000022	0.0016
5	120	949.37	949.37	949.37						0.00	0.00×10^{0}	0.000000	0.0000
										Average	3.90×10^{-10}	0.000034	0.0024







Laminar
Lugeon: 0.0024
Hydraulic Conductivity: 3.90E-10 m/s
Hydraulic Conductivity: 0.000034 m/d

Performed using a single packer test assembly. The packer test was performed as the borehole was advanced, with the bottom of the test interval bounded by the bottom of the drilled section of the borehole.

Note – although the Lugeon and Flow vs Pressure patterns suggest non-laminar flow, the arithmetic mean (average) of the hydraulic conductivity (K) values determined for each step is used to calculate the representative K value for the tested interval.

Flowing artesian conditions; static water level not determined. Estimated -0.5 m below ground surface for purposes of analysis.

Lugeon Test Analysis Report

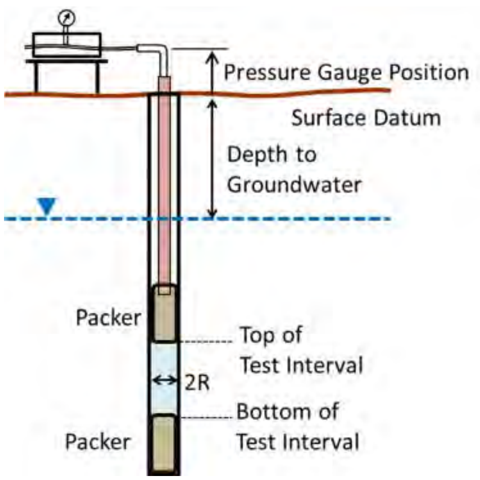
Project: Valentine Lake Pre-Feasibility Geotechnical Investigation

Number: 80047.02

Client: Terrane Geoscience Inc.

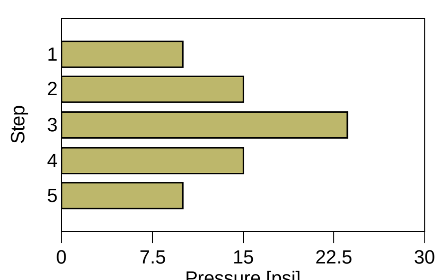
Location: Valentine Lake, NL	Lugeon Test: PT1	Tested bore: MA-GT-19-06
Test Conducted by: Terrane Geoscience Inc.		Test Date: 7/25/2019
Analysis Performed by: C. Anstey-Moore		Analysis Date: 10/22/2019

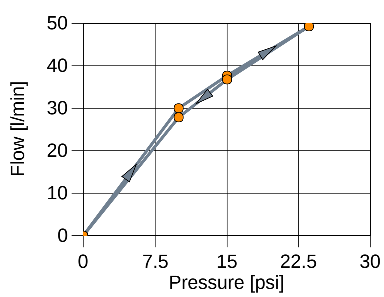
Lithology: Dark green, strongly foliated Conglomerate.



Top of Test Interval: 30.30 m
Bottom of Test Interval: 50.00 m
Length of Test Interval: 19.70 m
Gauge Position: 1.45 m
Depth to Groundwater: 3.50 m
Radius of Test Section: 0.05 m
Dip of bore: 37.55° from vertical
Vertical Top of Test Interval: 24.02 m
Vertical Bottom of Test Interval: 39.64 m

Step	Pressure [psi]	Flow Meter Readings [l]								Average Flow Rate [l/min]	Hydraulic Conductivity		
		1	2	3	4	5	6	7	8		[m/s]	[m/d]	Lugeon
1	10	1102.00	1132.00	1161.80	1191.80	1221.80	1251.80			29.96	2.03×10^{-6}	0.2	12.94
2	15	1300.00	1337.80	1375.50	1413.00	1450.50	1488.00			37.60	1.97×10^{-6}	0.2	12.56
3	24	1570.00	1617.70	1666.50	1714.50	1764.50	1815.00	1865.50		49.25	1.85×10^{-6}	0.2	11.83
4	15	1940.00	1977.00	2013.70	2050.50	2087.00	2123.50	2160.50	2197.50	36.79	1.92×10^{-6}	0.2	12.29
5	10	2240.00	2267.70	2295.60	2323.50	2351.50	2379.60	2407.50		27.92	1.89×10^{-6}	0.2	12.06
										Average	1.93×10^{-6}	0.2	12.34





Laminar
Lugeon: 12.34
Hydraulic Conductivity: 1.93E-6 m/s
Hydraulic Conductivity: 0.167 m/d

Performed using a single packer test assembly. The packer test was performed as the borehole was advanced, with the bottom of the test interval bounded by the bottom of the drilled section of the borehole.

Note – the Lugeon and Flow vs Pressure patterns suggest laminar flow conditions, and the arithmetic mean (average) of the hydraulic conductivity (K) values determined for each step is used to calculate the representative K value for the tested interval.

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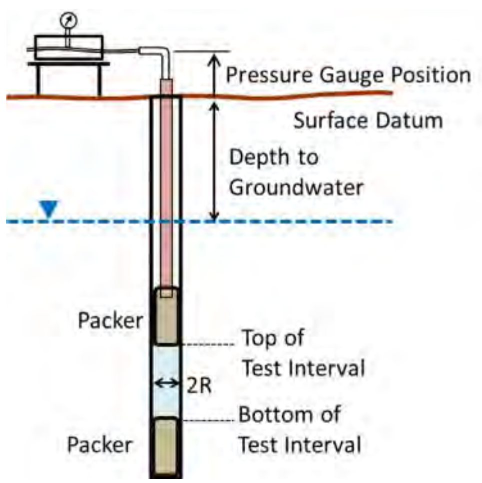
Lugeon Test Analysis Report	
Project:	Valentine Lake Pre-Feasibility Geotechnical Investigation
Number:	80047.02
Client:	Terrane Geoscience Inc.

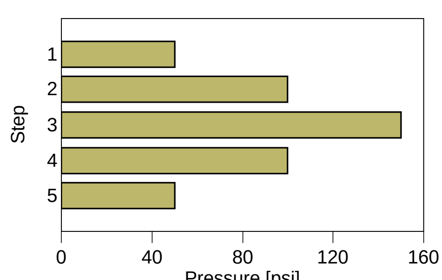
Location: Valentine Lake, NL	Lugeon Test: PT2	Tested bore: MA-GT-19-06
Test Conducted by: Terrane Geoscience Inc.		Test Date: 7/29/2019
Analysis Performed by: C. Anstey-Moore		Analysis Date: 10/22/2019

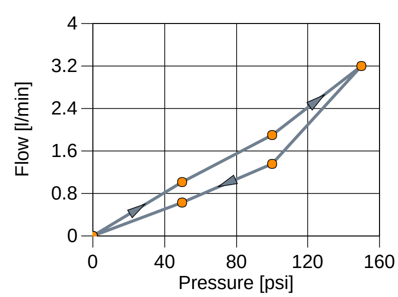
Lithology: Dark green, strongly foliated Conglomerate. Fault structure identified from 233 - 236 m; associated with good rock quality.

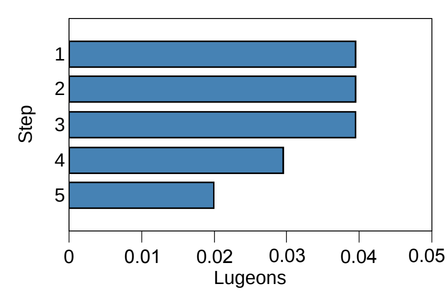


Top of Test Interval: 189.30 m
Bottom of Test Interval: 260.00 m
Length of Test Interval: 70.70 m
Gauge Position: 1.45 m
Depth to Groundwater: 3.50 m
Radius of Test Section: 0.05 m
Dip of bore: 37.55° from vertical
Vertical Top of Test Interval: 150.08 m
Vertical Bottom of Test Interval: 206.13 m

Step	Pressure [psi]	Flow Meter Readings [l]											Average Flow Rate [l/min]	Hydraulic Conductivity		
		1	2	3	4	5	6	7	8	9	10	11		[m/s]	[m/d]	Lugeon
1	50	2441.50	2442.75	2443.87	2444.93	2445.97	2446.98	2447.94	2448.88	2449.81	2450.70	2451.59	1.01	6.89×10^{-9}	0.00060	0.04
2	100	2457.00	2458.92	2460.83	2462.76	2464.76	2466.68	2468.59	2470.47	2472.31	2474.14	2475.94	1.89	6.89×10^{-9}	0.00060	0.04
3	150	2481.60	2485.08	2488.38	2491.65	2494.87	2498.05	2501.19	2504.32	2507.39	2510.47	2513.54	3.19	7.92×10^{-9}	0.00068	0.04
4	100	2513.30	2514.61	2515.97	2517.34	2518.69	2520.05						1.35	4.91×10^{-9}	0.00042	0.03
5	50	2515.00	2515.51	2516.03	2516.54	2517.63	2518.20	2518.80	2519.40	2520.00			0.63	4.27×10^{-9}	0.00037	0.02
													Average	6.18×10^{-9}	0.00053	0.03







Laminar
Lugeon: 0.03
Hydraulic Conductivity: 6.18E-9 m/s
Hydraulic Conductivity: 0.00053 m/d

Performed using a single packer test assembly. The packer test was performed as the borehole was advanced, with the bottom of the test interval bounded by the bottom of the drilled section of the borehole.

Note – although the Lugeon and Flow vs Pressure patterns suggest non-laminar flow, the arithmetic mean (average) of the hydraulic conductivity (K) values determined for each step is used to calculate the representative K value for the tested interval.

experience • knowledge • integrity

civil	civil
geotechnical	géotechnique
environmental	environnementale
field services	surveillance de chantier
materials testing	service de laboratoire des matériaux

