Yukon Zinc
CORPORATION

Wolverine Project

TAILINGS AND INFRASTRUCTURE DESIGN AND CONSTRUCTION

VERSION 2006-01

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and
Hatch

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1 Introduction

This report has been prepared by Klohn Crippen Berger in conjunction with Yukon Zinc Corporation and Hatch. It provides the design criteria and construction plans pertaining to the tailings and seepage dams, spillways and diversions for the proposed Wolverine Project.

Section 2 provides details pertaining to the diversions structures that are required to manage surface waters in the tailings facility and industrial complex areas. Section 3 includes details pertaining to the design, construction and stability analyses for the dams and spillways within the tailings facility area.

Design drawings, contained in Appendix A, are at a preliminary design level (not for construction) and have been sealed by a Professional Engineer licenced to practice in the Yukon.

2 Diversions

Temporary and operational diversions of surface water are required for the project and include the following:

- Temporary diversion, for several months, of a portion of the flow in Go Creek to the tailings facility to provide a water supply for startup of the milling facilities.
- Operational diversions to direct clean surface water runoff around the industrial complex and tailings facility.
- Surface water collection around the industrial complex to a settling pond and subsequent transport to the tailings facility, and a seepage collection ditch along the toe of the tailings dam to collect seepage for reclaim to the impoundment.

2.1 Go Creek Temporary Diversion Inlet Structure

A temporary diversion structure is required on Go Creek to divert approximately 50,000 m^3 of water to the tailings pond prior to start-up of the milling facility. The diversion structure will be constructed on Go Creek immediately upstream of the mine access road (Drawing J1-C-001). The estimated width of the existing Go Creek channel at the Ordinary High Water Mark (OHWM) at the proposed structure is approximately 3 m.

The structure will consist of a concrete headwall and sump at the inlet of one of the culverts under the road (see Drawings J1-C-013 and -014). A 600 mm diameter slide gate will be fitted to the concrete structure to serve as the water supply intake. The water will be carried by a 600 mm diameter pipe to a small sediment pond located adjacent to the creek, and then it will be conveyed from the sediment pond by a 150 mm diameter pipeline installed along the access road to the tailings pond. This pipeline could be installed over its entire length to the tailings pond (about 1.8 km). Alternatively, if the mill start-up schedule allows, the pipeline could be connected to the 250 mm diameter tailings pipeline to carry the water the rest of the way to the tailings pond. In the latter case, the connection to the tailings line would be removed and sealed prior to the start-up of mill operations.

After sufficient water has been transferred to the tailings pond, the slide gate at the diversion structure will be removed and a short length of the 600 mm intake pipe will be plugged with concrete.

2.1.1 Diversion Structure Design Criteria

The existing stream gradient at the diversion location is approximately 7%. The culverts under the mine access road will be approximately 25 to 30 m long. Riprap erosion protection in the stream will extend about 3 to 5 m upstream and downstream of the culverts. The culverts under the road are designed to pass the 100-year flow in Go Creek. The flow from Go Creek to the sediment pond will be limited by the gate setting and the 600 mm diameter culvert.

The mean annual flow in Go Creek upstream of the airstrip (Monitoring Station W31), which is located close to the proposed stream diversion site, is estimated to be 0.022 m^3/s (79 m^3/hr). High flows in Go Creek occur during the months of May, June and July, with estimated mean monthly flows of 0.048 m^3/s, 0.045 m^3/s and 0.034 m^3/s, respectively. A maximum flow of 0.015 m^3/s will be diverted, over a period of approximately 6 weeks, to provide the startup water volume of approximately 50,000 m^3.

The selection of the maximum design flow to divert (0.015 m^3/s) was based on minimizing the change in seasonal flow and maintaining a minimum base flow in the stream. The diversion has been designed such that a minimum flow of 0.02 m^3/s will always be available in Go Creek to reduce any potential impact on the stream.

2.1.2 Diversion Structure Construction Plan

The diversion structure on Go Creek and the pipeline to the tailings pond are required to be in place such that water can be diverted to the tailings pond during the spring freshet prior to mill start up. Construction of the diversion structure will take place during low flow before spring freshet (likely April) in 2007, prior to commencement of mining operations. The streambed material consists of cobbly gravels and construction will be managed to minimize any potential sedimentation. The construction area will be isolated by temporarily diverting stream flow around the construction site via a temporary pipe such that the culverts and the headwall structure can be installed in the dry, while maintaining flows downstream. The distance downstream to aquatic habitat known to have fish presence is approximately 6 km.

Quality assurance/control measures include general construction and environmental monitoring during culvert installation, and construction of the headwall structure and appurtenant works.

The diversion intake will require periodic inspection during operation to ensure that the pipe inlet area is not blocked by debris or sediments. Removal of accumulated debris and sediment may be required during operation of the intake.

2.2 Surface Water Diversion Ditches

The diversion ditches are all upland diversion channels, which will be used to direct clean surface water around the industrial complex and the tailings facility. There are two main diversion ditches associated with the tailings facility (Ditch A and Ditch B; Drawing J1-C-001), and one main diversion ditch associated with the industrial area (Ditch 9; Drawing A0-C-006). The ditches consist of open channel excavations with erosion protection in areas where the gradients are high. The excavation slopes are typically 2H:1V. Erosion protection will be either with riprap from a rock quarry or with a geosynthetic product, such as Turf Reinforcement Mat (TRM). Unlined sections of the ditches will be seeded with a native grass species.

Tailings Facility Ditch A

Runoff from the uphill area between Go Creek and the upstream end of the tailings pond will be intercepted by the mine access road and will be passed across the road via culverts located along the road. Ditch A will intercept this runoff at an earthfill diversion berm constructed immediately upstream of the tailings pond to bypass the impoundment and discharge it into Go Creek. A 600 mm diameter culvert, controlled by a slide gate, will be installed under the berm to allow the transfer of water from Ditch A to the tailings pond, if required. The proposed diversion berm and culvert details are shown in Drawing J1-C-014. Ditch A is approximately 0.5 km long, with the gradient ranging between 0.2 and 4%. Profiles and typical sections of the diversion ditches are shown on Drawing J1-C-016.

Tailings Facility Ditch B

The second tailings facility diversion ditch, Ditch B, will intercept runoff along the mine access road, which will pass directly uphill of the tailings basin. Ditch B will direct the flow into the tailings dam spillway channel, which flows into Go Creek downstream of the seepage collection pond. Ditch B is approximately 1.5 km long with gradients between 0.5 and 24% (Drawing J1-C-016).

Industrial Complex Diversion Ditch

Diversion Ditch 9 will collect surface runoff from the area above the industrial complex and direct the flow around the area to the road culvert located at Station 0+540, which drains to headwaters of the Wolverine Creek watershed. Ditch 9 is approximately 0.8 km long, with most of it at a gradient of about 0.5%. The downstream end of the ditch is steeper with the gradient in the 10% to 12% range.

