WEST TWIN DISPOSAL AREA CLOSURE PLAN

MARCH 4, 2004

CANZINCO LTD.

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NANISIVIK MINE, A DIVISION OF CANZINCO LTD.

NANISIVIK MINE CLOSURE STUDIES

WEST TWIN DISPOSAL AREA CLOSURE PLAN

(WATER LICENSE PART G, ITEM 15)

FINAL

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EXECUTIVE SUMMARY

Under the terms of Water Licence NWB1NAN0208 issued by the Nunavut Water Board (NWB), CanZinco Ltd., the current owner of the Nanisivik Mine is responsible for continuation of on-site environmental protection activities and developing for submission and approval of a final Reclamation and Closure Plan ("RCP").

The Nanisivik Mine FCRP has been developed, as per the terms of the Water Licence as a series of stand-alone documents, each addressing in detail the information and proposed closure measures for one specific component or topic area. This document and the information presented herein are provided in response to the requirements for report Part G, Item 15, the West Twin Disposal Area (WTDA) Reclamation and Closure Plan.

In accordance with Part G, Item 15, of the Water Licence, this report provides the following requirements for the WTDA Plan:

1. A brief description of historical operating practices, water movement and overall function of the system.
2. An updated water balance for the system.
3. Current site assessment including characterization of all tailings both for physical properties (gradation, density, mineralogy) and thermal conditions in the surface cell dike and possibly the reservoir.
4. Cover design and description of all construction activities associated with the closure plan, predictions of site stability and water quality with details of analyses that support the plan.
5. Contingency plans for dealing with uncertainties and adverse performance during the post-closure monitoring period. These plans need to include a discussion of events that trigger their implementation.
6. A monitoring program that includes: permafrost stability, deformations of both the dike and soil cover as well as water quality determinations.
7. An appendix that constitutes a construction plan with material specifications, a quality control plan and as-built drawings stamped by an engineer.

Additionally, requirements 6 and 9 of Part G, Item 4 (the Covers Report), which apply to the water covered tailings in the Reservoir, are addressed within the context of the closure plan provided herein. Those two requirements are as follows:

6. The bathymetry of sub-aqueous tailings in West Twin Lake Reservoir which shows the extent of tailings located within 1.0 m of the water surface, and plans for mitigation of wave action on these tailings.

9. An evaluation of alternatives for increasing minimum water depth in the Reservoir with emphasis on possible effects of waves and winter ice cover on long term water quality.

The report provided herein attempts to be comprehensive in its treatment of the elements and technical information and issues with regards to the closure of the WTDA. The following list provides guidance on the closure elements covered in this report, as opposed to other submitted reports:

- Surface Cell and Test Cell cover design – covered in Cover Designs Report (Part G, Item 4).
- Talik characterization – covered in Talik Report (Part G, Item 5).
- Reclamation materials – covered in the Quarry Report (Part G, Item 6).
- West Twin Dike spillway design – covered in Surface Cell Spillway report (Part G, Item 7).
- Reservoir water cover and submerged tailings – covered in current report.
- Reservoir shoreline protection design – covered in current report.
- Breaching of baffle dike in Reservoir and access road berm at Polishing Pond – covered in current report.
- Breaching of Polishing Pond Outlet Structure and design of Outlet Channel and Overflow Weir – covered in current report.
- Erosion protection for East Twin Diversion Dike and channel – covered in current report.

As noted, the current report attempts to be comprehensive on the topic, without repeating all applicable detail from each of the individual component reports.

The WTDA consists of an upper solids retention pond, known as the Surface Cell and a lower water retention pond, known as the Reservoir. An earthen dike, the West Twin Dike, separates the Surface Cell and Reservoir. The Reservoir is further divided by the Test Cell Dike, which separates the Reservoir and the Test Cell. The Test Cell was used to evaluate the performance of several test cover designs. Both dikes are constructed of frozen shale fill and are founded on frozen, settled tailings. The current crest elevation of the West Twin Dike is about 388 m. The Test Cell Dike was raised in two stages. The first stage was raised to elevation 383.5 and the second stage was to about elevation 385.5. The staged construction resulted in the formation of a bench on the Reservoir side of the dike.

Excess water from the Surface Cell is transferred to the Reservoir by pumping or a siphon system that controls water levels. The Reservoir and a final Polishing Pond are separated by a causeway and stop log structure, which controls the water level in the Polishing Pond. Water from the Polishing Pond is discharged to Twin Lakes Creek through the West Twin Outlet Structure, a 3 to 5 m high earth fill dam with a valve controlled, concrete lined spillway. Excess water from the WTDA is then discharged to the environment via the West Twin Outlet Structure between July and September of each year.