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Design Sector	Bench Face Dip Direction	Kinematic Failure Modes (Friction Angle of 32°)[8] – Planar: Controlling Joint Sets	Planar: Max BFA[1]	Wedge: Controlling Joint Sets	Wedge: Max BFA	Topple: Controlling Joint Sets	Topple: Max BFA	Summary of Kinematic and Limit Equilibrium Analysis – Dominant Failure Mode	Bench POF%[2] (A / B)	Maximum Kinematic BFA (A / B)	Bench Design Summary – BFA	Bench Width[3] (m)	Bench Height[4] (m)	Comments
1	136	Secondary[5]	NC[6]	Secondary[5]	NC[6]	S_1	75	Toppling	23.0 / NA	75 / NA	71[9]	8.1	18	- Toppling instabilities returned the largest probability of failure; However, the bench geometry is controlled by the recommended maximum overall slope angle (OSA) of 47°. The maximum OSA is guided by benchmarking studies and experience at operating mines. - Kinematic analysis suggests that up to 80° benches are possible, however, benches in excess of 71° would increase the potential for overall slope instabilities and cause the OSA to exceed 47°. An opportunity to increase the BFA beyond 71° may be possible as more data becomes available.
2	164	Secondary[5]	NC[6]	Secondary[5]	NC[6]	S_1	74	Toppling	24.8 / NA	74 / NA	71[9]	8.1	18	- Toppling instabilities returned the largest probability of failure; However, the bench geometry is controlled by the recommended maximum OSA of 47°. The maximum OSA is guided by benchmarking studies and experience at operating mines. - Kinematic analysis suggests that up to 74° benches are possible, however, benches in excess of 71° would increase the potential for overall slope instabilities and cause the OSA to exceed 47°. An opportunity to increase the BFA beyond 71° may be possible as more data becomes available.
3	194	Secondary[5]	NC[6]	Secondary[5]	NC[6]	Secondary[5]	75	Toppling	14.4 / NA	75 / NA	71[9]	8.1	18	- Toppling instabilities returned the largest probability of failure; However, the bench geometry is controlled by the recommended maximum OSA of 47°. The maximum OSA is guided by benchmarking studies and experience at operating mines. - Kinematic analysis suggests that up to 80° benches are possible, however, benches in excess of 71° would increase the potential for overall slope instabilities and cause the OSA to exceed 47°. An opportunity to increase the BFA beyond 71° may be possible as more data becomes available.
4	241	Secondary[5]	NC[6]	Secondary[5]	NC[6]	Secondary[5]	75	Toppling	18.2 / NA	75 / NA	71[9]	8.1	18	- Toppling instabilities returned the largest probability of failure; However, the bench geometry is controlled by the recommended maximum OSA of 47°. The maximum OSA is guided by benchmarking studies and experience at operating mines. - Kinematic analysis suggests that up to 80° benches are possible, however, benches in excess of 71° would increase the potential for overall slope instabilities and cause the OSA to exceed 47°. An opportunity to increase the BFA beyond 71° may be possible as more data becomes available.
5	273	Secondary[5]	NC[6]	Secondary[5]	75	Secondary[5]	NC[6]	Wedge	12.3 / NA	75 / NA	71[9]	8.1	18	- Wedge instabilities returned the largest probability of failure; However, the bench geometry is controlled by the recommended maximum OSA of 47°. The maximum OSA is guided by benchmarking studies and experience at operating mines. - Kinematic analysis suggests that up to 80° benches are possible, however, benches in excess of 71° would increase the potential for overall slope instabilities and cause the OSA to exceed 47°. An opportunity to increase the BFA beyond 71° may be possible as more data becomes available.
6	316	S_1	NC[6]	S_1 vs. JS_1	75	Secondary[5]	NC[6]	Wedge	18.1 / NA	75 / NA	71[9]	8.1	18	- Wedge instabilities returned the largest probability of failure; However, the bench geometry is controlled by the recommended maximum OSA of 47°. The maximum OSA is guided by benchmarking studies and experience at operating mines. - Kinematic analysis suggests that up to 80° benches are possible, however, benches in excess of 71° would increase the potential for overall slope instabilities and cause the OSA to exceed 47°. An opportunity to increase the BFA beyond 71° may be possible as more data becomes available.
7	014	Secondary[5]	NC[6]	S_1 vs. Secondary	67	Secondary[5]	NC[6]	Wedge	24.6 / 20.2	67 / **75**	75[7]	8.1	18	- The kinematic failure mode controlling bench geometry in design sector 7 is wedge instabilities. Wedge sliding may occur on the pervasive discontinuity set S_1 in combination with secondary discontinuities. - Where kinematic analysis returned a POF > 25% for BFA < 75º, Limit-Equilibrium (L-E) analyses were conducted in SWedge (Rocscience 2019) to further evaluate the affect of wedge instability on the bench geometry.
8	071	Secondary[5]	NC[6]	S_1 vs. Secondary	75	Secondary[5]	NC[6]	Wedge	23.9 / NA	**75** / NA	75[7]	8.1	18	- The kinematic failure mode controlling bench geometry in design sector 8 is wedge instabilities. Wedge sliding may occur on the pervasive discontinuity set S_1 in combination with secondary discontinuities. - Kinematic analysis suggests that up to 76° benches are possible, however, based on operational experience this is not recommended. An opportunity to increase the BFA beyond 75° may be possible as more data becomes available.
9	105	Secondary[5]	NC[6]	Secondary[5]	75	Secondary[5]	NC[6]	Wedge	15.6 / NA	**75** / NA	75[7]	8.1	18	- Wedge instabilities returned the largest probability of failure; However, the bench geometry is controlled by the recommended maximum OSA of 52°. The maximum OSA is guided by benchmarking studies and experience at operating mines. - Kinematic analysis suggests that up to 80° benches are possible, however, based on operational experience this is not recommended. An opportunity to increase the BFA beyond 75° may be possible as more data becomes available.

NOTES

1. BFA - Bench Face Angle.
2. Kinematic Analysis POF% - Probability of Failure, the percentage of data within critical zone for kinematic failure (i.e. FOS <1) or Limit - Equilibrium POF% - Probability of Failure, The number of failures (i.e. FOS <1) / total number of potential failures.
3. Bench width determined by using Ryan and Pyor (2000) where: Bench width = 0.2 x bench height + 4.5 m.
4. Bench height as defined by Marathon to be 6 m using triple benches (i.e. 18 m total).
5. Secondary less pervasive discontinuities sets (i.e. not S_1 or JS_1.).
6. NC - Non controlling kinematic failure mode as defined by our bench scale design criteria (POF % <25, Read and Stacey, 2009).
7. Final recommended BFA controlled by kinematic analysis and/or L-E analysis to the maximum recommended angle of 75°.
8. Friction angle determine from average direct shear laboratory results.
9. Final recommended BFA controlled by maximum recommended inter ramp and overall slope angle in combination with geotechnical berms at intervals not exceeding 90 m.

LEGEND

- Low probability of kinematic instability. (i.e.POF < 20%)
- Moderate probability of kinematic instability involving secondary discontinuities (i.e.POF 20 - 25%).
- Moderate / High probability of kinematic instability involving pervasive discontinuity sets (i.e.POF 20 - 25%)

A / B: A - Kinematic Results; B - Limit-Equilibrium Results

VALENTINE GOLD PROJECT PRE-FEASIBILITY GEOTECHNICAL INVESTIGATION - MARATHON DEPOSIT - INTER RAMP DESIGN SUMMARY

Design Sector	Bench Face Dip Direction	Kinematic Failure Modes[1]: Planar – Controlling Major Structure[2]	Planar – Max IRA[3]	Wedge – Controlling Major Structure	Wedge – Max IRA	Toppling – Controlling Major Structure	Toppling – Max IRA	Summary of Kinematic and Limit Equilibrium Analysis: IRA POF%[4]	Maximum IRA	L-E IRA FOS[5]: Domain	Design	Design min	Inter Ramp Design Summary: IRA	Maximum Inter Ramp Height[6] (m)	Maximum Bench Height[8] (m)	Comments
1	136	-	NC[7]	-	NC[7]	-	NC[7]	0.0	NC[7] / 51.5	QEP MFI	5.4 3.1	1.9 **1.0**	**51.5**	90	18	- Major structures are not anticipated to cause kinematic failure in design sector 1. The inter ramp angle is controlled by the maximum recommended overall slope angle (OSA) of 47°. The maximum OSA is guided by benchmarking studies and experience at open pit mines. - L-E analysis was conducted to determine if the FOS of the inter ramp meets the design criteria of FOS ≥1.2 (Read and Stacey, 2009). Both the anticipated average (Design) and worse conditions ($Design_{min}$) have been analyzed using L-E software *SLIDE2* (Rocscience, 2018). - The Designmin conditions within the MFI domain produced a FOS of 1.0. In the event that rock mass conditions are closer to the minimum anticipated values slope depressuring may be required.
2	164	-	NC[7]	-	NC[7]	-	NC[7]	0.0	NC[7] / 51.5	QEP MFI	5.4 3.1	1.9 **1.0**	**51.5**	90	18	- Major structures are not anticipated to cause kinematic failure in design sector 2. The inter ramp angle is controlled by the maximum recommended OSA of 47°. The maximum OSA is guided by benchmarking studies and experience at open pit mines. - L-E analysis was conducted to determine if the FOS of the inter ramp meets the design criteria of FOS ≥1.2 (Read and Stacey, 2009). Both the anticipated average (Design) and worse conditions ($Design_{min}$) have been analyzed using L-E software *SLIDE2* (Rocscience, 2018). - The Designmin conditions within the MFI domain produced a FOS of 1.0. In the event that rock mass conditions are closer to the minimum anticipated values slope depressuring may be required.
3	194	-	NC[7]	-	NC[7]	-	NC[7]	0.0	NC[7] / 51.5	QEP CG	5.4 4.0	1.9 1.3	**51.5**	90	18	- Major structures are not anticipated to cause kinematic failure in design sector 3. The inter ramp angle is controlled by the maximum recommended OSA of 47°. The maximum OSA is guided by benchmarking studies and experience at open pit mines. - L-E analysis was conducted to determine if the FOS of the inter ramp meets the design criteria of FOS ≥1.2 (Read and Stacey, 2009). Both the anticipated average (Design) and worse conditions ($Design_{min}$) have been analyzed using L-E software *SLIDE2* (Rocscience, 2018).
4	241	-	NC[7]	-	NC[7]	-	NC[7]	0.0	NC[7] / 51.5	QEP CG	5.4 4.0	1.9 1.3	**51.5**	90	18	- Major structures are not anticipated to cause kinematic failure in design sector 4. The inter ramp angle is controlled by the maximum recommended OSA of 47°. The maximum OSA is guided by benchmarking studies and experience at open pit mines. - L-E analysis was conducted to determine if the FOS of the inter ramp meets the design criteria of FOS ≥1.2 (Read and Stacey, 2009). Both the anticipated average (Design) and worse conditions ($Design_{min}$) have been analyzed using L-E software *SLIDE2* (Rocscience, 2018).
5	273	-	NC[7]	-	NC[7]	-	NC[7]	0.0	NC[7] / 51.5	QEP CG	5.4 4.0	1.9 1.3	**51.5**	90	18	- Major structures are not anticipated to cause kinematic failure in design sector 5. The inter ramp angle is controlled by the maximum recommended OSA of 47°. The maximum OSA is guided by benchmarking studies and experience at open pit mines. - L-E analysis was conducted to determine if the FOS of the inter ramp meets the design criteria of FOS ≥1.2 (Read and Stacey, 2009). Both the anticipated average (Design) and worse conditions ($Design_{min}$) have been analyzed using L-E software *SLIDE2* (Rocscience, 2018). - The Valentine Lake thrust fault, and fault 4 strike near parallel to design sector 5. However, they do not daylight out of the pit wall (i.e. the faults have an approximate dip of 75°). These faults were modelled as a sliding planes during the L-E analysis.
6	316	-	NC[7]	-	NC[7]	-	NC[7]	0.0	NC[7] / 51.5	QEP CG	5.4 4.0	1.9 1.3	**51.5**	90	18	- Major structures are not anticipated to cause kinematic failure in design sector 6. The inter ramp angle is controlled by the maximum recommended OSA of 47°. The maximum OSA is guided by benchmarking studies and experience at open pit mines. - L-E analysis was conducted to determine if the FOS of the inter ramp meets the design criteria of FOS ≥1.2 (Read and Stacey, 2009). Both the anticipated average (Design) and worse conditions ($Design_{min}$) have been analyzed using L-E software *SLIDE2* (Rocscience, 2018). - The Valentine Lake thrust fault, and fault 4 strike near parallel to design sector 6. However, they do not daylight out of the pit wall (i.e. the faults have an approximate dip of 75°). These faults were modelled as a sliding planes during the L-E analysis.
7	014	-	NC[7]	-	NC[7]	-	NC[7]	0.0	NC[7] / 54.3	QEP CG	4.9 3.7	1.7 **1.1**	**54.3**	90	18	- Major structures are not anticipated to cause kinematic failure in design sector 7. The inter ramp angle is controlled by the maximum recommended OSA of 52°. The maximum OSA is guided by benchmarking studies and experience at open pit mines. - L-E analysis was conducted to determine if the FOS of the inter ramp meets the design criteria of FOS ≥1.2 (Read and Stacey, 2009). Both the anticipated average (Design) and worse conditions ($Design_{min}$) have been analyzed using L-E software *SLIDE2* (Rocscience, 2018). - The Designmin conditions within the CG domain produced a FOS of 1.1. In the event that the rock mas conditions are closer to the minimum anticipated values slope depressurring may be required.
8	071	-	NC[7]	-	NC[7]	-	NC[7]	0.0	NC[7] / 54.3	QEP -	4.9 -	1.7 -	**54.3**	90	18	- Major structures are not anticipated to cause kinematic failure in design sector 8. The inter ramp angle is controlled by the maximum recommended OSA of 52°. The maximum OSA is guided by benchmarking studies and experience at open pit mines. - L-E analysis was conducted to determine if the FOS of the inter ramp meets the design criteria of FOS ≥1.2 (Read and Stacey, 2009). Both the anticipated average (Design) and worse conditions ($Design_{min}$) have been analyzed using L-E software *SLIDE2* (Rocscience, 2018).
9	105	-	NC[7]	-	NC[7]	-	NC[7]	0.0	NC[7] / 54.3	QEP -	4.9 -	1.7 -	**54.3**	90	18	- Major structures are not anticipated to cause kinematic failure in design sector 9. The inter ramp angle is controlled by the maximum recommended OSA of 52°. The maximum OSA is guided by benchmarking studies and experience at open pit mines. - L-E analysis was conducted to determine if the FOS of the inter ramp meets the design criteria of FOS ≥1.2 (Read and Stacey, 2009). Both the anticipated average (Design) and worse conditions ($Design_{min}$) have been analyzed using L-E software *SLIDE2* (Rocscience, 2018).