2.2.1 Diversion Ditch Design Criteria

The diversion ditches are temporary and will be decommissioned upon mine closure. Consequently a 100-year design return period for the ditches was selected to provide sufficient safety margin against normal runoff events and to provide for a low likelihood of exceedance occurring during operations. In addition, the dam safety design of the tailings dam assumes that the diversions could fail during an extreme rainfall event.

The estimated catchment areas, the design criteria and the design flows for the diversion ditches are presented in Table 2.1, and the proposed sizes of the ditches are shown in Drawings J1-C-013, J1-C-016 and A0-C-006.

Table 2.1 Diversion Ditch Design Parameters

Diversion Ditch	Estimated Catchment Area (ha)	Design Storm	Estimated Peak Design Flow (m^3/s)	Minimum Channel Freeboard (m)
Ditch A	67	100-yr 25-min	3.2	0.3
Ditch B	31	100-yr 20-min	1.7	0.3
Ditch 9	53	100-yr 25-min	3.3	0.3

The peak flows for the design of the diversion ditches were estimated using the Rational Method with a runoff coefficient of 0.6. The storm durations shown in Table 2.1 are the estimated times of concentration for the individual ditches. Direct measurements of snowmelt for the project site are not available, however regional analysis of snowmelt indicates a peak annual daily change in snowpack of about 5 to 15 cm, with a mean of 8.6 cm. Assuming a snowmelt rate of 15 cm/day and taking that as a snow water equivalent of 15 mm over a 12-hour period, and combining the snowmelt with the rainfall runoff, the peak flows presented in Table 2.1 would increase by less than 10%. This will slightly increase the flow depth in the ditches. The minimum freeboard of 0.3 m provided for each ditch is more than adequate to accommodate the extra flow, which may occur due to a rain on snow event.

Mean annual flows in Ditches A, B and 9 are expected to be 3, 2, and 3 L/s, respectively. Low flow in the ditches would occur during the winter, with almost zero discharge. High flows are expected to occur during the months of May, June and July, ranging between 5 and 7 L/s for Ditch A, between 3 to 4 L/s for Ditch B and between 4 and 5 L/s for Ditch 9.

2.2.2 Diversion Ditch Construction Plan

The diversion ditches will be excavated with conventional earthworks equipment. Ditches A and B will be constructed in fall 2006 to minimize infiltration in the construction area of the starter dam and impoundment lining areas, for construction after the following year’s freshet. The industrial complex diversion ditch will be constructed in conjunction with the earthworks proposed for July and August 2006.

Quality assurance/control measures include general construction monitoring of the channel excavation and placement of erosion protection materials such as grass seeding, TRM and riprap. The source of riprap will be qualified as part of the overall project and will be inspected visually for quality and size during placement.

The diversion ditches will be inspected a minimum of three times per year, with particular attention paid to inspection prior to and during spring freshet to ensure that the ditches and the culverts are not blocked. The inspection will clearly identify the potential for ice blockage. Clearing of vegetation and repair of erosion protection may be required from time to time during the 12-year mine life. A significant portion of Ditch B also serves as the roadside ditch for the mine access road. Some maintenance of this ditch may be required, if the road is kept operational beyond mine closure.

2.3 Industrial Complex Drainage Works

The drainage works will primarily consist of open channel ditches within and adjacent to the industrial complex area, with culverts where required to convey the storm water across the roads (Drawing A0-C-006). The collected flows are directed to a settling pond, which is provided to attenuate the flows, and to settle out solids, prior to pumping to the tailings pond.

There are two locations within the industrial complex area where the storm water has to be dropped over a substantial difference in elevation. The first location is at the road culverts at the entrance to the industrial complex, and the second location is at the two road culverts just to the west of the mill building. The flow will be conveyed down the slope by extending the culverts to the bottom of the slope. Riprap lined stilling pools will be provided at the pipe outlets for dissipation of energy.

The flow velocities along the drop pipe are expected to be very high during the design peak flows, and very large and likely impractical riprap sizes would be required to resist these velocities at the bottom of the drop. Smaller riprap sizes (Class 250), considered to be more reasonable and practical, have been selected for the stilling basins (Drawing A0-C-006). Given the short mine life, these riprap sizes should provide adequate service under most conditions; however, the stilling pools will require periodic inspection for movement of the riprap and signs of erosion, and remedial measures taken if necessary.

All ditches within the industrial area will be lined with relatively impervious till in order to reduce seepage from the ditches. All ditches, except those that require riprap protection, will be seeded with grass. Storm water collection and/or settling ponds are provided at two locations: one to the west of the mine portal and the other on the south side of the industrial complex. Water collected in the ponds will be discharged to the tailings facility.

2.3.1 Drainage Works Design Criteria

The drainage ditches within the industrial complex vary in length from as 50 m to 550 m and gradients vary between 0.3 and 7%. The sizes of the culverts within the industrial complex are shown in Drawing A0-C-006. Minimum culvert size will be 600 mm, regardless of the catchment size. A rainfall return period of 100 years was selected for estimating the sizes of the ditches and culverts within the industrial complex.

The design capacity for the main settling pond is 4500 m^3, which is approximately equivalent to a 24-hour, 50-year rainfall with a runoff coefficient of 0.9.

The total volume of surface runoff collected from the industrial complex on a mean annual basis is estimated to be about 51,000 m^3. The relatively long return period of 100-years was selected to minimize potential disruption of mill operations during heavy storm events. Also, the 1 in 100-year design should accommodate the snowmelt, which was not allowed for separately in the design.

2.3.2 Drainage Works Construction Plan

The drainage works will be constructed concurrent with site preparation work such as excavation, fill placement and grading.

Quality assurance/control measures include general construction monitoring of the channel excavation, placement of till linings, and placement of erosion protection materials such as grass seeding, Turf Reinforcement Mats (TRM) and riprap. The source of riprap will be qualified as part of the overall project material sourcing, and the riprap will be inspected visually for quality and size during placement.

The drainage works will be inspected a minimum three times per year to ensure that the ditches and culverts are not blocked and that the erosion control materials are intact. Some maintenance of the drainage works may be required from time to time during the mine life. The drainage works will be decommissioned and the industrial complex area reclaimed at mine closure.

3 Dams and Spillways

The tailings facility is comprised of the tailings dam and spillway and the seepage collection pond and spillway.