The primary risks posed by the WTDA facilities are related to the potential for acid rock drainage, the potential for the physical movement of tailings to the environment and the loss of surface land use values. The specific reclamation objectives for the WTDA reclamation plan are as follows:

1. Isolate potentially acid generating tailings from the atmosphere to minimize the risk of acid rock drainage.
2. Minimize the risk of physical movement of tailings to the environment.
3. Provide a safe and usable surface environment that corresponds to the natural surroundings.

Specific reclamation measures described in the report include:

- Minimization of oxygen exchange in the Surface Cell and Test Cell tailings by placing a cover of shale and sand and gravel over the exposed tailings. The cover will provide thermal insulation, to maintain frozen conditions and allow for permafrost aggradation. The sand and gravel surface layer will provide a durable cap of local material. The cover will be thick enough to maintain continuous frozen conditions within the underlying tailings during mean annual and warmer conditions, even with a worst case estimate of future climate change predictions over the next 100 years.
- Minimization of oxygen exchange in the Reservoir tailings by means of a minimum 1 m water cover. Erosion protection will be placed over the tailings within the shoreline area to minimize the risk of re-suspension of tailings due to wave and ice action. The final water level in the Reservoir and Polishing ponds will be the same as the original, pre-mining elevation of West Twin Lake.
- Transfer of water flow from the Surface Cell to the Reservoir via a new spillway and outlet channel around the south end of West Twin Dike. This structure will be designed to safely pass seasonal run-off and the routed 24-hour probable maximum precipitation (PMP) storm event.
- West Twin Dike will remain in place during closure for permanent retention of the Surface Cell tailings. The outer slope of the dike will be graded smooth to a flatter slope and covered with Twin Lakes sand and gravel to prevent erosion.
- The Test Cell Dike will remain to retain the tailings solids. The crest of the Test Cell Dike will be graded as a portion of the grading plan for the entire cell.
- The water control outlet structure that was used to release water in a controlled manner during mine operations will be removed and replaced with an open outlet channel and overflow weir. The new outlet channel will be located at the natural, original elevation (370.2 m) of the outlet of West Twin Lake. Water passing through the outlet channel will join with water flowing from East Twin Lake and flow into Twin Lakes Creek. This structure will not be removed until after the main work is done to ensure that WTDA water can be managed in the unlikely event that treatment is required.

Design details are presented for each of the areas to be reclaimed. Specific design details for the shale covers on the Surface and Test Cells are provided in the report "Engineering Design of Surface Reclamation Covers" (Water Licence Part G, Item 4).

Discharge water quality from the Polishing Pond is predicted to remain constant over the next 25 years, at about 0.07 mg/L zinc. This is similar to the natural background level of 0.056 mg/L zinc. Over time, the volume of water passing through the Polishing Pond is projected to decrease marginally, due to decreasing rates of porewater expulsion from the tailings.

Construction quality control and quality assurance requirements are included for all aspects of construction associated with the WTDA reclamation. These include:

- Fill placement
- Soil excavation
- Rock excavation
- Drilling and grouting
- Concrete
- Rip rap
- Surveying
- Instrumentation

A comprehensive program for evaluating the long-term performance of the proposed reclamation measures is to be implemented. The monitoring program will also identifies areas where maintenance, repairs or contingencies may be required in the short term. The monitoring information to be collected includes:

- Ground temperatures within the cover, the tailings and the natural ground.
- Subsurface water pressures related to the freezing of the taliks.
- Quality of water entering the environment.
- Climate data.
- Regular inspections of surface conditions by trained technical staff.
- Scheduled inspections of surface conditions by a professional geotechnical engineer.

In general, the monitoring program provides for performance monitoring during the 2 year Reclamation Period and for a subsequent 5 year Closure Period. During the Reclamation Period, worker presence at the mine site is anticipated for construction monitoring and general reclamation activities. This presence will enable the proposed monitoring programs to be carried out by the on-site personnel under the direction of an Environmental Coordinator and geotechnical engineer. During the Closure Period, performance monitoring will be conducted to evaluate the success of the reclamation measures. Continuous worker presence at the mine site is not planned during the closure period and environmental monitoring programs will be carried out during scheduled site visits and possibly utilizing trained, local field assistants and staff hired from nearby Arctic Bay.