NOTES

1. Inter ramp design based on potential planar sliding, wedge sliding, and direct toppling failures, flexural toppling was not analyzed at the inter ramp scale.
2. Major Structure - Modelled faults and average values from S_1 and JS_1, pervasive discontinuity sets.
3. IRA - Inter Ramp Angle.
4. Kinematic Analysis POF% - Probability of Failure, the percentage of data within critical zone for kinematic failure (i.e. FOS <1).
5. Limit - Equilibrium FOS - Factor of Safety based on inter ramp geometry using *SLIDE2* (Rocscience, 2018). For Design and $Design_{min}$ see Appendix G. All analysis completed assuming a saturated slope.
6. Maximum recommended inter ramp height without a ramp or geotechnical berm.
7. NC - Non controlling kinematic failure mode as defined by inter ramp design criteria (i.e. POF % <10 and FOS ≥1.2, Read and Stacey, 2009).
8. Maximum bench height based on triple 6 m benches.

LEGEND

CG - Conglomerate Domain
MFI - Mafic Intrusive Domain
QEP - Quartz eye porphyry Domain

Low probability of kinematic instability. (i.e.POF < 5%)

Moderate probability of kinematic instability involving major structures (i.e.POF 5 - 10%).

Moderate / High probability of kinematic instability involving major structures (i.e.POF >10%)

A / B: A - IRA Kinematic Results; B - Calculated IRA from bench geometry

		Kinematic Failure Modes (Friction Angle of 32[8])						Summary of Kinematic and Limit Equilibrium Analysis			Bench Design Summary			
		Planar		Wedge		Topple								
Design Sector	Bench Face Dip Direction	Controlling Joint Sets	Max BFA[1]	Controlling Joint Sets	Max BFA	Controlling Joint Sets	Max BFA	Dominant Failure Mode	Bench POF%[2] (A / B)	Maximum Achievable BFA (A / B)	BFA	Bench Width[3] (m)	Bench Height[4] (m)	Comments
1	188	Secondary[5]	NC[6]	Secondary[5]	NC[6]	S_1	75	Toppling	17.7 / NA	**75** / NA	75	8.1	18	- Toppling instabilities returned the largest probability of failure; the bench geometry is controlled by the recommended maximum bench face angle (75°). - Kinematic analysis suggests that up to 80° benches are possible, however, based on operational experience at mine sites this is not recommended. An opportunity to increase the BFA beyond 75° may be possible as more data becomes available.
2	174	Secondary[5]	NC[6]	Secondary[5]	NC[6]	S_1	72	Toppling	24.9 / 10.3[7]	72 / **75**	75	8.1	18	- The kinematic failure mode controlling bench geometry in design sector 2 is toppling instabilities. Analyses indicate toppling may occur on the pervasive discontinuity set S_1. - Where kinematic analysis returned a POF > 25% for BFA <75°, Limit-Equilibrium (L-E) analyses were conducted in RocTopple (Rocscience 2019) to further evaluate the affect of toppling instability on the bench geometry.
3	234	Secondary[5]	NC[6]	Secondary[5]	75	S1 vs. JS1	NC[6]	Toppling	17.9 / NA	**75** / NA	75	8.1	18	- Toppling instabilities returned the largest probability of failure; the bench geometry is controlled by the recommended maximum BFA of 75°. - Kinematic analysis suggests that up to 80° benches are possible, however, based on operational experience at mine sites this is not recommended. An opportunity to increase the BFA beyond 75° may be possible as more data becomes available.
4	269	Secondary[5]	NC[6]	Secondary[5]	75	Secondary[5]	NC[6]	Wedge	19.2 / NA	**75** / NA	75	8.1	18	- Wedge instabilities returned the largest probability of failure; the bench geometry is controlled by the recommended maximum BFA of 75°. - Kinematic analysis suggests that up to 80° benches are possible, however, based on operational experience at mine sites this is not recommended. An opportunity to increase the BFA beyond 75° may be possible as more data becomes available.
5	319	S_1	75	S_1 vs. JS_1	60	Secondary[5]	NC[6]	Wedge	23.8 / 19.1	55 / **65**	65	8.1	18	- The kinematic failure mode controlling bench geometry in design sector 5 is wedge instabilities. Analyses indicate wedge sliding may occur on the pervasive discontinuity sets (S_1 & JS_1). Additionally, planar sliding along the S_1 discontinuity set is possible and may control the final bench geometry. - Where kinematic analysis returned a POF > 25% for BFA <75°, L-E analyses were conducted in SWedge (Rocscience 2019) to further evaluate the affect of wedge instability on the bench geometry.
6	357	S_1	66	S_1 vs. JS_1	55	Secondary[5]	NC[6]	Wedge	24.5 / 24.7	55 / **65**	65	8.1	18	- The kinematic failure mode controlling bench geometry in design sector 6 is wedge instabilities. Analyses indicate wedge sliding may occur on the pervasive discontinuity sets (S_1 & JS_1). Additionally, planar sliding along the S_1 discontinuity set is possible and may control the final bench geometry. - Where kinematic analysis returned a POF > 25% for BFA < 75°, L-E analyses were conducted in SWedge (Rocscience 2019) to further evaluate the affect of wedge instability on the bench geometry.
7	057	Secondary[5]	NC[6]	Secondary[5]	75	Secondary[5]	NC[6]	Wedge	24.0 / NA	**75** / NA	75	8.1	18	- Wedge instabilities returned the largest probability of failure; the bench geometry is controlled by the recommended maximum BFA of 75°. - Kinematic analysis suggests that up to 77° benches are possible, however, based on operational experience at mine sites this is not recommended. An opportunity to increase the BFA beyond 75° may be possible as more data becomes available.
8	113	Secondary[5]	NC[6]	Secondary[5]	NC[6]	Secondary[5]	75	Toppling	11.5 / NA	**75** / NA	75	8.1	18	- Toppling instabilities returned the largest probability of failure; the bench geometry is controlled by the recommended maximum BFA of 75°. - Kinematic analysis suggests that up to 80° benches are possible, however, based on operational experience at mine sites this is not recommended. An opportunity to increase the BFA beyond 75° may be possible as more data becomes available.
9	155	Secondary[5]	NC[6]	Secondary[5]	NC[6]	S_1	67	Toppling	24.6 / 10.3[7]	67 / **75**	75	8.1	18	- The kinematic failure mode controlling bench geometry in design sector 9 is toppling. Analyses indicate that toppling may occur on the pervasive discontinuity set S_1. - Where kinematic analysis returned a POF > 25% for BFA < 75º, L-E analyses were conducted in RocTopple (Rocscience 2019) to further evaluate the affect of toppling instabilities on the bench geometry.

NOTES

1. BFA - Bench Face Angle.
2. Kinematic Analysis POF% - Probability of Failure, the percentage of data within critical zone for kinematic failure (i.e. FOS <1) or Limit - Equilibrium POF% - Probability of Failure, The number of failures (i.e. FOS <1) / total number of potential failures.
3. Bench width determined by using Ryan and Pyor (2000) where: Bench width = 0.2 x bench height + 4.5 m.
4. Bench height as defined by Marathon to be 6 m using triple benches (i.e. 18 m total).
5. Secondary less pervasive discontinuities sets (i.e. not S_1 or JS_1.).
6. NC - Non controlling kinematic failure mode as defined by our bench scale design criteria (POF % <25, Read and Stacey, 2009).
7. Analysis assumes a cohesion of 50 KPa (foliated rock with clay coatings, Read and Stacey,2009).
8. Friction angle determine from average direct shear laboratory results.

LEGEND

- Low probability of kinematic instability. (i.e.POF < 20%)
- Moderate probability of kinematic instability involving secondary discontinuities (i.e.POF 20 - 25%).
- Moderate / High probability of kinematic instability involving pervasive discontinuity sets (i.e.POF 20 - 25%)

A / B: A - Kinematic Results; B - Limit-Equilibrium Results

VALENTINE GOLD PROJECT PRE-FEASIBILITY GEOTECHNICAL INVESTIGATION - LEPRECHAUN DEPOSIT - INTER RAMP DESIGN SUMMARY

Design Sector	Bench Face Dip Direction	Kinematic Failure Modes[1.]: Planar – Controlling Major Structure[2.]	Planar – Max IRA[3.]	Wedge – Controlling Major Structure	Wedge – Max IRA	Toppling – Controlling Major Structure	Toppling – Max IRA	Summary of Kinematic and Limit Equilibrium Analysis: IRA POF%[4.]	Maximum IRA	IRA FOS[5.] Design	IRA FOS[5.] Design min	Inter Ramp Design Summary: IRA	Maximum Inter Ramp Height[6.] (m)	Maximum Bench Height[8.] (m)	Comments
1	188	-	NC[7.]	-	NC[7.]	-	NC[7.]	0.0	NC[7.] / **54.3**	4.2	1.2	54.3	126	18	- Major structures are not anticipated to cause kinematic failure in design sector 1. The inter ramp angle is controlled by the bench design. - Limit Equilibrium (L-E) Analysis was conducted to determine if the FOS of the inter ramp meets the design criteria of FOS ≥ 1.2 (Read and Stacey, 2009). Both the anticipated average (Design) and worse conditions (Designmin) have been analyzed using L-E software *SLIDE2* (Rocscience, 2018).
2	174	-	NC[7.]	-	NC[7.]	-	NC[7.]	0.0	NC[7.] / **54.3**	4.2	1.2	54.3	126	18	- Major structures are not anticipated to cause kinematic failure in design sector 2. The inter ramp angle is controlled by the bench design. - Limit Equilibrium (L-E) Analysis was conducted to determine if the FOS of the inter ramp meets the design criteria of FOS ≥ 1.2 (Read and Stacey, 2009). Both the anticipated average (Design) and worse conditions (Designmin) have been analyzed using L-E software *SLIDE2* (Rocscience, 2018).
3	234	-	NC[7.]	-	NC[7.]	-	NC[7.]	0.0	NC[7.] / **54.3**	4.2	1.2	54.3	126	18	- Major structures are not anticipated to cause kinematic failure in design sector 3. The inter ramp angle is controlled by the bench design. - Limit Equilibrium (L-E) Analysis was conducted to determine if the FOS of the inter ramp meets the design criteria of FOS ≥ 1.2 (Read and Stacey, 2009). Both the anticipated average (Design) and worse conditions (Designmin) have been analyzed using L-E software *SLIDE2* (Rocscience, 2018).
4	269	-	NC[7.]	-	NC[7.]	-	NC[7.]	0.0	NC[7.] / **54.3**	2.5	1.2	54.3	126	18	- Major structures are not anticipated to cause kinematic failure in design sector 4. The inter ramp angle is controlled by the bench design. - Limit Equilibrium (L-E) Analysis was conducted to determine if the FOS of the inter ramp meets the design criteria of FOS ≥ 1.2 (Read and Stacey, 2009). Both the anticipated average (Design) and worse conditions (Designmin) have been analyzed using L-E software *SLIDE2* (Rocscience, 2018).
5	319	-	NC[7.]	-	NC[7.]	-	NC[7.]	0.0	NC[7.] / **47.4**	2.5	1.2	47.4	126	18	- Major structures are not anticipated to cause kinematic failure in design sector 5. The inter ramp angle is controlled by the bench design. - The Valentine Lake thrust fault, fault 9, and fault 10 strike near parallel to design sector 5. However, they do not daylight out of the pit wall (i.e. the faults have an approximate dip of 70°). These faults were modelled as a sliding planes during the L-E analysis to determine their affect on the inter ramp angle FOS. It was determined, from L-E analysis, the FOS meets or exceeds the design criteria (FOS > 1.2) for the recommended inter ramp angle.
6	357	-	NC[7.]	-	NC[7.]	-	NC[7.]	0.0	NC[7.] / **47.4**	2.5	1.2	47.4	126	18	- Major structures are not anticipated to cause kinematic failure in design sector 6. The inter ramp angle is controlled by the bench design. - The Valentine Lake thrust fault, fault 9, and fault 10 strike near parallel to design sector 6. However, they do not daylight out of the pit wall (i.e. the faults have an approximate dip of 70°). These faults were modelled as a sliding planes during the L-E analysis to determine their affect on the inter ramp angle FOS. It was determined, from L-E analysis, the FOS meets or exceeds the design criteria (FOS > 1.2) for the recommended inter ramp angle.
7	057	-	NC[7.]	-	NC[7.]	-	NC[7.]	0.0	NC[7.] / **54.3**	4.2	1.2	54.3	126	18	- Major structures are not anticipated to cause kinematic failure in design sector 7. The inter ramp angle is controlled by the bench design. - Limit Equilibrium (L-E) Analysis was conducted to determine if the FOS of the inter ramp meets the design criteria of FOS ≥ 1.2 (Read and Stacey, 2009). Both the anticipated average (Design) and worse conditions (Designmin) have been analyzed using L-E software *SLIDE2* (Rocscience, 2018).
8	113	-	NC[7.]	-	NC[7.]	-	NC[7.]	0.0	NC[7.] / **54.3**	4.2	1.2	54.3	126	18	- Major structures are not anticipated to cause kinematic failure in design sector 8. The inter ramp angle is controlled by the bench design. - Limit Equilibrium (L-E) Analysis was conducted to determine if the FOS of the inter ramp meets the design criteria of FOS ≥ 1.2 (Read and Stacey, 2009). Both the anticipated average (Design) and worse conditions (Designmin) have been analyzed using L-E software *SLIDE2* (Rocscience, 2018).
9	155	-	NC[7.]	-	NC[7.]	-	NC[7.]	0.0	NC[7.] / **54.3**	4.2	1.2	54.3	126	18	- Major structures are not anticipated to cause kinematic failure in design sector 9. The inter ramp angle is controlled by the bench design. - Limit Equilibrium (L-E) Analysis was conducted to determine if the FOS of the inter ramp meets the design criteria of FOS ≥ 1.2 (Read and Stacey, 2009). Both the anticipated average (Design) and worse conditions (Designmin) have been analyzed using L-E software *SLIDE2* (Rocscience, 2018).