3.1 Tailings Dam

The tailings dam is a compacted earthfill water-retention dam, which creates an impoundment approximately 600 m long and 300 m wide. The maximum dam height is 25 m and 31 m at the project start-up (starter dam) and at the end of operation, respectively. The L-shaped dam will have a downstream slope of 3H:1V. The impoundment will store mill tailings, DMS float coarse waste and waste rock, i.e., all by-products of the mining and milling processes. The tailings impoundment will also be used to store mill process water (carried with the tailings slurry), underground discharges, sanitary discharge and contaminated surface water runoff from the industrial complex. Water within the facility will either be reclaimed to the mill to be used as process water, stored in the voids of the solids, lost to evaporation, or treated and discharged to Go Creek.

3.1.1 Site Conditions

The tailings facility is located near elevation 1300 m within a natural, northwest-southeast trending elongated depression, on the northeast valley slope of Go Creek, as shown on Drawing J1-C-001,. The depression is flanked on the downhill side by a natural ridge trending in the same direction that drops gently in elevation toward the upstream edge of the impoundment, and ends at the turning point of the L-shaped tailings dam.

The general topography of the project area consists of gently rolling hills and mountains, with elevation up to 1800 m. The tailings impoundment area is covered with small shrubs and grasslands. Permafrost has not been observed in the site investigation and drilling programs.

The dam foundation consists of up to 20 m thick competent till-like overburden overlying bedrock (Drawing J1-C-004).

Geology and Soils

The Wolverine Lake area lies within the limits of the McConnell Glaciation (youngest of the four glaciations in Yukon Territory), and most of the geomorphic features in the area are related to this glaciation. McConnell glacial ice covered this area between 14,000 and 35,000 years ago. As the McConnell ice retreated and down-wasted, a complex network of ice tongues developed in valley bottoms. Morainal deposits are found at lower to mid-elevation and valley floors, and may contain a more complex assemblage of glacio-fluvial, colluvial and fluvial sediments (Mougeot 1996).

Drawing J1-C-002, reproduced from Mougeot (1996), shows the Quaternary surficial geology units in the area. This local mapping is in general agreement with the regional setting presented by Jackson (1993; 1994) and Dyke (1990). The main glacial soils in the vicinity of the tailings impoundment consist of up to 20 m of silty, sand and gravel, with cobbles.

Superimposed on the drawing is the approximate exploration bedrock geology map prepared by Expatriate (2004). The area is underlain by bedrock strata generally paralleling the valley trend, i.e., striking in the direction of the valley. The bedrock consists of an interlayered sequence of volcanoclastic (rhyolite and quartz feldspar) and carbonaceous/argillic sediments, overlain with basalt. The iron formation, which hosts the ore zone, trends northwest-southeast throughout the project area.

Climate and Hydrology

The estimated mean annual precipitation for the Wolverine Minesite is 570 mm, and the estimated mean annual lake evaporation is 400 mm. Average snowpack for the minesite is estimated to be 175 mm snow water equivalent.

Table 3.1 presents the ratios of dry and wet year annual precipitations and mean monthly runoff flows, to the average mean annual precipitation and mean monthly flows, respectively (Madrone 2006). Additional climate and hydrology data is provided in the Environmental Assessment Report (YZC and AXYS 2005).

Table 3.1 Ratios of Dry and Wet Year Annual Precipitations and Mean Monthly Runoff Flows

Event	Ratio							
	200-yr dry	100-yr dry	10-yr dry	average	10-yr wet	100-yr wet	200-yr wet	1000-yr wet
Precipitation	0.586	0.622	0.762	1	1.159	1.388	1.441	1.561
Runoff flow	0.611	0.641	0.779	1	1.248	1.524	1.60	1.773

Table 3.2 presents the monthly precipitation and runoff distribution, and Table 3.3 presents a summary of monthly and annual flows for various site locations.

Table 3.2 Monthly Precipitation and Runoff Distribution

Month	% Precipitation	% Flow
January	8	0
February	6	0
March	5	0
April	4	1
May	7	19
June	11	35
July	14	17
August	11	9
September	10	9
October	9	6
November	8	3
December	8	1

Table 3.3 Expected Mean Monthly and Annual Flows (m^3/s) for Selected Stations

Station	W31	W16	W12	W44	W14
Month	Go Creek at airstrip (near Go Creek diversion) (4.7 km^2)	Go Creek at Hawkowl Creek (downstream of tailings facility) (10.2 km^2)	Go Creek at Money Creek (36.5 km^2)	Tailings Dam catchment (prior to diversion) (1.05 km^2)	Money Cr. downstream of Go Creek (238 km^2)
Jan	0.007	0.017	0.065	0.0015	0.48
Feb	0.007	0.016	0.061	0.0015	0.44
Mar	0.007	0.015	0.055	0.0014	0.39
Apr	0.010	0.021	0.079	0.0021	0.54
May	0.048	0.11	0.41	0.0099	2.94
Jun	0.045	0.11	0.49	0.0079	4.32
Jul	0.034	0.083	0.35	0.0062	2.970
Aug	0.021	0.050	0.21	0.0038	1.77
Sep	0.020	0.047	0.20	0.0036	1.64
Oct	0.018	0.044	0.18	0.0035	1.45
Nov	0.013	0.030	0.12	0.0025	0.91
Dec	0.009	0.021	0.083	0.0018	0.63
Year	0.022	0.051	0.21	0.0041	1.64

Groundwater

In the vicinity of tailings impoundment area the groundwater table within the bedrock is generally sloping southeast following the trend of the topography. Near the downstream end of the impoundment basin at TH05-8 and MW05-7, the piezometric pressure in the bedrock is slightly artesian, and the water table rises on the dam abutments. In general, the water table in the overburden is slightly lower than that in the bedrock. The groundwater table exhibits seasonal variation, reaching highest elevation after spring runoff season.

Baseline groundwater flow rates for the region have been estimated on the basis of a 10% infiltration rate over the hydrologic catchment areas and an annual precipitation of 570 mm as summarized in Table 3.4.

Table 3.4 Summary of Baseline Groundwater Flow for Selected Locations

Location	Catchment Area (km^2)	Groundwater Flow (m^3/s)	Predicted March Average Flow (m^3/s)
Go Creek, Near Dam	10	0.015	0.01
W – 12 (Go Creek)	36.5	0.060	0.083
W – 14 (Money Creek)	420	0.420	0.54

The main groundwater aquifer is the 10 to 20 m thick overburden overlying bedrock within the Go Creek Valley, which appears to be a hanging valley. Downstream of Go Creek, the morphology changes to a broader terraced valley where much thicker deposits of postglacial outwash soils provide a larger groundwater flow area.