CanZinco and other technical experts, as required, will review the information obtained from the monitoring program. CanZinco will take appropriate action to maintain or repair the reclaimed areas. The data obtained from the monitoring program will be forwarded to the Nunavut Water Board for their review and public posting.

Several contingency plans have been developed in order to address performance issues that may be identified during the reclamation and closure monitoring periods. For each issue, the consequences and suggested mitigation approaches are also identified. Common to all suggested mitigation measures is identification of the root cause and appropriate reaction to limit the environmental consequences. The mitigation measures range from performing localized maintenance of the covers to treatment of Reservoir water.

In 2010, a comprehensive, all encompassing assessment of the monitoring information is to be conducted that will determine whether the reclamation objectives have been achieved and whether the WTDA is considered to be successfully reclaimed.

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This report was prepared by BGC Engineering Inc. (BGC) and associated companies (as noted in Section 1.3) for the account of CanZinco Ltd. The material in it reflects the judgement of BGC staff in light of the information available to BGC at the time of report preparation. Any use which a Third Party makes of this report, or any reliance on decisions to be based on it are the responsibility of such Third Parties. BGC Engineering Inc. accepts no responsibility for damages, if any, suffered by any Third Party as a result of decisions made or actions based on this report.

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1.0 INTRODUCTION

1.1 Overview of Development of the Nanisivik Mine Final Closure and Reclamation Plan

The Nanisivik Mine began production of zinc and lead concentrates in 1976. The current owner of the mine, CanZinco Ltd. (CanZinco), has been in possession of the mine since 1996.

Prior to mid-2002, the Nanisivik Mine was scheduled to operate until the depletion of economic ore reserves in 2004 or 2005. However, depressed international base-metal prices necessitated a re-evaluation of the mine production plan in mid-2002. This assessment resulted in a reduction of economic ore reserves such that these reserves were depleted in September 2002. The mine was permanently closed at that time.

An interim mine reclamation plan had been developed and updated on a regular basis by CanZinco in response to terms of the Water License. However, the announcement of permanent closure in October 2001 triggered a requirement in the (then) current water licence for submission of a Final Closure and Reclamation Plan. In response to this trigger, CanZinco submitted a Closure and Reclamation Plan (C&R Plan) in February 2002 that described CanZinco's approach to closure and reclamation of the Nanisivik site.

Subsequent to a Public Hearing on renewal of the water licence held in the community of Arctic Bay in July 2002 and a technical meeting held in August 2002, the Nunavut Water Board ("NWB") issued Water Licence No. NWB1NAN0208, with an expiry date of May 1, 2008 (the "Water Licence"). The Water License provides for the continuation of on-site environmental protection activities during the development and submission, for approval, of a final Reclamation and Closure Plan ("RCP").

The Nanisivik Mine 2004 RCP has been developed, per the terms of the Water License, as a series of stand alone documents, with each document providing, in detail, information and proposed closure measures for one specific component or topic area. The individual reports that have been developed in this manner are listed under Section G of the Water License as summarized in Table 1.

This document and the information presented herein are provided in response to the requirements for component Part G Item 15, the West Twin Disposal Area (WTDA) Closure Plan.

1.2 Specific Requirements for the Reclamation and Closure Plan

The specific requirements for the WTDA Closure Plan come from Part G, Item 15 of the Water Licence. This clause of the Water Licence, as excerpted below, provides the following requirements for the plan:

The Licensee shall submit to the Board for approval a West Twin Disposal Area Closure Plan, which shall include, but not be limited to:

1. *A brief description of historical operating practices, water movement and overall function of the system;*
2. *An updated water balance for the system;*
3. *Current site assessment including characterization of all tailings both for physical properties (gradation, density, mineralogy) and thermal conditions in the surface cell, dike and possibly the reservoir;*
4. *Cover design and description of all construction activities associated with the closure plan, predictions of site stability and water quality with details of analyses that support the plan;*
5. *Contingency plans for dealing with uncertainties and adverse performance during the post-closure monitoring period. These plans need to include a discussion of events that trigger their implementation;*
6. *A monitoring program that includes: permafrost stability, deformations of both the dike and soil cover as well as water quality determinations; and*
7. *An appendix that constitutes a construction plan with material specifications, a quality control plan and as-built drawings stamped by an engineer.*

Additionally, requirements 6 and 9 of Part G, Item 4 (the Covers Report) which apply to the water covered tailings in the Reservoir, are addressed within the context of the closure plan provided herein. Those two requirements are as follows:

6. *The bathymetry of sub-aqueous tailings in West Twin Lake Reservoir which shows the extent of tailings located within 1.0 m of the water surface, and plans for mitigation of wave action on these tailings;*
9. *An evaluation of alternatives for increasing minimum water depth in the Reservoir with emphasis on possible effects of waves and winter ice cover on long term water quality.*

The report provided herein attempts to be comprehensive in its treatment of the elements and technical information and issues with regards to the closure of the WTDA. To provide guidance on the closure elements covered herein the current report, as opposed to other submitted reports, the following list summarizes the breakdown:

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- Erosion protection for East Twin Diversion Dike and Channel – covered in current report.