NOTES

1. Inter ramp design based on potential planar sliding, wedge sliding, and direct toppling failures, flexural toppling was not analyzed at the inter ramp scale.
2. Major Structure - Modelled faults and average values from S_1 and JS_1, pervasive discontinuity sets.
3. IRA - Inter Ramp Angle.
4. Kinematic Analysis POF% - Probability of Failure, the percentage of data within critical zone for kinematic failure (i.e. FOS <1).
5. Limit - Equilibrium FOS - Factor of Safety based on inter ramp geometry.
6. Maximum recommended inter ramp height without a ramp or geotechnical berm.
7. NC - Non controlling kinematic failure mode as defined by inter ramp design criteria (i.e. POF % <10 and FOS ≥1.2, Read and Stacey, 2009).
8. Maximum bench height based on triple 6 m benches.

LEGEND

- Low probability of kinematic instability. (i.e.POF < 5%)
- Moderate probability of kinematic instability involving major structures (i.e.POF 5 - 10%).
- Moderate / High probability of kinematic instability involving major structures (i.e.POF >10%)

A / B
- A - IRA Kinematic Results
- B - Calculated IRA from bench geometry

VALENTINE GOLD PROJECT PRE-FEASIBILITY GEOTECHNICAL INVESTIGATION

PROJECT: Valentine Gold Project - Pre-Feasibility Geotechnical Investigation
DATE: 06-Dec-19
PROJECT No: 19-0015-H
SUBJECT: Marathon Deposit - Inter Ramp and Overall Slope Stability Modelling
Prepared by: Andrew Guest, P. Eng.
Checked by: Tony L. Gilman, M. Sc., P. Geo, P. Eng.

LEGEND
- QEP - Quartz Eye Porphyry Domain
- QEP - Quartz Eye Porphyry Domain, Damage Factor "D" = 0.85
- MFI - Mafic Intrusive Domain
- MFI - Mafic Intrusive Domain, Damage Factor "D" = 0.85
- CG - Conglomerate Domain
- CG - Conglomerate Domain, Damage Factor "D" = 0.85

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www.terranegeoscience.com

Summary
Limit-Equilibrium (L-E) analyses were conducted for typical slope geometries for the NW,SE, and SW Design Areas (Table G1). Design sectors were grouped for L-E analysis based on geotechnical domains and their recommended bench geometry. The analysis was conducted using the L-E software *SLIDE2*, Rocscience (2018). The design criteria for the inter ramp and overall slope stability is a minimum FOS of 1.2 and 1.3 respectively (Read and Stacey, 2009).
Rock strength was classified using generalized Hoek-Brown criteria (Table G2). Generalized Hoek-Brown m_i values were determined from analyzing field and laboratory testing results in *Rocdata* (Rocscience, 2018). GSI was determined through the relationship RMR_{76} = GSI for RMR_{76} >18 (Hoek et al. 1995). All analyses were conducted using both the mean design values and the minimum design values for the intact rock strength (UCS) and RMR_{76} values (Tables G3 & G4).
Figures G1 to G5 show the critical slip surface for the inter ramp for the mean design values. The critical slip surfaces were determined using a non circular optimized slip surface with a potential tension crack forming at surface (10° tolerance). Based on the geotechnical model, and the typical slope geometry, the inter ramp slope FOS_{mean} meets or exceeds the criteria for design.
The IRA FOS_{min} for both the NW Area MFI domain, and the SW Area CG domain produced a FOS <1.2. If rock mass conditions are closer to the $Design_{min}$ values, slope depressurization may be required.
Figures G6 to G8 show the critical slip surface for the overall slope for the mean design values. The critical slip surfaces were determined using a non circular optimized slip surface with a potential tension crack forming at surface (10° tolerance). Based on the geotechnical model, and the typical slope geometry, the overall slope FOS meet the criteria for design.

Table G1 - Design Sector Summary - Open Pit Wall Angle

Design Sector	Geotechnical Domain (Primary/Secondary)	BFA[1.]	IRA[1.]	OSA[1.]	L-E[2.] Analysis
1	QEP/MFI	71	51.5	47	NW Area
2	QEP/MFI	71	51.5	47	
3	QEP/CG	71	51.5	47	
4	CG/QEP	71	51.5	47	SE Area
5	CG/QEP	71	51.5	47	
6	CG/QEP	71	51.5	47	
7	QEP/CG	75	54.3	52	SW Area
8	QEP	75	54.3	52	
9	QEP	75	54.3	52	

Table G2 - Generalized Hoek Brown Criteria Summary

Domain	m_i	UCS_{mean} (MPa)	UCS_{min} (MPa)	$RMR_{76\ mean}$	$RMR_{76\ min}$	D Factor[3.]
QEP	11.2	145	89	68	59	0 (0.85)
CG	11.1	113	54	67	59	0 (0.85)
MFI	25[13.]	120	62	66	55	0 (0.85)

Table G3 - Limit Equilibrium Results Summary - Inter Ramp

Slope Angle	Area[2.]	Domain	FOS_{mean}	FOS_{min}[4.]
51.5	NW/SE	MFI	3.1	1.0
		QEP	5.4	1.9
		CG	4.0	1.3
54.3	SW	QEP	4.9	1.7
		CG	3.7	1.1

Table G4 - Limit Equilibrium Results Summary - Overall Slope

Slope Angle	Area[2.]	FOS_{mean}	FOS_{min}[4.]
46.9	NW	2.5	1.5
46.9	SE	2.4	1.3
51.1	SW	3.3	1.7

Figure G1 - Inter Ramp - L-E Analysis
NW Area - MFI Domain - Typical Section[5.6.7.8.9.10.11.]

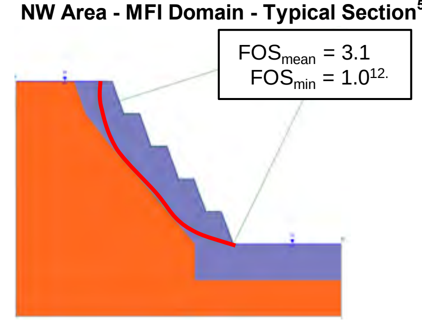


Figure G2 - Inter Ramp - L-E Analysis
NW/SE Area - QEP Domain - Typical Section[5.6.7.8.9.10.11.]

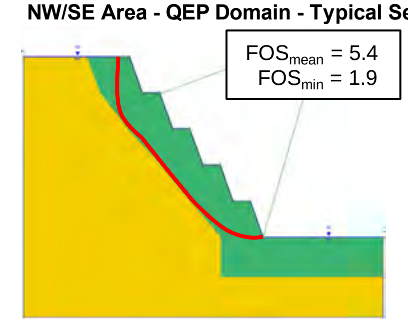


Figure G3 - Inter Ramp - L-E Analysis
SE Area - CG Domain - Typical Section[5.6.7.8.9.10.11.]

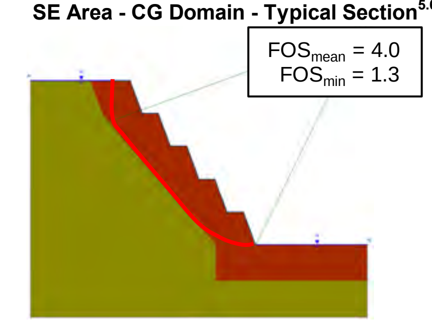


Figure G4 - Inter Ramp - L-E Analysis
SW Area - QEP Domain - Typical Section[5.6.7.8.9.10.11.]

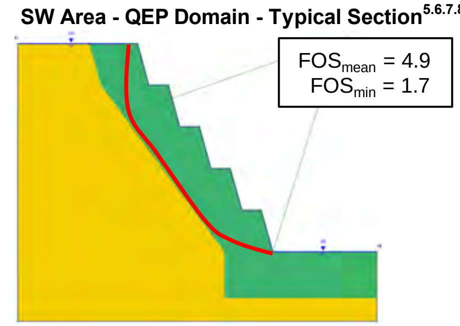


Figure G5 - Inter Ramp - L-E Analysis
SW Area - CG Domain - Typical Section[5.6.7.8.9.10.11.]

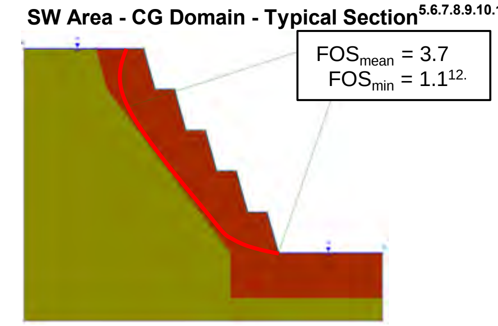


Figure G6 - Overall Slope - L-E Analysis
NW Area - Typical Section[5.6.7.8.9.10.11.]

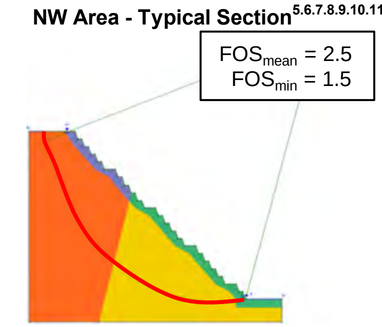


Figure G7 - Overall Slope - L-E Analysis
SE Area - Typical Section[5.6.7.8.9.10.11.]

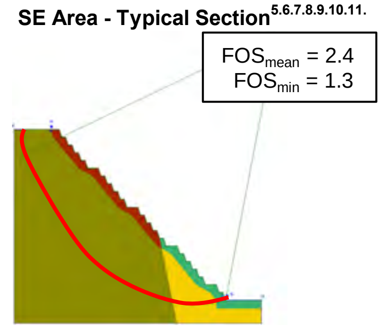
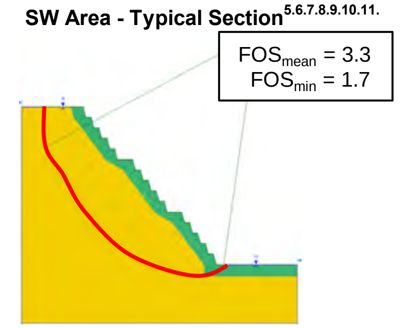


Figure G8 - Overall Slope - L-E Analysis
SW Area - Typical Section[5.6.7.8.9.10.11.]