Geotechnical Site Investigations

Geotechnical investigations for the tailings facility were carried out from July to September 2005 for the proposed site on the northeast valley slope of Go Creek. Further descriptive details and drill hole and test pit logs, as well as field penetration and in situ permeability test data are included in the Environmental Assessment Report (YZC and AXYS 2005). The investigations for the tailings facility included six test holes, two groundwater monitoring wells and 23 test pits as summarized below (see Drawing J1-C-003):

- Test Holes TH05-7 to TH05-11 were drilled along the dam alignment, and TH05-12 was drilled inside the tailings impoundment.
- Test Pits TP05-71 to TP05-83, TP05-94 and TP05-95 were excavated in the footprint of tailings dam, TP05-84 to TP05-86 excavated in the footprint of the seepage dam, and TP05-91 to TP05-93, TP05-96 and TP05-97 excavated along the diversion ditches and spillway channels.
- Groundwater monitoring well MW05-6 was drilled upstream of the impoundment, while MW05-7 drilled near the downstream toe of the tailings dam.

The drilling program included Standard or Large Penetration (SPTs and LPTs) tests and falling-head permeability tests in overburden materials (see Table 3.5), packer permeability tests and diamond drill coring with HQ3 or NQ2 core barrel in bedrock (see Table 3.6). The penetration tests were carried out to retrieve soil samples for further laboratory testing as well as to evaluate in situ soil density. Similarly, core samples of bedrock were obtained by diamond coring and kept on site.

Most of the test pits were excavated to a maximum depth of about 5 m using the backhoe. In areas inaccessible to the backhoe, shallower test pits were excavated manually or drilled manually using a hand-operated auger drill to a maximum depth of 1 m. Samples retrieved from the drillholes and test pits were further tested in Klohn Crippen Berger’s laboratory in Vancouver. Geotechnical laboratory testing on retrieved soil samples included visual classification, moisture content, gradation, standard Proctor compaction, triaxial permeability and shear strength tests.

Two 1" diameter 30 cm long piezometer tips were installed in most test holes with 1" Schedule 40 PVC riser pipes. One 2" diameter well screen with Schedule 80 PVC well pipe was installed in each monitoring well. A pressure gauge with a by-pass valve set up was installed at the top of each artesian installation. Ground temperature profiles were also recorded at Test Holes TH05-8 to TH05-11.

Falling-head permeability tests were conducted through the bottom of the monitoring wells or test holes, and the test results as summarized in Table 3.5 appear to overestimate the in situ permeability, based on comparison with gradations. This may be due to the boundary condition at the hole bottom/seal contact, which could increase the measured permeability values significantly.

Table 3.5 Summary of Falling-Head Permeability Test Results

Test Hole No.	Test Section Depth (m)		Test Section Diam. (mm)	Number of Tests	Range of k (cm/sec)
	from	to			
TH05-7	1.52	24.38	101.6	10	2.8E-04 to 7.8E-02
TH05-8	1.52	6.10	101.6	4	6.9E-03 to 1.7E-01
TH05-9	1.52	30.48	76.2	11	3.1E-03 to 4.1E-01
TH05-10	1.52	33.53	76.2	9	3.6E-03 to 7.1E-02
TH05-11A	3.05	9.14	76.2	4	3.6E-03 to 1.2E-01
TH05-11B	6.10	42.67	76.2	10	1.9E-03 to 2.1E-01
TH05-12	1.52	24.38	76.2	9	6.1E-04 to 2.6E-01
MW05-6	1.52	21.34	102	9	2E-04 to 1.8E-02

Table 3.6 Summary of Packer Permeability Test Results

Test Hole No.	Test Section Depth (m)		Test Section Diam. (mm)	Average k (cm/s)
	from	to		
TH05-8	24.70	30.80	96.0	5.5E-05
TH05-9	30.50	35.10	75.7	3.1E-04
TH05-10	35.05	38.10	75.7	1.5E-04
TH05-11	44.20	46.30	75.7	1.4E-05
TH05-12	27.58	29.60	75.7	1.6E-05
MW05-6A	21.30	25.70	96	1.2E-05
MW05-7A	24.70	30.20	96	3.6E-05

Laboratory Testing

Geotechnical laboratory testing included visual classification, moisture content, and gradation tests for overburden samples retrieved from field investigations. Additional standard Proctor compaction tests, triaxial permeability and shear strength tests were carried on the potential borrow materials for the dam fill as well as waste materials including tailings, DMS float and waste rock (Table 3.7).

Table 3.7 Summary of Engineering Properties Obtained from Laboratory Tests on Dam Fill, Tailings, DMS Float and Waste Rock

Type of Material	Unit Weight γ_{bulk} (kN/m^3)	Effective Shear Strength		Hydraulic Permeability k (cm/s)
		cohesion c' (kPa)	friction angle ϕ' (degree)	
Dam Fill	22.7	0	37	3 E-6
Tailings	23.1	0	34	7 E-6
Waste Rock	21	0	35	5 E-6
DMS Float	~21	0	46	3 E-1

Test results for dam fill, waste rock and tailings were obtained by consolidated-undrained triaxial shear tests with permeability measurement after consolidation and pore pressure measurement during shear. The shear strength and permeability results for the DMS float were determined by direct shear test and permeability test in a permeameter. The consolidation stresses used in the laboratory are selected to simulate the field condition.

The tailings sample was a mixture of four samples provided by Lakefield SGS (referred to as: F11 and F12 (Zn, Rougher Scavenger Tail), F23 and F32 (Zn 1st Cleaner Scavenger Tail). The specific gravity of the tailings sample was 3.71 and its grain size distribution was 80% silt and 10% clay, as obtained by a hydrometer analysis. The tailings settled out readily from a tailings slurry of about 14% solid content.

3.1.2 Dam Construction Materials

The Starter and Ultimate Tailings Dam are shown in plan and section in Drawing J1-C-012. The dam is designed as a zoned embankment dam using the centreline construction method. The main construction materials are summarized as follows:

Upstream and Downstream Shell Zones

General fill will be borrowed from the impoundment area and used to construct the upstream and downstream shell zones of most of the dam. The material consists of a silty sand and gravel with cobbles.

Upstream Shell Zone – Starter Dam to Elevation 1306 m

The upstream shell zone, up to elevation 1306 m, will consist of compacted mine waste rock, which is a silty sand and gravel, with cobbles.

Core Zone

A vertical central low-permeability core zone will consist of silty sand and gravel (with a minimum of 30% silt sizes), with a geosynthetic liner placed within the zone.

Geomembrane Liner

A 20 mil Enviro Liner 6000 will be placed within the low-permeability core and will be extended upstream to cover the entire footprint of the impoundment to minimize seepage potential.

Liner Protection

A liner protection layer will be required between the liner and the coarse angular mine waste rock, which will be placed in the upstream shell of the Starter Dam. The material will consist of select borrow material from the impoundment area, which will consists of rounded particles to minimize the potential for damage to the liner.