As noted, the current report attempts to be comprehensive on the topic, without repeating all applicable detail from each of the individual component reports.

1.3 Authorship

This closure plan is multi-disciplinary in content and was prepared with input from three companies including BGC Engineering Inc. (BGC), Gartner Lee Ltd. (GLL) and Golder Associates Ltd. (GAL). Each of the companies provided technical input, report text and related figures and tables for their area of responsibility of the plan. BGC provided overall compilation and production of the report while CanZinco provided overall review of the information submitted herein.

Primary authors for the various companies included the following personnel:

1. BGC – Mr. Jim Cassie, P.Eng., Mr. Geoff Claypool, P.Eng. and Mr. Mike McCrank, E.I.T.
2. GLL – Mr. Eric Denholm, P.Eng. and Mr. Alistair Kent, P.Eng.
3. GAL – Mr. Ken Bocking, P.Eng., Mr. David Ritchie, P.Eng. and Mr. Alex Gordine, P.Eng.

2.0 SITE CONDITIONS

2.1 Location and Topography

Nanisivik Mine is located at the northern end of Baffin Island at approximately 73°N latitude as shown in Figure 1. The mine is approximately 5 km from Strathcona sound, which is connected to the Arctic Ocean through Admiralty Inlet.

The mine area consists of a few intermittent planar areas predominantly surrounded by relatively steep high-relief hills rising out of Strathcona Sound. The surface topography is moderately steep rising from sea level to a high of approximately 650 m immediately west of the mine area (Mt. Fuji).

2.2 Climate

Nanisivik Mine is located on Baffin Island in the Canadian Arctic and is therefore subjected to a harsh climatic environment year round. Atmospheric Environment Services (AES) of Environment Canada maintains a network of climate monitoring stations in Nunavut, including one at the Nanisivik Airport, Resolute, Pond Inlet and Arctic Bay. These stations have systematically collected detailed climatic data for the Canadian Arctic. A detailed review of the available climatic conditions at Nanisivik and proximal stations was conducted to aid in reclamation and closure planning activities. The results of this review are discussed in detail in Section 3.2 of the report “Engineering Design of Surface Reclamation Covers” (Water License Part G, Item 4) and the main parameters are summarized in Table 2.

In addition to a review of all available historical climatic conditions at Nanisivik, an estimate of future climate trends was conducted in order to include potential long-term climate changes in the design of the reclamation works. This estimate was conducted utilizing current guidelines for climate warming estimates. This is also discussed in detail in Section 3.2 of the report “Engineering Design of Surface Reclamation Covers” (Water License Part G, Item 4). The estimated climate warming values are also provided in Table 2.

2.3 Geology

2.3.1 Bedrock

The bedrock geology of the area has been mapped in detail by Patterson and Powis (2002) and Patterson et al. (2003). The regional bedrock stratigraphy is illustrated on Figure 2. The Nanisivik region is underlain by carbonate and terrigenous clastic strata of the Mesoproterozoic Bylot Supergroup. The Bylot Supergroup is comprised of two terrigenous formations (Adam’s Sound and Arctic Bay formations) and two carbonate formations (Society Cliffs and Victor Bay formations) and a mixed carbonate and terrigenous clastic formation (Strathcona Sound Formation). Quartz arenite of the Gallery Formation unconformably overlies the Proterozoic strata.

The Adams Sound Formation is a beige- to light orange-brown, well cemented, medium- to coarse-grained quartz arenite. It is over 100 m thick and is exposed in the Nanisivik Area.

The Arctic Bay Formation is a medium grey to brown, micaceous, fine sandy siltstone interbedded with dark grey micaceous, silty shale. This formation outcrops southeast and southwest of Nanisivik and is approximately 200 m thick.

The Society Cliffs Formation is over 500 m thick and is exposed in the Nanisivik Area. This formation has been subdivided into three units:

- Microbial dolostone (lower);
- Intraclastic dolostone (middle); and,
- Laminated dolomudstone (upper).