NOTES
1. BFA - Bench face angle, IRA - Inter Ramp Angle, OSA - Overall Slope Angle.
2. Design Areas for Limit - Equilibrium Analysis.
3. (#) - Damage Factor of 0.85 used within 20 m of slope face.
4. FOS_{min} = Factor of Safety of slope if using minimum values for UCS and RMR_{76} within each domain.
5. Models were constructed using *Slide2* (Rocscience,2018).
6. Slope assumed to be fully saturated for conservative results.
7. Design Criteria - Inter Ramp FOS ≥ 1.2, Overall Slope ≥1.3 (Read and Stacey, 2009).
8. Critical slip surface for each model is displayed and it's FOS is reported.
9. Inter ramp heights modelled were limited to 90 m for L-E analyses.
   Overall slope heights were modelled to 396 m (NW and SE Areas) and 270 m (SW Area).
10. Modelling included assessment of modelled Terrane, 2019 faults.
11. All analysis completed using Janbu method of slices.
12. Does not meet design criteria: In the event rock mass conditions are closer to the $design_{min}$ values, slope depressurization may be necessary.
13. mi value based on *Rocdata* (Rocscience,2019) typical values from mafic intrusive rock types.

PROJECT: Valentine Gold Project - Pre-Feasibility Geotechnical Investigation
DATE: 02-Oct-19
PROJECT No: 19-0015-H
SUBJECT: Leprechaun Deposit - Inter Ramp and Overall Slope Stability Modelling
Prepared by: Andrew Guest, P. Eng.
Checked by: Tony L. Gilman, M. Sc., P. Geo, P. Eng.

LEGEND
- TRJ - Trondhjemite Domain
- TRJ - Trondhjemite Domain, Damage Factor "D" = 0.85
- CG - Conglomerate Domain
- CG - Conglomerate Domain, Damage Factor "D" = 0.85

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Halifax, NS
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Table G1 - Design Sector Summary - Open Pit Wall Angle

Design Sector	Geotechnical Domain (Primary/Secondary)	BFA[1]	IRA[1]	OSA[1]	L-E[2] Analysis
1	TRJ	75	54.3	50.6	NW Area
2	TRJ	75	54.3	50.6	
3	TRJ	75	54.3	50.6	
4	TRJ/CG	75	54.3	50.6	
5	CG/TRJ	65	47.4	44.4	SE Area
6	CG/TRJ	65	47.4	44.4	
7	TRJ/CG	75	54.3	50.6	NW Area
8	TRJ	75	54.3	50.6	
9	TRJ	75	54.3	50.6	

Table G2 - Generalized Hoek Brown Criteria Summary

Domain	m_i	UCS_{mean} (MPa)	UCS_{min} (MPa)	$RMR_{76\ mean}$	$RMR_{76\ min}$	D Factor[3]
TRJ	14.7	112	72	71	60	0 (0.85)
CG	11.1	126	54	71	60	0 (0.85)

Table G3 - Limit Equilibrium Results Summary - Inter Ramp and Overall Slope

Slope	Slope Angle	Area[2]	FOS_{mean}	$FOS^{4.}_{min}$
Inter Ramp	54.3	NW	4.2	1.2
	47.4	SE	2.5	1.2
Overall Slope	50.6	NW	2.7	1.3
	44.4	SE	2.1	1.3

NOTES
1. BFA - Bench face angle, IRA - Inter Ramp Angle, OSA - Overall Slope Angle.
2. Design Areas for Limit - Equilibrium Analysis.
3. (#) - Damage Factor of 0.85 used within 20 m of slope face.
4. FOSmin = Factor of Safety of slope if using minimum design values for UCS and RMR_{76}.
5. Models were constructed using *Slide 2018* (Rocscience).
6. Slope assumed to be fully saturated for conservative results.
7. Design Criteria - Inter Ramp FOS $\geq$ 1.2, Overall Slope $\geq$1.3 (Read and Stacey, 2009).
8. Critical slip surface for each model is displayed and it's FOS is reported.
9. Inter ramp heights modelled were limited to 126 m for L-E analyses. Overall slope heights were modelled to 306 m with two 25 m wide ramps or geotechnical berms.
10. Modelling includes the minimum FOS for modelled faults.
11. All analysis completed using Janbu method of slices.

Summary

Limit-Equilibrium (L-E) analyses were conducted for typical slope geometries for the NW Area and the SE Area (Table G1). Design sectors were grouped for L-E analysis based on geotechnical domains and their recommended bench geometry. The analysis was conducted using the L-E software *Slide2*, Rocscience (2018). The design criteria for the inter ramp and overall slope stability is a minimum FOS of 1.2 and 1.3 respectively (Read and Stacey, 2009).

Rock strength was classified using generalized Hoek-Brown criteria (Table G2). Generalized Hoek-Brown m_i values were determined from analyzing field and laboratory testing results in *Rocdata* (Rocscience, 2018). GSI was determined through the relationship RMR_{76} = GSI for RMR_{76} >18 (Hoek et al. 1995). All analyses were conducted using both the mean design values and the minimum design values for the intact rock strength (UCS) and RMR_{76} values (Table G3).

Figures G1 and G2 show the critical slip surface for the inter ramp and overall slope for the mean design values in the NW Area. The critical slip surfaces were determined using a non circular optimized slip surface with a potential tension crack forming at surface. Based on the geotechnical model, and the typical slope geometry, the NW Area inter ramp and overall slope FOS meet or exceed the criteria for design.

Figures G3 and G4 show the critical slip surface for the inter ramp and overall slope for the mean design values in the SE Area. The critical slip surfaces were determined using a non circular optimized slip surface with a potential tension crack forming at surface (10° tolerance.). Based on the geotechnical model, and the typical slope geometry, the SE Area inter ramp and overall slope FOS meet or exceed the criteria for design.

Figure G1 - Inter Ramp - L-E Analysis
NW Area - Typical Section[5.6.7.8.9.10.11.]

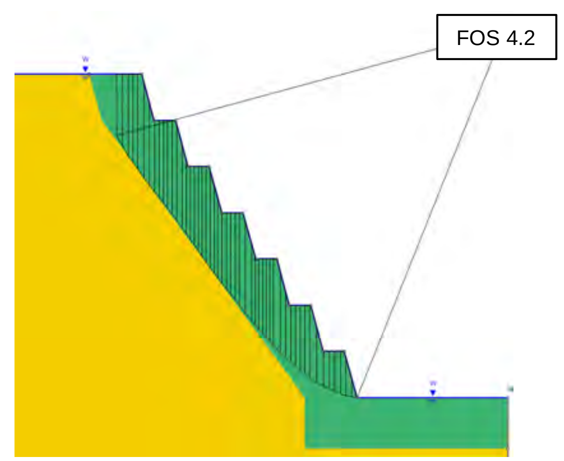


Figure G2 - Overall Slope - L-E Analysis
NW Area - Typical Section[5.6.7.8.9.10.11.]

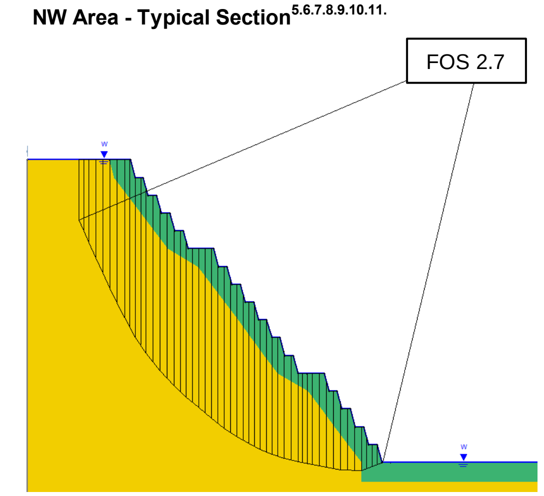


Figure G3 - Inter Ramp - L-E Analysis
SE Area - Typical Section[5.6.7.8.9.10.11.]

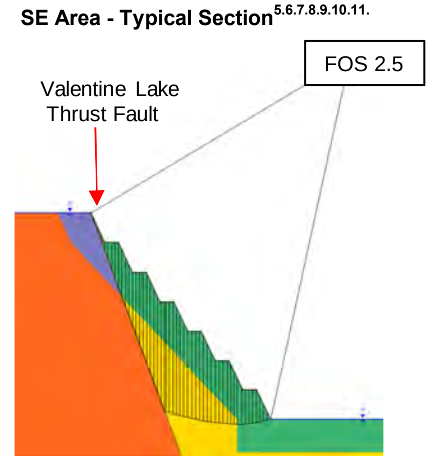
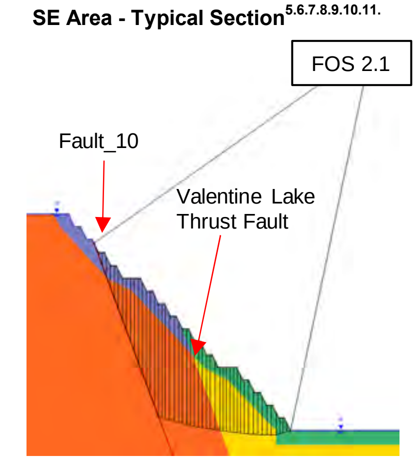
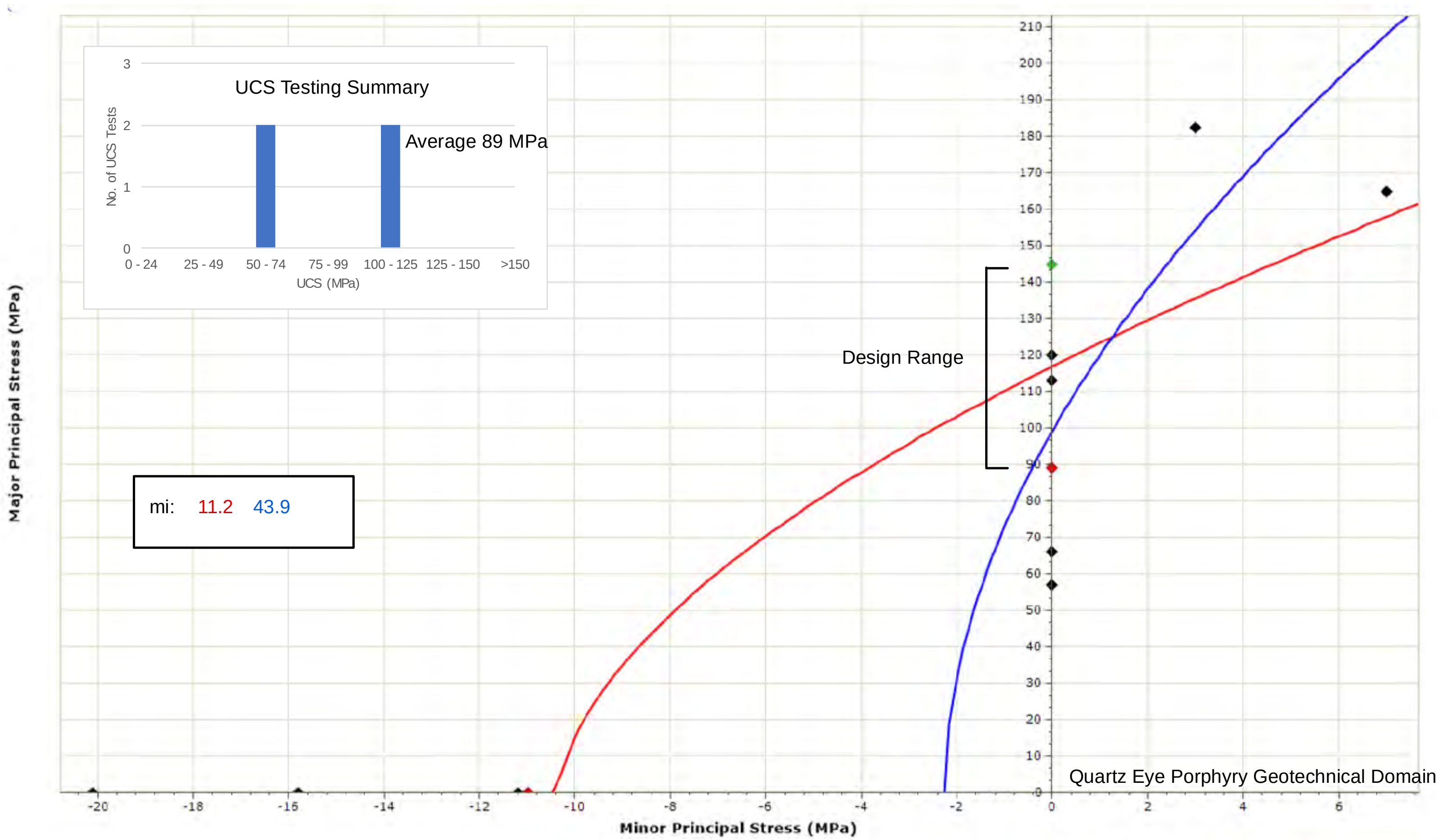


Figure G4 - Overall Slope - L-E Analysis
SE Area - Typical Section[5.6.7.8.9.10.11.]