Downstream Drainage Blanket

A provision is included for constructing a horizontal downstream drainage blanket, which would be installed if the downstream shell saturation level is high. It is expected that with the installation of the geosynthetic liner the need for such a drainage blanket would be reduced. Pervious granular materials required for constructing the drainage blanket would be produced by screening material borrowed from within the impoundment.

Geotechnical Parameters

Drawing J1-C-012 shows the geometry and zoning of the Starter Dam and Ultimate Dam used in the seepage and stability analyses. Table 3.8 summarizes the properties of various materials used in the seepage and limit-equilibrium slope stability analyses. These properties are based on field and laboratory test data acquired for the project as discussed in Section 3.1.1 and supplemented by general properties available in literature.

Table 3.8 Summary of Material Properties Used in Seepage and Slope Stability Analyses

Material	Unit Weight γ_{bulk} (kN/m^3)	Effective Shear Strength		Hydraulic Conductivity k (cm/s)
		cohesion c' (kPa)	friction ϕ' (deg)	
Overburden	21.8	0	34	3.00E-03
Bedrock	22.8	0	40	1.00E-05
Waste Rock	21.8	0	32	1.00E-05
Dam Core	21.8	0	34	1.00E-06
Enviro Liner/soil contact	-	0	24	1.00E-11
Dam Shell	21.8	0	36	5.00E-05
Blanket Drain	21.8	0	32	1.00E-03
Tailings	23.2	0	25	7.00E-06
DMS Float	21.8	0	35	5.00E+00

Geochemical tests, including shake flask tests and acid base accounting (ABA) tests, were carried out for samples collected from the test pits located in the impoundment and from the project borrow area located northwest of the tailings facility (see Drawing J1-C-003). These tests indicate that the borrow materials have a low leaching potential, and are not potentially acid generating.

3.1.3 Dam Construction Plan

The dam will be constructed in two stages, the Starter Dam to elevation 1310 m and the Ultimate Dam to elevation 1316 m. Drawings J1-C-009 and -010 provide the excavation and fill plans for the Starter Dam and Ultimate Dam, respectively, while Drawing J1-C-011 provides typical sections.

The dam foundation area will be drained with perimeter diversion ditches and internal ditches, as required to ensure a drained foundation suitable for placement of damfill materials. Surface runoff, if present during construction, will be collected in an upstream ditch and pumped to the downstream side of the dam.

The dam footprint will be stripped of all organic, loose and soft materials and proof-rolled with five passes of a 10 tonne vibratory roller compactor. All damfills will be placed in 300 mm thick lifts and compacted to a minimum of 97% of the standard Proctor density. For the mine waste rock that may be impractical to conduct Proctor density tests, the material will be watered and compacted with a minimum of five passes of a 10 tonne vibratory roller compactor.

The geosynthetic liner will be placed in segments and joined in the field. The liner will be placed in a 'zig-zag' geometry up the centerline of the dam through the impervious vertical core, alternating along upstream and downstream side slopes, as the fill on either side is raised.

The Starter Dam will be constructed prior to startup of the mining operation and the second stage will be constructed in approximately Year 4 of mining operation. All work will be inspected and monitored by a qualified professional engineer. The main QA/QC components include the following:

- foundation inspection and approval
- density and moisture content testing of fills every 4000 m^3
- gradation analysis of fills every 10,000 m^3
- liner joint tests and QA/QC of liner placement

Inspection and maintenance procedures will include minimum daily visual inspections of dam and area directly downstream of the dam, construction progress reviews by a qualified professional engineer as required and annual dam safety review by a qualified professional engineer. In addition, the following will be conducted:

- four piezometers will be installed in the dam downstream shell to monitor damfill saturation level
- inclinometer(s) will be installed through the dam foundation overburden into bedrock and raised with dam fill and monitored to confirm favourable foundation behaviour, as required
- ten survey monuments on the crest/downstream slope will be monitored quarterly
- four groundwater monitoring wells will be installed downstream of the dam, with screened locations in the foundation soils and weathered bedrock unit. Water quality testing will be carried out quarterly on samples collected from the wells
- the water balance will be updated annually to include: actual precipitation/runoff and mill inflows (tailings, DMS float, waste rock, and transport water), actual surveyed bathymetry of pond and volumes of water treated for discharge

3.1.4 Seismic Dam Design

Seismicity

The most significant inland seismicity occurs along segments of the Denali fault zone system, where the seismicity rate is an order of magnitude lower than that in the coastal region. The region between the Denali and Tintina systems is relatively aseismic, with few small earthquakes. There appears to be an alignment of epicentres along the Tintina fault, and most of the activity is at the northern end, close to the Alaska border. Farther inland, the only significant seismicity is along the eastern edge of the Cordillera, more than 600 km from the active plate boundary. This fold and thrust belt seismicity is concentrated in two areas: the MacKenzie-Ogilvie mountains region and the Richardson Mountains region (Hyndman et al. 2005).

The probabilistic seismic hazard assessment has been updated to use both the GSC-H and GSC-R seismic source zonal models developed by the Geological Survey of Canada for the new National Building Code of Canada-NBCC 2005 (Adams and Halchuk 2003). Moreover, the update incorporated the work conducted by Atkinson (2004), where an apparent linear alignment of seismicity in the region along the Tintina Trench fault system was grouped into a Tintina seismic source zone. The computed site peak horizontal acceleration is shown in Table 3.9. De-aggregation of the seismic hazard at the site, corresponding to the 10,000-year return period, indicates that the mean magnitude is 6.1, and mean epicentral distance is 34.8 km.

Table 3.9 Probabilistic Evaluation of Peak Horizontal Ground Acceleration at Project Site

Annual Probability of Exceedance	Return Period (years)	Peak Ground Acceleration PGA (G)	
		GSC-H 2005 model	GSC-H 2005 model with Tintina Source zone
0.0021	475	0.08	0.097
0.001	1000	0.10	0.12
0.00040	2475	0.14	0.15
0.0001	10,000	0.20	0.22

Two design earthquake scenarios were considered for the deterministic evaluation for the site peak horizontal ground acceleration as shown in Table 3.10: a local earthquake at the site with magnitude 6; and a nearby earthquake at the Tintina Fault with magnitude 7.2.

Table 3.10 Deterministic Evaluation of Peak Horizontal Ground Acceleration at Project Site

Earthquake Scenario	Magnitude	Epicentral Distance (km)	Focal Depth (km)	Peak Ground Acceleration PGA (G)
Local	6	0	2.9	0.34
Tintina Fault	7.2	53	2.9	0.11

The design earthquake selected for the tailings dam was based on the Canadian Dam Safety Guidelines (CDA 1999). Since the tailings facility is classified as 'High' consequence, the annual probability of exceedance of horizontal peak ground acceleration is chosen as 0.0001, corresponding to a return period of 10,000 years. For the seepage dam, the annual probability of exceedance of horizontal peak ground acceleration is selected as 0.0021, corresponding to a return period of 475 years, as no tailings are stored behind the dam.