All components of the formation exhibit dolomite mineralization. Known sulphide deposits in the Nanisivik area are hosted within the middle and upper subdivisions of this formation.

The Victor Bay Formation is characterized by a gradual upward change from organic rich pyritic shale, to dolomudstone, to more intraclastic and dolomitic facies. It is exposed throughout the Nanisivik area. The formation has been subdivided into three units:

- Shale and dolomitic mudstone unit (lower);
- Dolomitic mudstone and intraclast floatstone (middle); and,
- Silty dolomitic mudstone and intraclast rudstone (upper).

The lower unit is approximately 180 m thick and consists of interbedded, organic rich, fissile shale and light-grey, planar-bedded dolomitic mudstone.

The middle unit is approximately 80 m thick and is marked by the appearance of intraclast rudstone and floatstone. Occasional occurrences of light-grey dolomudstone and black shale similar to those found in the lower unit are observed within the middle unit.

The upper unit is approximately 70 m thick and is characterized by the absence of shale and the prevalence of dolomitic intraclastic carbonate and the presence of terrigenous material within clastic carbonate rocks. This unit forms a transitional unit into the terrigenous Strathcona Sound Formation.

The Strathcona Sound Formation contains two mappable units; a carbonate pebble to boulder conglomerate and an interbedded quartz wacke and shale unit.

The conglomerate unit is a matrix-supported, carbonate pebble to boulder conglomerate that consists of light brown to orange, subrounded to angular 1 to 50 cm diameter dolomudstone clasts in a grey-green, poorly sorted quartz wacke matrix. It thickness ranges from several metres to approximately 50 m.

The interbedded quartz wacke and shale unit is green to dark grey, planar bedded and forms a thick (>130 m), monotonous succession. The quartz wacke and shale are interbedded at scales ranging from 1 to 200 cm. Beds of carbonate pebble to boulder conglomerate are common within 10 m of the contact between the two units.

The Phanerozoic rocks of the Gallery Formation overlie the Mesoproterozoic strata and are exposed predominantly west of Nanisivik. The Gallery Formation forms a thick (>300 m) succession of interbedded red and white, poorly cemented, medium- to fine-grained quartz arenite. The Gallery Formation is easily identified by its deep red hematite-stained colour, and friable texture.

2.3.2 Overburden

Overlying the bedrock, specifically in the area of West Twin Lake, is a combined unit of lakebed sediments (silt and sand, trace to some clay) and glacial till (silty sand with gravel fragments). The lakebed sediments are identified by their red colour, a product of hematite oxidation staining. The till is generally very granular in nature, frozen below the depth of active layer thaw and may contain excess ice content. The thickness of the lakebed sediments/ till unit varies from 1 to 4 m. The lake bed sediments/ till upper surface generally follows the underlying bedrock topography.

2.4 Regional and Local Seismicity

Information provided by the Geological Survey of Canada and referenced in BGC (2000) notes that the 1 in 476 year event is 0.076g and the 1 in 1,000 year event is 0.099g. According to the Canadian Foundation Engineering Manual (1993), the site is situated in the second lowest seismic hazard zone in Canada, Z_a=1 (peak horizontal ground acceleration = 0.04 to 0.08g, 10% exceedance in 50 years). This value is relatively low compared to areas in the highest seismic hazard zone (Z_a=6, >0.32g) in areas such as the southern coast of British Columbia.

2.5 Permafrost

Nanisivik Mine is located in the region of continuous permafrost, as shown in Figure 3. Permafrost has been observed to extend to a depth of at least 430 m below the mine, as observed in a borehole drilled from the underground workings. Ground conditions in the area have been characterized by NRC (1995) as having the potential for medium amounts of ground ice (as high as 20%) and mean annual ground temperatures colder than -10°C. This has been verified by ground temperature measurements at various locations around the mine site as cold as -13°C at depth. The depth of the active layer in natural ground has been observed to average between 1 to 2 m below ground surface.