NOTES:

1. Analysis conducted using Rocdata, Rocscience 2018.
2. Generalized Hoek Brown Criteria material constant mi determined using average laboratory results.
3. Average results from the brazilian tensile testing have been reduced to 70% of the original value (Bewick et. al., 2011)

LEGEND

- Average Laboratory Results
- All Laboratory Results
- Average Rebound Hammer Results
- Including Brazilian Tensile Testing
- Excluding Brazilian Tensile Testing (i.e Triaxial and UCS only)

MARATHON GOLD CORP.		
VALENTINE GOLD PROJECT PRE-FEASIBILITY GEOTECHNICAL INVESTIGATION		
Appendix G - Generalized Hoek-Brown mi Quartz Eye Porphyry Geotechnical Domain		
Date: 12-05-19	Drawn By: AG	TERRANE GEOSCIENCE INC.
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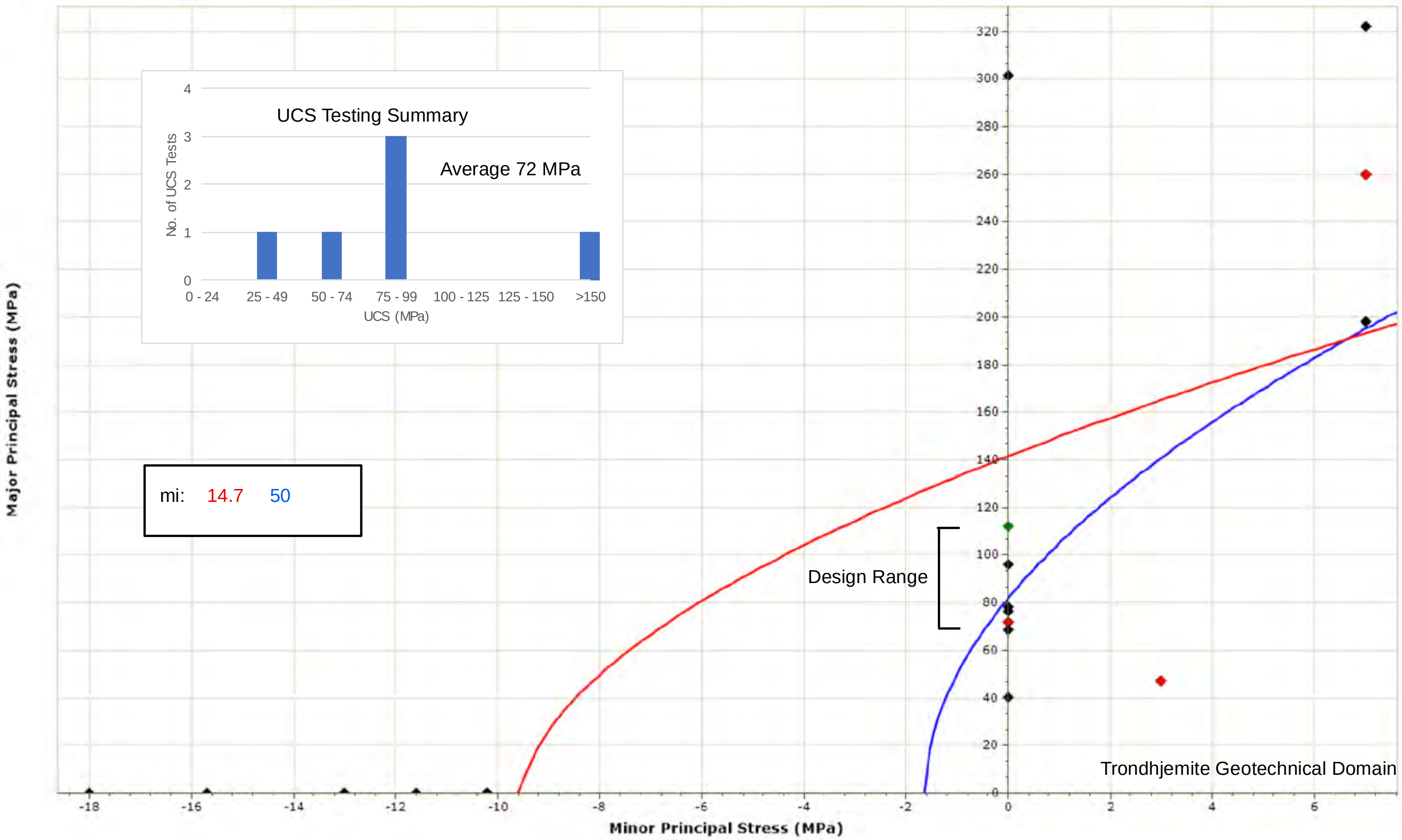

NOTES:

1. Analysis conducted using Rocdata, Rocscience 2018.
2. Generalized Hoek Brown Criteria material constant mi determined using average laboratory results.
3. Average results from the brazilian tensile testing have been reduced to 70% of the original value (Bewick et. al., 2011)

LEGEND

- Average Laboratory Results
- All Laboratory Results
- Average Rebound Hammer Results
- Including Brazilian Tensile Testing
- Excluding Brazilian Tensile Testing (i.e Triaxial and UCS only)

MARATHON GOLD CORP.

VALENTINE GOLD PROJECT PRE-FEASIBILITY GEOTECHNICAL INVESTIGATION

Appendix G - Generalized Hoek-Brown mi Trondhjemite Geotechnical Domain

Date: 10-15-19	Drawn By: AG
REV 0.0	Checked By: TLG
NO. 19-0015-H	Approved By: TLG

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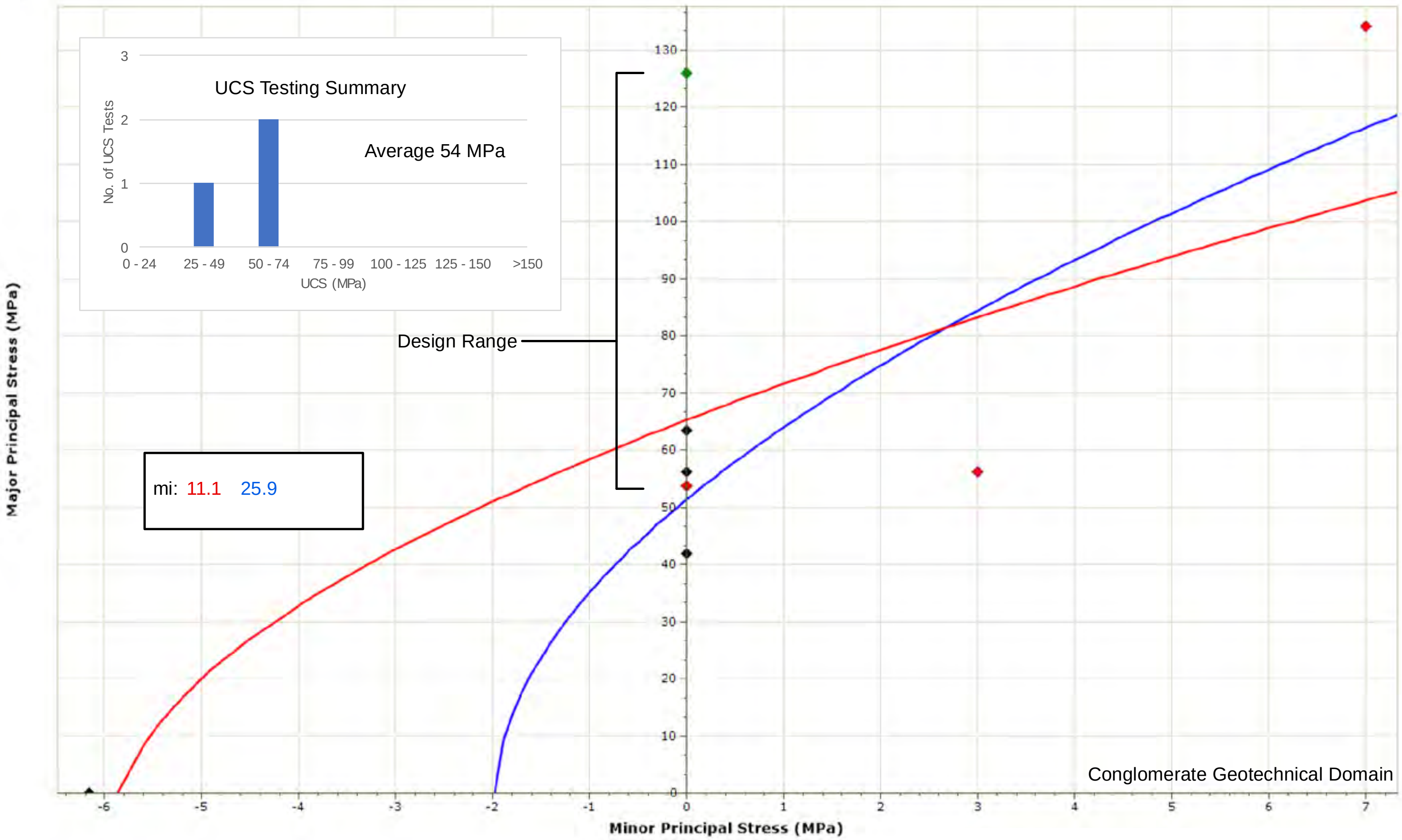

NOTES:

1. Analysis conducted using Rocdata, Rocscience 2018.
2. Generalized Hoek Brown Criteria material constant mi determined using average laboratory results.
3. Average results from the brazilian tensile testing have been reduced to 70% of the original value (Bewick et. al., 2011)

LEGEND

- Average Laboratory Results
- All Laboratory Results
- Average Rebound Hammer Results
- Including Brazilian Tensile Testing
- Excluding Brazilian Tensile Testing (i.e. Triaxial and UCS only)

MARATHON GOLD CORP.	
VALENTINE GOLD PROJECT PRE-FEASIBILITY GEOTECHNICAL INVESTIGATION	
Appendix G - Generalized Hoek-Brown mi Conglomerate Geotechnical Domain	
Date: 10-15-19	Drawn By: AG
REV 0.0	Checked By: TLG
NO. 19-0015-H	Approved By: TLG