Pseudo-static slope stability analyses were carried out using a seismic coefficient (k_h) of 0.125, corresponding to a design earthquake magnitude of seven based on interpolation (Seed 1979). In the analyses, no seismic-induced excess pore pressure was assumed in either the dam or foundation material. Static and pseudo-static stability analyses were carried out using the computer program SLOPE/W (Geo-Slope 2004) and the Morgenstern-Price method to determine the factor of safety and the results are summarized in Table 3.11.

Table 3.11 Summary of Safety Factors for Tailings Dam – Downstream Slope

Dam Crest	Static	Pseudo-Static Seismic Coefficient (0.125) a = 0.125 g
Starter Dam - Crest El. 1310 m	2.2	1.5
Ultimate Dam – Crest El. 1316 m	2.2	1.5

Note: The upstream slope of the Starter Dam has temporary static factor of safety of 1.5.

Dam Foundation Liquefaction Assessment

The liquefaction assessment was carried out in general accordance with the Seed simplified approach as described in Youd et al. (2001). The earthquake induced Cyclic Stress Ratio (CSR) was computed using the Seed's simplified relationship for level ground condition. The Cyclic Resistance Ratio (CRR) was estimated based on SPT $(N_1)_{60cs}$ values derived from the measured SPT and LPT blow count data. The factor of safety against liquefaction (FOS_{Liq}), which is defined as the ratio of CRR to CSR was determined to evaluate the liquefaction potential of granular soils at the site. The liquefaction assessment was based on the LPT data at test holes TH05-07 and TH05-08 and SPT data at test holes, TH05-09 and TH05-10. The LPT data was first converted to equivalent SPT data based on Daniel et al. (2003).

The following two design earthquake scenarios were considered in the liquefaction assessment:

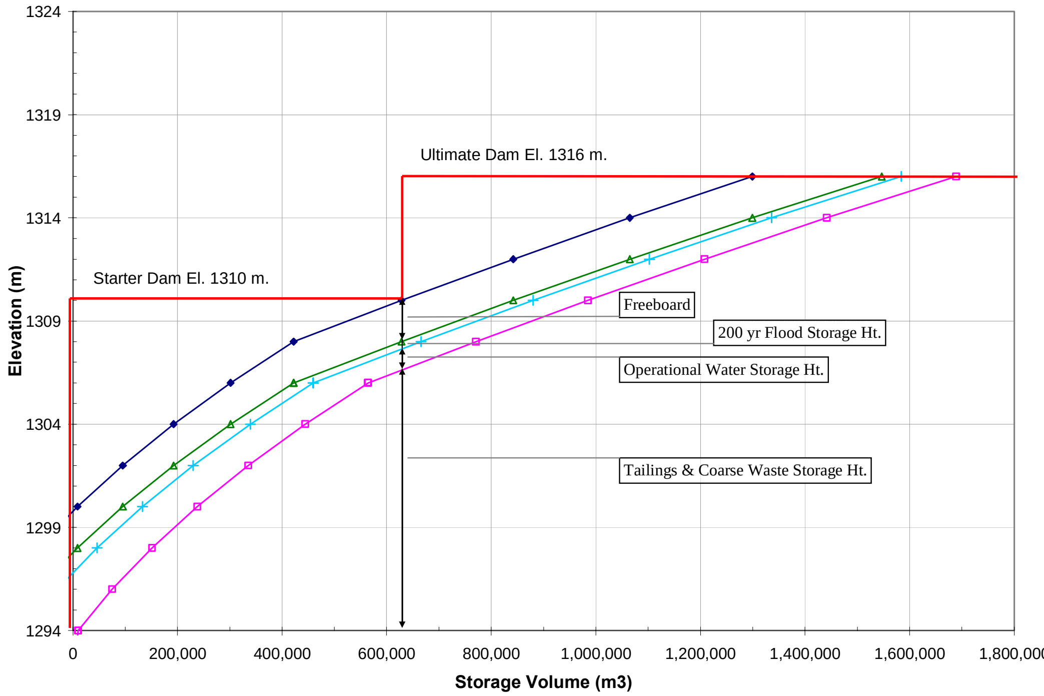
- Scenario 1: Earthquake with magnitude M7 and Peak Ground Acceleration (PGA) of 0.22 g
- Scenario 2: Earthquake with magnitude M6 and PGA of 0.34 g

The PGA values listed above are representative for the 'firm ground' conditions and they were amplified at the surface. Note that conventional method was used to determine the LPT/SPT blow counts N_{LPT}/N_{SPT}. However, if refusal was reached after 150 mm of penetration during the LPT or SPT testing, the LPT/SPT blow counts were estimated based on the data up to the point of refusal. The calculated factor of safety against liquefaction (FOS_{Liq}) is generally greater than 1.5 for Scenario 1 M7 event, and greater than 1.8 for Scenario 2 M6 event.

3.1.5 Reservoir Parameters

The tailings dam will form an impoundment with a maximum surface area of approximately 16 ha and will store a total volume of 1.1 Mm^3. The catchment area of the impoundment is 105 ha, of which, approximately 90 ha will be diverted around the impoundment during operations. Figure 3.1 shows the volume versus elevation curves for the solids storage, operating water and flood water requirements.

Figure 3.1 Tailings Impoundment: Elevation versus Storage



The tailings and coarse waste will be deposited within the tailings impoundment as shown in Drawings J1-C-005 and J1-C-006 (Starter Dam and Ultimate Dam on closure, respectively). Typical sections are shown schematically in Drawings J1-C-007 and J1-C-008 for various stages of mining operation. The tailings will be spigotted from a single point on the left flank of the impoundment and a water reclaim barge will be located near the left abutment of the dam. The tailings will form a beach above water, which will slope at approximately 1% towards the water pond. The DMS float material will be placed by trucks in the interior sections of the impoundment. Waste rock will also be placed by trucks within the active DMS float placement area.

Treatment and discharge of tailings supernatant will be required during the operational phase of the mine to prevent accumulation of excess water within the tailings impoundment. An emergency spillway will be maintained during mine operation to discharge floodwater for severe hydrological events. At closure, the diversion ditches will be decommissioned and a permanent spillway will be constructed. On closure, a minimum water cover of 0.5 m will be maintained within the impoundment to prevent the development of acid rock drainage from the mine waste (combined tailings, DMS float and waste rock materials).

Precipitation runoff and any tailings pond seepage will be collected in seepage ditches and the seepage recovery pond, and recycled back to the tailings pond.

3.1.6 Seepage Analysis

A geosynthetic liner system is to be placed over the entire footprint of the tailings impoundment to minimize the potential for seepage entering the groundwater supply.