2.6 Hydrology

A hydrological study of Nanisivik Mine was conducted by Golder (2004b). This study is included in its entirety in Appendix I. The following list contains the principle conclusions of that study:

- The amount of precipitation at the Nanisivik Mine is relatively small. The average annual total precipitation is estimated to be 243 mm. The daily rainfall PMP event is estimated to be approximately 140 mm.
- The average annual lake evaporation at Nanisivik is estimated to be 203 mm; the highest reported monthly lake evaporation of 101 mm occurs in July. The estimated average annual sublimation (evaporation from the snow surface) is approximately 50 mm.
- The watershed area of West Twin Lake (Figure 4) is approximately 300 ha, which includes estimated 127 ha of the Surface Cell drainage basin and 173 ha of the Reservoir drainage area.
- The watershed area of East Twin Lake (Figure 4) is approximately 3460 ha. The lake surface area is approximately 18 ha.
- The frozen ground conditions result in high surface runoff. The estimated average annual runoff coefficient is approximately 0.7. The estimated PMP event runoff coefficient is approximately 0.94.

3.0 WEST TWIN DISPOSAL AREA COMPONENT AND OPERATION REVIEW

3.1 Current Configuration and Operational Practices

The WTDA, illustrated in Figure 5, is comprised of an upper, solids retention pond, the Surface Cell, and a lower, water retention pond, the Reservoir. An earthen dike, the West Twin Dike, separates the Surface Cell and the Reservoir. The Reservoir is further divided by the Test Cell Dike, which separates the Reservoir and the Test Cell. The Test Cell was used to evaluate the performance of several test cover designs. Both dikes are constructed of frozen shale fill and are founded on frozen, settled tailings.

The first shale lift of the West Twin Dike was built in 1991. The initial lift had a top width of approximately 4 m, which brought the elevation of the top of the dike to approximately 374 m. The dike has been raised in an upstream manner where each new lift begins on top of beached tailings material deposited previously and sealing lift of frozen shale rockfill. The dike was raised every year between 1991 and 1999, except 1994. The current crest elevation of the dike is nominally 388 m.

The Test Cell Dike is also constructed of frozen shale fill overlying frozen (and thawed at depth) tailings. The dike was constructed in two stages. The first stage increased the height of the Test Cell Dike to an approximate elevation of 383.5 m. The second stage increased the height of the Test Cell Dike to an approximate elevation of 385.5 m. The second stage of the dike is partially

founded on the first stage dike and partially founded on the tailings in the Test Cell. The staged construction resulted in the formation of a bench on the Reservoir side of the dike.
Excess water from the Surface Cell is transferred to the Reservoir by pumping or a siphon system that controls water levels. The Reservoir and a final Polishing Pond are separated by a causeway and stop log structure, which controls the water level in the polishing pond. Water from the polishing pond is then discharged to Twin Lakes Creek through the West Twin Outlet Structure, a 3 to 5 m high earth fill dam with a valve controlled, concrete lined spillway. Excess water from the WTDA is discharged to the environment via the West Twin Outlet Structure between July and September of each year.

A baffle dike is located immediately upstream of the Polishing Pond inlet, constructed of end-dumped rock fill. The baffle dike was constructed to act as a flow-through structure that would enhance retention and improve water quality. This was constructed prior to 1990 to address marginal water quality issues, but became redundant after the construction of the Surface Cell and was breached. The crest of the baffle is approximately 372 m asl.

3.2 Historical Tailings Deposition Practices

Tailings were initially deposited sub-aqueously into West Twin Lake beginning in 1977. Based on the known mine reserves at the time, West Twin Lake had sufficient tailings storage capacity for the life of the mine. Mine reserves, however, continued to expand each year and by the mid 1980's, it was apparent that continued production would exceed the storage capacity of West Twin Lake. In 1988, an approval was received from the NWT Water Board to begin surface deposition of tailings at Nanisivik. To accommodate this, West Twin Lake was divided into two sections using an earthen dike. The dike was constructed using a north/south trending causeway that was developed by beaching of tailings in the lake as a foundation. The eastern portion of the lake, the Reservoir, retained its original lake level and was used as a stand-by when surface disposal was not practical (i.e. during annual dike construction periods). The western part of the lake, the Surface Cell, became the main deposition area for the tailings and accommodated sub-aerial tailings.

On-land deposition of tailings in the Surface Cell commenced on August 27, 1990. The tailings were mainly discharged from the north-west corner of the Surface Cell. The full capacity level of the Surface Cell was raised to an elevation of 374 m with the construction of the initial lift of the West Twin Dike in 1991. Tailings discharge in the Surface Cell continued throughout the 1990's with annual (except for 1994) raises of the West Twin Dike in 2 m increments. Tailings continued to be discharged mainly from the north-west corner of the Surface Cell with a deeper section of the pond in the southeast corner for water reclamation and mineral processing purposes.