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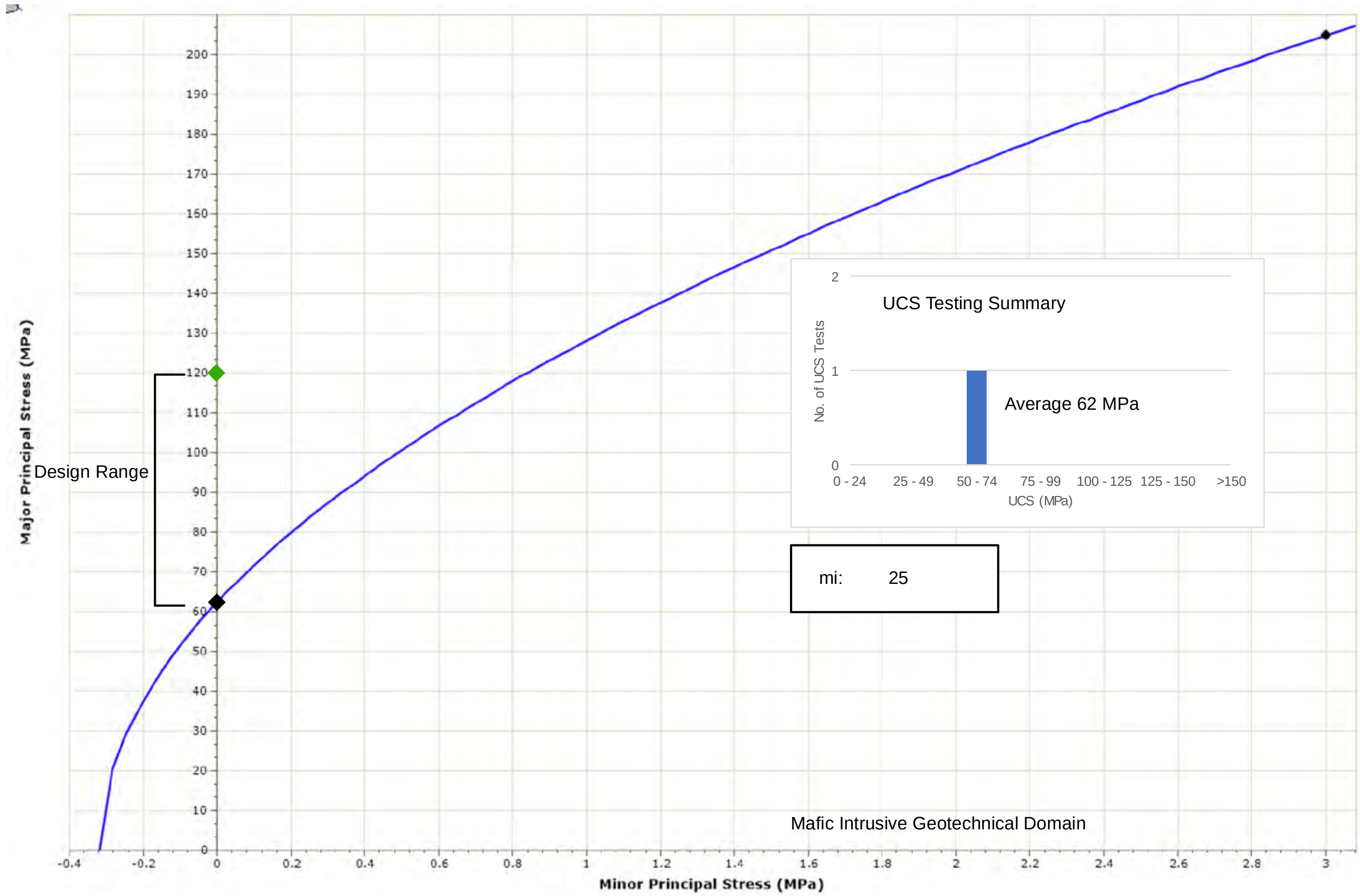

NOTES:

1. Analysis conducted using Rocdata, Rocscience 2018.
2. Generalized Hoek Brown Criteria material constant mi determined using Rocdata library values. The mi value reported falls within the typical range for similar rock types (i.e. Basalt/Gabbro).

LEGEND

- Average Laboratory Results
- All Laboratory Results
- Average Rebound Hammer Results
- Including Brazilian Tensile Testing
- Excluding Brazilian Tensile Testing (i.e Triaxial and UCS only)

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VALENTINE GOLD PROJECT PRE-FEASIBILITY GEOTECHNICAL INVESTIGATION

Appendix G - Generalized Hoek-Brown mi
Mafic Intrusive Geotechnical Domain

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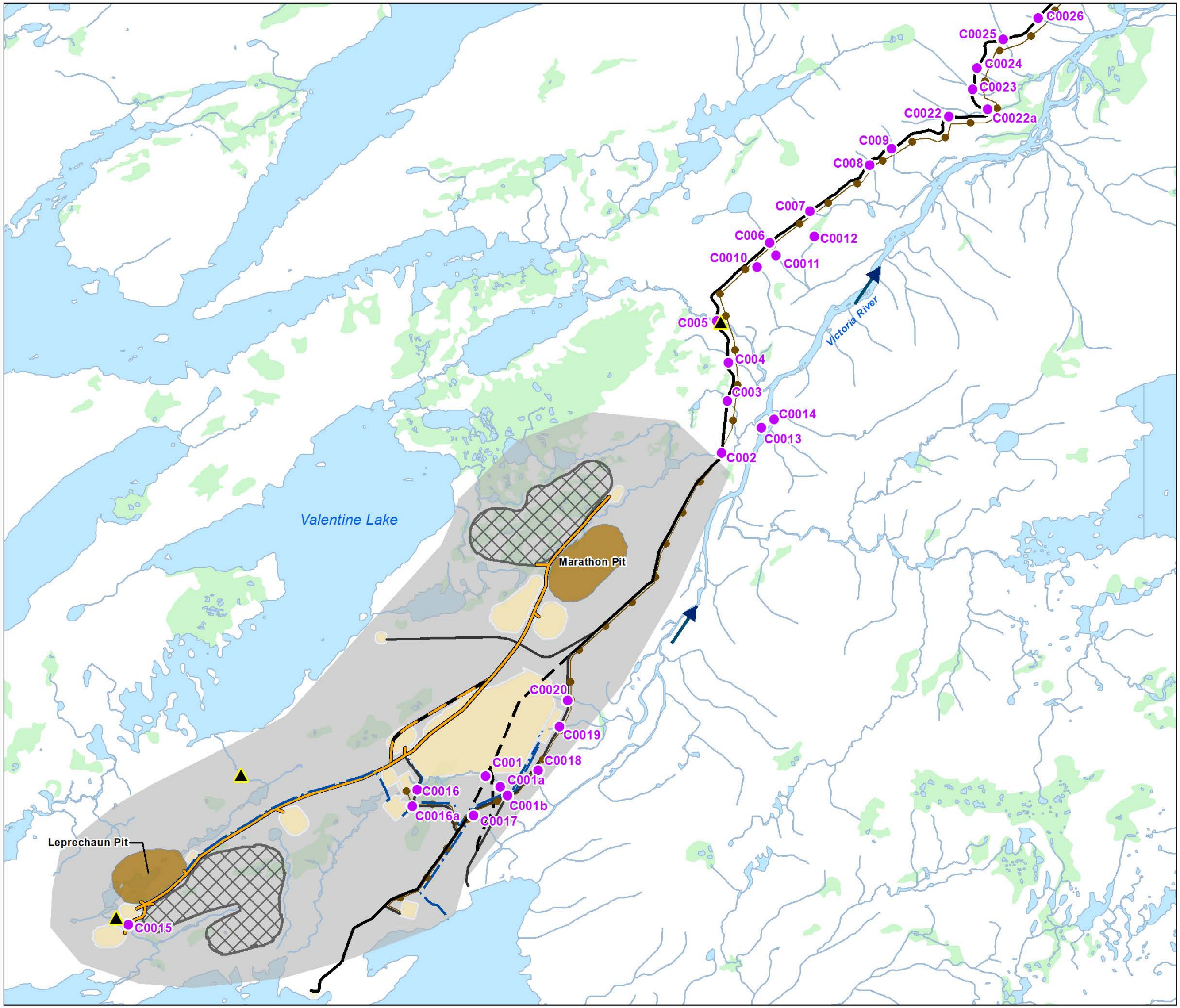
APPENDIX 2D

Stream Crossings Along Access Road



Stantec

Existing Conditions for Fish and Fish Habitat

- Fish Passage Barrier
- Stream Crossing
- Flow Direction

Mine Features

- Initial Haul Road (years 1 to 5)
- Future Haul Road (years 5+)
- Existing Access Road
- Existing Access Road (to be removed)
- Site Road
- Pipeline
- Power Line
- Open Pit
- Waste Rock Pile
- Other Mine Infastructure
- Mine Site

Other Feature

- Watercourse
- Wetland
- Waterbody

N

0 0.5 1
Kilometers

Project Location
Valentine Lake
Central Newfoundland, CA

Client/Project
121416408_041b
Marathon Gold Corporation
Valentine Gold Project
Environmental Impact Statement

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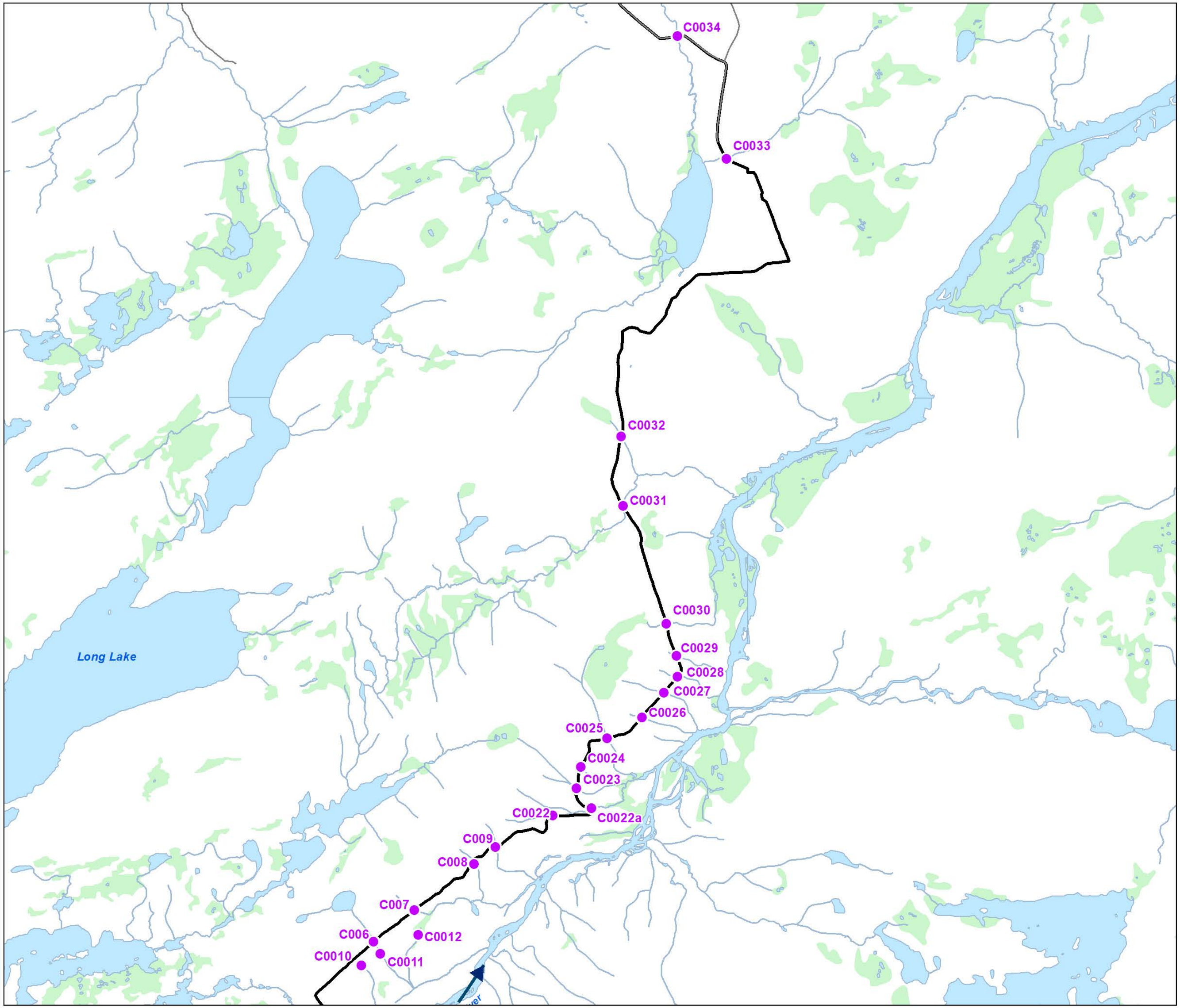

Stantec

Stream Crossing

Flow Direction

Mine Features

Existing Access Road

Other Features

Resource / Recreation Road

Watercourse

Wetland

Waterbody

N

0 0.5 1

Kilometers

Project Location
Valentine Lake
Central Newfoundland, CA

Client/Project 121416408_041c
Marathon Gold Corporation
Valentine Gold Project
Environmental Impact Statement