A steady-state seepage analysis was carried out using the computer program SEEP/W (Geo-slope 2004). A two dimensional representative section through the main part of the impoundment was analyzed, with an average applicable dam length of 250 m used for calculation of the total seepage. A parametric analysis was carried out assuming various permeabilities for the impoundment basin and dam core to determine the lining requirements for the facility, and the results are summarized in Table 3.12.

Table 3.12 Summary of Seepage Analyses for Ultimate Tailings Dam

Impoundment Condition		Total Seepage (L/s)
liner preparation	basin permeability (cm/s)	
Unlined facility	10^{-4}	5.6
Soil lined facility	10^{-6}	4.2
Degraded geomembrane	10^{-8}	0.17
Complete geomembrane liner	-	$2\ (10)^{-4}$

An estimate of potential liner leakage has been made on the basis of a 2-D SEEP/W model, simulating defect holes in the liner, assuming medium quality installation procedures (note that this is conservative as the liner will be installed will high quality installation procedures). Unlike liner leakage calculations associated with the heap leach and landfill industries, the leakage is 'inflow' controlled by the permeability of the tailings. The Wolverine tailings permeability is in the order of $7\ (10)^{-6}$ cm/s, although there could be some segregation and areas where tailings may not be placed beneath the DMS float material. Based on the conservative tailings permeability, the liner leakage rate would be equivalent to 700 L/Ha/year, or $2\ (10)^{-4}$ L/s, which is negligible. For comparison purposes, an estimate of the liner leakage was made on the basis of standard EPA liner calculations for heap leach and landfill facility, assuming no tailings, and an impoundment full of water. For a moderate level of quality control, the estimated leakage would be 0.2 L/s, which is below the design criteria.

3.2 Tailings Dam Spillway

The tailings dam spillway is designed to route floodwater without overtopping the dam crest. The spillway inlet is a 3 m wide trapezoidal channel with 2H:1V side slopes and the channel invert is located 2.0 m below the crest of the tailings dam (Drawings J1-C-013 and -015).

The general site conditions described for the tailings dam are applicable to the spillway area. The site investigations carried out to date indicate the foundations for the spillway will consist primarily of medium dense to dense silty sand and gravel with cobbles, with potential bedrock over portions of the spillway. Detail design and construction of the spillway will maximize the extent of spillway over rock, thereby reducing the requirements for riprap. The main construction materials will be riprap bedding and riprap. The material will consist of hard durable rock obtained from a rock quarry.

3.2.1 Spillway Construction Plan

The spillway will be constructed using conventional earthworks equipment. Drilling and blasting will be required for sections of the spillway cut through bedrock. If the control section of the spillway is excavated through soil, then it will be lined with riprap to not only provide erosion protection but to also discourage future vegetation growth in the channel. The channel side slopes will be excavated at 2H:1V; however, this will be assessed during construction and the side slopes will be adjusted, if deemed necessary, to provide long-term stable slopes. If the control section of the spillway is located in bedrock then the channel side slopes will be based on rock quality. Depending on the depth of cut and rock quality, benches may be provided to intercept rockfalls. All loose rock will be scaled and removed during excavation and at mine closure.

Quality assurance/control measures include general construction monitoring of the channel excavation and placement of erosion protection materials such as grass seeding and riprap. The source of riprap will be qualified as part of the overall project and will be inspected visually for quality and size during placement.

The spillway will be inspected a minimum of three times per year, with particular attention paid to inspection prior to the spring freshet to ensure that the channel is not blocked and that the channel erosion protection is intact. The performance of the spillway will be monitored during the high runoff periods while the mine is in operation, such that potential future problems, if any, can be identified and remedied prior to mine closure.

3.2.2 Spillway Flood Design

The selection of the design flood for the tailings impoundment is based on the Canadian Dam Safety Guidelines (CDA 1999). The consequence classification of the Tailings Dam was assessed to be ‘High’. According to the Dam Safety Guidelines, the inflow design flood for facilities in the ‘High’ category should be between a 1000-year flood and the Probable Maximum Flood (PMF), and a 10,000-year return period was selected for the design of the closure spillway (Table 3.13).

The expected operating life of the mine was also taken into account in the selection of the design floods for temporary facilities, such as the Starter Tailings Dam spillway and the Seepage Recovery Dam spillway. Based on current resources, the mine is expected to be active for a period of about 12 years. During this time all facilities related to the tailings impoundment would be closely and frequently monitored, and personnel, equipment and materials are expected to be readily available in the event that remedial measures are required to be taken under routine and/or emergency maintenance. Therefore, a 200-year design return period was selected for the Starter Dam spillway, with the proviso that the spillway would pass the 10,000-year flood without overtopping the dam. A 100-year design return period was selected for the Seepage Pond spillway.

Table 3.13 Flood Design Criteria for Tailings Dam Spillway

Stage	Min. Design Flood Return Period (years)	Flood Storage & Freeboard Allowance	Comments
Starter Dam Spillway	200 & 10,000	0.9 m (see below)	Assume that upland surface water diversion ditches have failed. Spillway also must be able to pass the 10,000-year flood without overtopping the dam.
Ultimate Dam Spillway	10,000	0.95 m (see below)	Assume that upland surface water diversion ditches have been decommissioned.
Tailings Pond flood storage allowance below spillway level during mine operation	-	0.3 m	For routing of design flood through the tailings pond after closure, the initial water level is assumed to be at the spillway level, i.e., flood storage allowance is assumed to be zero.
Tailings Dam freeboard to normal pond operating level	-	2.0 m	

The 1 to 30-day flood inflow hydrographs for the Tailings and the Seepage Ponds were developed using the method outlined herein. First, the peak runoff for short duration (i.e., 30 min.) rainfall for the appropriate return period was estimated using the Rational Method with a runoff coefficient of 1.0. Inflow hydrographs were then prepared based on the estimated short duration peak runoff and approximately 2 to 4 times the estimated long duration (i.e., 1 to 30-day) volume of runoff. The inflow hydrographs for various scenarios were routed through the Tailings Pond in order to estimate the peak pond water level and the peak discharge. The size of the spillway was determined such that adequate dam freeboard is available under flood conditions with a reasonable spillway size.

The results of the flood routing with the selected spillway size are summarized in Table 3.14 and discussed below.

Starter Dam Condition

The 10,000-year, 30-day storm with the diversion ditches assumed to have failed was found to be more critical in terms of flood discharge rate and dam freeboard. The peak inflow to the pond for the 10,000-year storm is estimated to be about 9.5 m^3/s, which attenuates to about 8.0 m^3/s as the flood passes through the pond and the spillway. The pond water level will peak at Elev. 1309.1, thus leaving a freeboard of about 0.9 m between the dam crest and the peak flood level.