Tailings discharge practices were amended during the mid-1990's to promote stratification of sediments by discharging from the perimeter of the Surface Cell. Excess supernatant pond water continued to be collected in a depression situated upstream from the south end of the dike. This water was siphoned to the Reservoir. The ultimate raise of the dike, to a nominal elevation of 388 m, was completed in 1999 and tailings continued to be placed in the Surface Cell until the mine closed in September 2002.

Figure 6 illustrates how the tailings level in the Surface Cell evolved over time, with the original lake bottom elevations shown, based on bathymetry maps produced by the mine. As can be seen, an east/west causeway had become fully exposed in the Surface Cell by 1982. Although not shown on the section in Figure 6, a north/south tailings causeway had also developed within the lake at the same time. This north/south causeway was eventually used as the foundation for the West Twin Dike. Tailings continued to be deposited sub-aqueously into the Surface Cell until 1991, at which the first lift of the West Twin Dike was constructed. Construction of the initial lift and subsequent annual raises of the West Twin Dike allowed for sub-aerial and sub-aqueous deposition of tailings to continue throughout the 1990's. Tailings were generally discharged from an on-land deposition point near the north-west corner of the Surface Cell. Tailings discharge was also conducted along the upstream face of the West Twin Dike sealing the dike face and providing the foundation for the next raise of the dike. Surface water was generally directed to the south-east portion of the Surface Cell where a depression was located, referred to in the remainder of the report as the "deep water storage pond". In total, it is estimated that 6.5 million m^3 of tailings were deposited into the Surface Cell between 1978 and 2002.

Figure 7 illustrates the evolution of the tailings level in the Reservoir and Test Cell over time. Tailings were deposited sub-aqueously into the Reservoir beginning in 1976. The discharge of tailings generally took place near the current location of the West Twin Dike resulting in the aggradation of the tailings deposit in a south-easterly direction into the Reservoir. Tailings deposition along an east-west trending line from the centre of the West Twin Dike resulted in exposure of a tailings causeway in the Reservoir by 1988. This causeway eventually became the foundation for the east/west arm of the Test Cell Dike. Tailings deposition along a northwest-southeast trending line resulted in a second tailings causeway near the reclaim water pumphouse, which was exposed by 1992. This causeway eventually became the foundation for the north/south arm of the Test Cell Dike. The majority of the tailings discharge activities in the 1990's occurred in the Surface Cell. The Test Cell Dike was constructed in 2000 and 2001, increasing the storage capacity of the Test Cell, thereby allowing for the additional placement of tailings into the Test Cell. Deposition of tailings at the toe of the West Twin Dike in 2000 resulted in aerial exposure of tailings in the Reservoir and Test Cell. In total, it is estimated that 3.5 million m^3 of tailings have been deposited into the Reservoir and Test Cell since 1978.

4.0 CHARACTERIZATION OF WEST TWIN DISPOSAL AREA TAILINGS

The geotechnical, geothermal and geochemical properties of the WTDA tailings have been assessed during several studies completed for Nanisivik Mine throughout the operational history of the WTDA. The following sections summarize the results of those studies.

4.1 Geotechnical Characterization

4.1.1 Geotechnical Investigations

Several geotechnical investigations of the WTDA have been conducted over the operational history of the mine. Approximately 50 geothermal monitoring instruments were installed during investigations prior to 2002. In 2002 and 2003, a comprehensive investigation of the WTDA tailings was undertaken by BGC in which 36 geothermal monitoring instruments were installed. The following is a chronological history of the significant geotechnical investigations conducted at the WTDA:

- 1989/1990 – Geotechnical investigations of the causeways that had developed in West Twin Lake to assess permafrost aggradation. Thermocouples were installed in five boreholes.
- 1992 – Geotechnical investigations of WTDA including drilling and installation of thermocouples and frost gauges from the 376 m bench of the West Twin Dike and each of the test covers in the Test Cell. Geotechnical drilling and installation of thermocouples were also completed at various other locations around the Nanisivik Mine site.
- 1994/1995 – Additional geotechnical drilling and thermocouple installation on the 376 m bench of the West Twin Dike.
- 1997 – Geotechnical drilling and installation of thermocouples at various locations of the West Twin Dike and at the toe of the dike in the Reservoir.
- 1998 – Geotechnical drilling in the delta fan between ETL and WTL to assess the hydraulic connectivity between the two lakes.
- 1999 – Geotechnical drilling at various locations on the West Twin Dike.
- 2001 – Geotechnical drilling and installation of thermocouples within the Surface Cell and Test Cell Dike.
- 2002 and 2003 – Geotechnical drilling and installation of geothermal and piezometric monitoring instruments at the WTDA.