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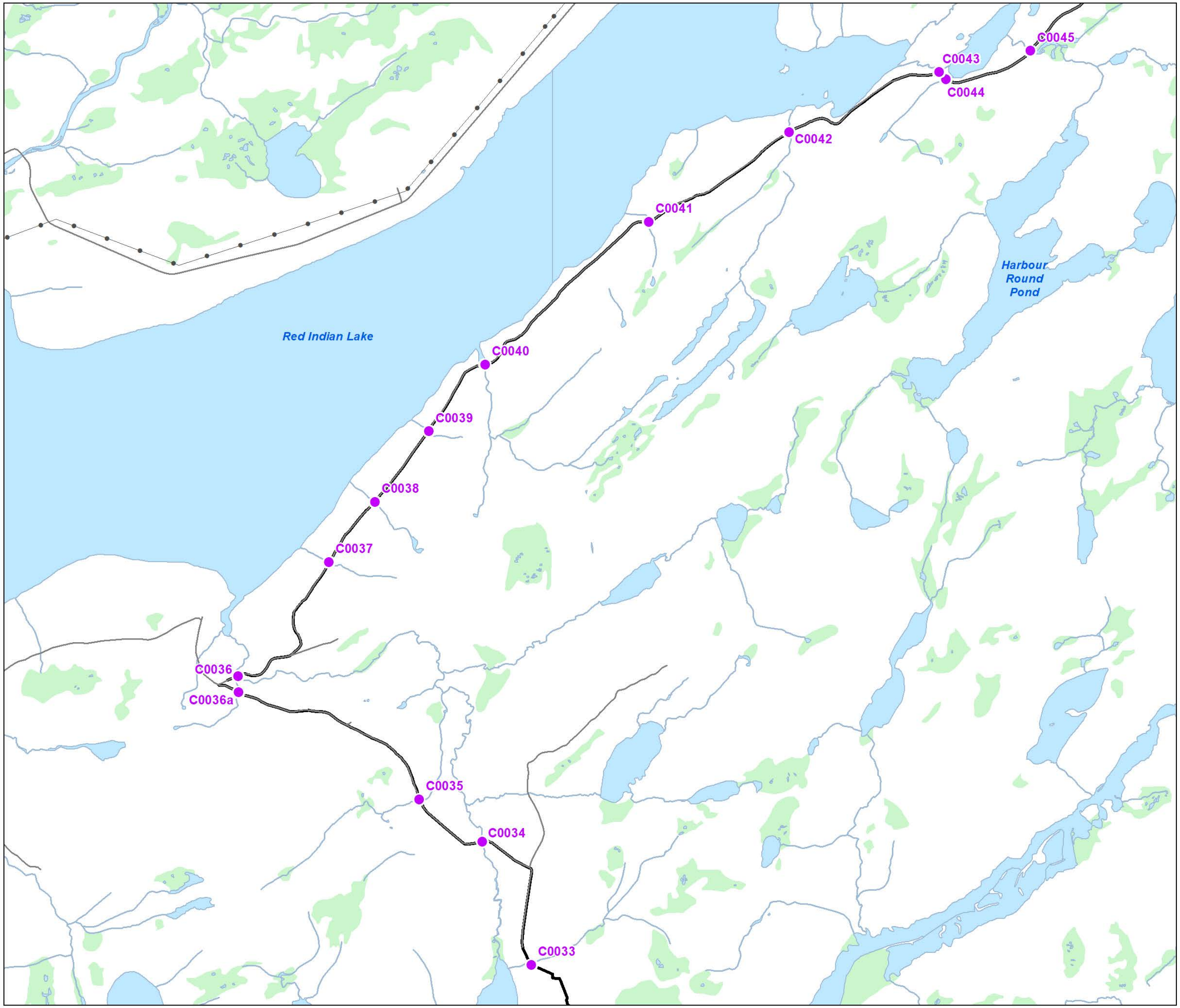

Stantec

- Stream Crossing

Mine Features

- Existing Access Road

Other Features

- Resource / Recreation Road
- Watercourse
- Wetland
- Waterbody

Project Location
Valentine Lake
Central Newfoundland, CA

Client/Project
121416408_041c
Marathon Gold Corporation
Valentine Gold Project
Environmental Impact Statement

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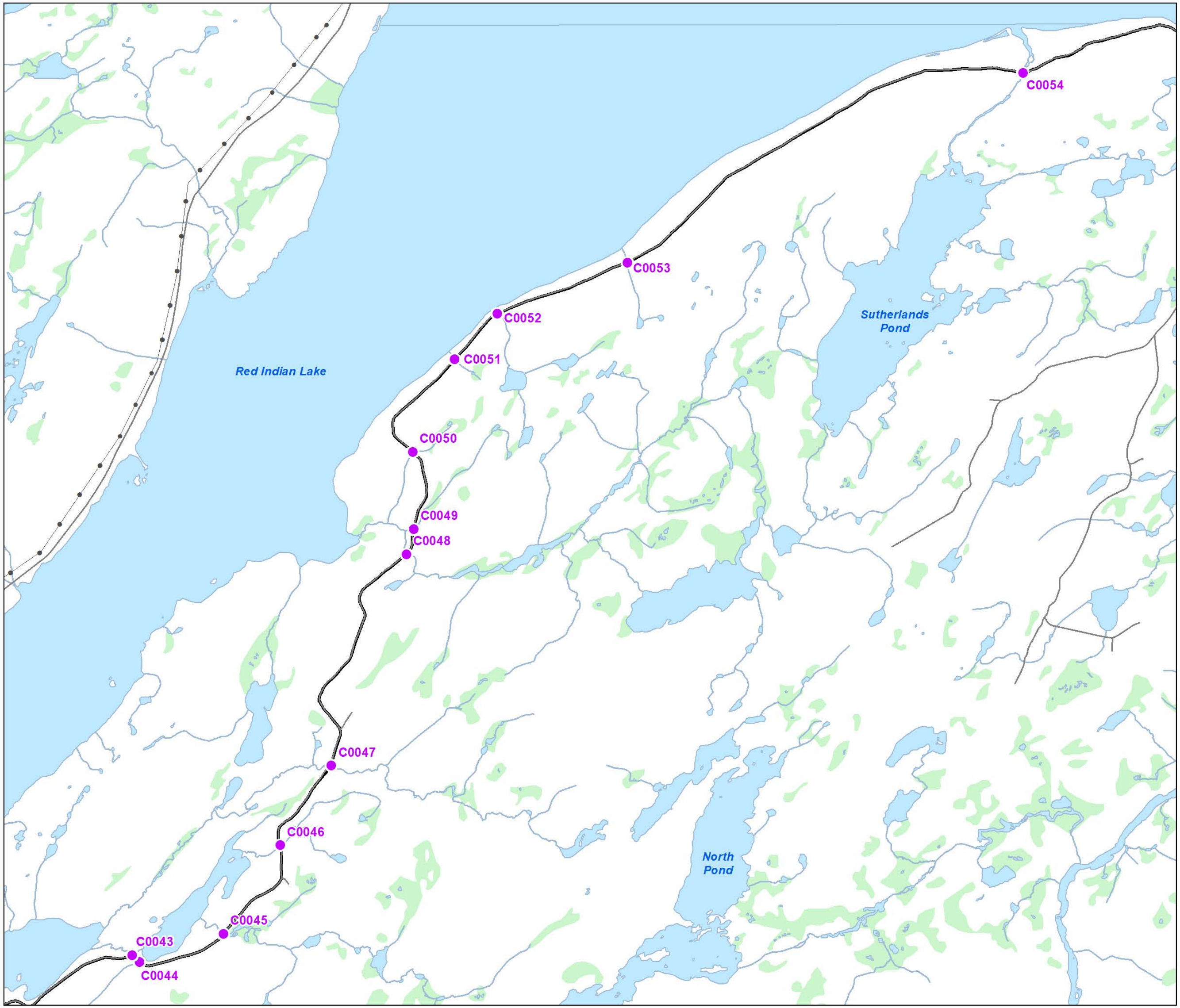

Stantec

- Stream Crossing

Mine Features

- Existing Access Road

Other Features

- Resource / Recreation Road
- Watercourse
- Wetland
- Waterbody

N

0 0.5 1

Kilometers

Project Location
Valentine Lake
Central Newfoundland, CA

Client/Project
121416408_041c
Marathon Gold Corporation
Valentine Gold Project
Environmental Impact Statement

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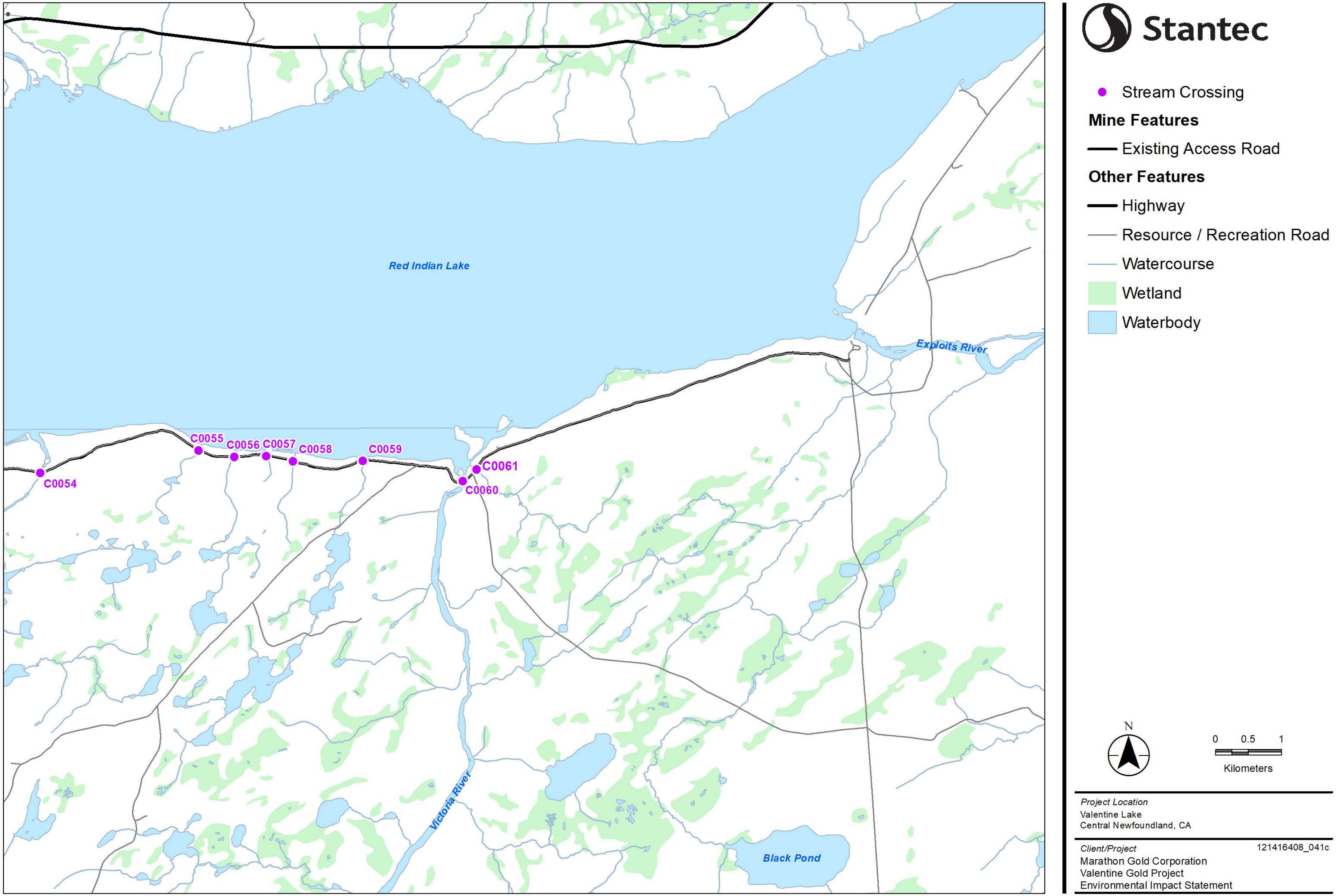

Project Location
Valentine Lake
Central Newfoundland, CA

Client/Project 121416408_041c
Marathon Gold Corporation
Valentine Gold Project
Environmental Impact Statement

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APPENDIX 2E

Site Photographs

Photo 1 Victoria Lake Reservoir and Victoria Dam

Photo 2 Existing Access Road

Photo 3 Victoria River Bridge

Photo 4 Area of Leprechaun Pit and Waste Rock Pile

Photo 5 Area of Marathon Deposit

Photo 6 Area of Marathon Waste Rock Pile

Photo 7 Area of Tailings Management Facility