Ultimate Dam Condition

The 10,000-year flood inflows for the ultimate tailings dam will be similar to that for the Starter Dam but, because of the larger pond size, the flow will attenuate to about 7.7 m^3/s as the flood passes through the pond. For the 10,000-year, 30-min. storm, minimal rise in pond water level is expected and about 1.95 m of freeboard between the peak flood level and the top of the dam will be available. For the 10,000-year, 30-day storm, the available freeboard will be reduced to about 0.95 m which is considered to be reasonable for such an extreme event.

The flood routing presented above is based on rainfall. The effect of combined runoff from rainfall and snowmelt on the Tailings Pond freeboard was also investigated. The maximum snowmelt rate of 15 mm/day snow water equivalent was added as the base flow to the rainfall inflow hydrograph and the combined runoff was routed through the Tailings Pond. The flood routing indicates that the pond water level will rise an additional 0.0 to 0.05 m due to the snowmelt. Therefore, the freeboard between the peak flood level and the top of the dam could be up to 0.05 m less than that indicated in Table 3.14. The reduced freeboard would still be within the acceptable range of flood freeboard for this type of facility.

The method used to develop the inflow hydrographs tends to over-estimate the rise in pond water level, and pond outflows. Furthermore, the highest regionally recorded snowmelt rate was assumed for the project and actual snowmelt rates for the site may be lower. The inflow hydrographs and the snowmelt rates should be re-examined, if additional hydrology data become available during the detailed design phase.

Table 3.14 Estimated Spillways Catchment Areas, Design Flood Flows and Freeboards

Water Management stage	Estimated Catchment Area (ha)	Design storm	Estimated Peak Discharge (m^3/s)	Minimum Channel Freeboard (m)	Estimated Dam Freeboard to Flood Level (m)	Assumptions
Starter Dam	16	200-yr, 30-min	0.0	0.6	2.2	Surface water diversion ditches assumed to be functioning
	16	200-yr, 30-days	0.0	0.6	2.0	
	16	10,000-yr, 24-hour	0.4	0.6	1.9	
	105	200-yr, 30-min	0.0	0.6	2.1	Surface water diversion ditches assumed to be not functioning
	105	10,000-yr, 30-min	0.0	0.6	2.0	
	105	10,000-yr, 30-days	8.0	0.6	0.9	
Final Dam	105	10,000-yr, 30-min	0.2	0.6	1.95	Surface water diversion ditches assumed to be decommissioned
		10,000-yr, 30-days	7.7	0.6	0.95	

Notes: [1]The peak design flows and freeboard for spillways are based on a 3 m wide trapezoidal channel with 2H:1V side slopes, with channel invert located 2.0 m below dam crest.
[2]For routing of the 200-year flood through the Starter Dam, the 0.3 m flood storage allowance is assumed to be fully available. That is, the initial pond level at the beginning of the storm is assumed to be 0.3 m below the spillway invert. For routing of the 10,000-year flood, the flood storage allowance is assumed to be zero for both the Starter Dam and the Ultimate Dam.

3.3 Seepage Collection Pond

The seepage collection pond is a safety measure that is installed to provide for collection of seepage from the tailings dam, if it occurs, for subsequent return to the impoundment (Drawing J1-C-005). It is probable, however, that the flow into the seepage recovery pond will be mostly due to precipitation and runoff from the tailings dam downstream slope and the immediate catchment area of the seepage pond, with little or no seepage contribution from the tailings impoundment. The quality of water in the seepage pond will be monitored, and if it meets criteria for discharge water quality, the water could be released.

The general site conditions are the same as described for the tailings dam.

The dam will consist of a 5 m high earthfill dyke, which will constructed with a cut and fill operation. The dyke will be a homogeneous embankment fill structure consisting of silty sand and gravel. The pond will be approximately 40 by 60 by 4 m deep. A spillway will be located in the left abutment of the seepage dam. The seepage dam and spillway will be used during operations and decommissioned on closure.

3.3.1 Seepage Collection Pond Construction Plan

Dyke construction will be as per the tailings dam.

3.3.2 Seepage Dam Design

Seismic design criteria is for the 1 in 475-year return period. The annual probability of exceedance of horizontal peak ground acceleration is selected as 0.0021, corresponding to a return period of 475 years, as no tailings are stored behind the dam.

The design criteria for the spillway are as follows:

- design flood return period is 100 years
- design storm duration is 10 minutes
- upland surface water diversion ditches/spillways are assumed to be functioning
- catchment area with diversion ditches/spillways functioning is 9 ha
- minimum spillway channel freeboard is 0.3 m

Results of the design flood routing for the estimated peak spillway discharge is 0.7 m^3/s and the estimated dam freeboard to flood level is 0.5 m.

3.3.3 Reservoir Parameters

The seepage collection pond has an area of about 0.5 ha and a storage volume of approximately 5000 m^3. The estimated flow rate is 350 USgpm and heads for pumping the water from the seepage recovery pond to the tailings pond are 26 m to the Starter Tailings Dam crest and 32 m to the Closure Tailings Dam crest. The pumping rate of 350 USgpm is approximately equal to the average pond inflow rate for a 5-year, 24-hour rainfall. The heads are the difference in elevation between the minimum seepage pond water level and the crest of the tailings dam. At 350 USgpm, it will take about 2.6 days to pump 5000 m^3 out of the pond.

4 References

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Appendix A

DRAWINGS

J1-C-001 Tailings Facility – General Arrangement

J1-C-002 Tailings Facility – Surficial and Bedrock Geology Plan

J1-C-003 Tailings Facility – Site Investigation Plan

J1-C-004 Tailings Facility – Subsoil Profiles

J1-C-005 Tailings Facility – Starter Impoundment – Tailings and Coarse Waste Placement Plan

J1-C-006 Tailings Facility – Ultimate Impoundment – Closure Plan

J1-C-007 Tailings Facility – Starter Impoundment – Typical Sections

J1-C-008 Tailings Facility – Ultimate Impoundment – Typical Sections

J1-C-009 Tailings Facility – Starter Dam – Excavation and Fill Plan

J1-C-010 Tailings Facility – Ultimate Dam – Excavation and Fill Plan

J1-C-011 Tailings Facility – Impoundment – Excavation and Fill – Typical Sections

J1-C-012 Tailings Facility – Tailings Dam – Plan and Sections and Impoundment Storage Volume

J1-C-013 Tailings Facility – Diversion Ditches and Spillways – Plan and Typical Sections

J1-C-014 Tailings Facility – Go Creek and Ditch “A” Diversion Structures

J1-C-015 Tailings Facility – Closure Spillway Plan, Profile and Section

J1-C-016 Tailings Facility – Diversion Ditches A and B Profiles and Sections

A0-C-006 Industrial Complex – Area Layout