Although the results of the investigations prior to 2002 were considered in the overall characterization of the WTDA tailings deposit, the following sections focus on the results of the investigations undertaken in 2002 and 2003.

4.1.2 Ground Conditions

In 2002 and 2003, a total of 44 boreholes were drilled into the tailings at the Surface Cell, Test Cell Dike and at the toe of the West Twin Dike. Limited access due to water cover prevented the drilling of boreholes in the Test Cell area. The approximate location of each borehole is illustrated on Figure 8 and a summary of the borehole information is provided in Table 3. The drilling was completed utilizing the Atlas Copco 262 diamond drill available at the mine site. Drilling was conducted utilizing chilled brine and samples were recovered in a double tube core barrel. Instrumentation installed in the boreholes includes thermistors, thermocouples, vibrating wire piezometers and standpipe piezometers. For borehole logs and further discussion regarding the drilling method, types of instrumentation installed and installation procedures, refer to the Taliks Assessment Report (Water License Part G, Item 5).

4.1.2.1 Surface Cell

Soil types encountered during drilling in the Surface Cell included shale fill, frozen and thawed tailings, lake bed sediments, till and shale and dolostone bedrock. Typically, frozen tailings were well bonded (N_{bn}) or friable with no visible ice (N_f), but occasional zones with visible ice contents up to 50% were observed. Occurrences of ground ice were encountered within the tailings in boreholes BGC03-07, 03-11, 03-12, 03-20, 03-21, 03-31, 03-38 and 03-39. Artesian pore pressures were observed in several boreholes, including the monitoring wells installed in boreholes BGC03-12 and 03-14. The ground conditions observed in each borehole are discussed in the remainder of this section.

The geotechnical drilling completed in the Surface Cell in 2002 and 2003 can be divided into four distinct areas, as shown on Figure 8:

- **Area 1** – located near the historical location of the deep water storage pond on the Surface Cell.
- **Area 2** – located immediately upstream of the West Twin Dike between the dike and the deep water storage pond.
- **Area 3** – located north of Areas 1 and 2 between the West Twin Dike and the retained pond.
- **Area 4** – located along the west side of the Surface Cell between the two retained ponds.

The following sections describe the ground conditions encountered during the geotechnical drilling in each of the four areas.

Area 1

Area 1 was typically used as the historical location of the deep water storage pond. A total of ten boreholes (BGC02-01, 02-02, 02-03, 02-04, 02–05, 02-07, 02-12, 03-11, 03-35 and 03-36) were drilled in this area. Prior to discussing the results of the drilling conducted in this area, it is important to review the following facts regarding the history of the tailings deposition at this location:

- The storage pond was filled with tailings in the summer of 2002 prior to the geotechnical drilling conducted in September 2002.
- Prior to 2002, tailings in this area generally had a water cover of at least 6 m.
- The southern portion of this area is located near the original shoreline of West Twin Lake.
- The original bathymetry indicated that the water depth increased to the northern portion of this area. Therefore, a thicker sequence of tailings would be expected in the northern portion of this area.

The following is a list of the principal observations made during the drilling completed in this area:

- In September 2002, the ground conditions in this area were generally thawed from surface to a depth ranging between 10.5 and 24.7 m, increasing in thickness to the north portion of the area.
- The boreholes drilled in September 2003 (03-11, 03-35 and 03-36) encountered frozen tailings from surface to a depth of between 4.5 and 4.9 m overlying thawed tailings. This indicates permafrost aggradation had occurred into the tailings.
- Boreholes 02-01, 02-02 and 02-03 encountered 1.5 to 4.0 m of frozen tailings overlying the bedrock or lake bed sediments at the bottom of each borehole. This indicates that some cooling of the tailings from underlying bedrock has occurred at this location. This is likely related to the fact the bedrock in this area was above the original shoreline of the lake.
- Borehole BGC03-11 encountered ice between 4.3 and 6.1 m.
- Flowing sands (lake bed sediments) were encountered in borehole BGC02-07 between a depth of 24.4 and 24.7 m. This indicates high pore pressures at depth.

Thermistors were installed in boreholes BGC02-01, 02-02, 02-03 and 03-11. The thermistors in boreholes BGC02-01 and 02-02 were damaged in late 2002 and are no longer operational. A thermocouple was installed in boreholes BGC02-12 and 03-36. Standpipe piezometers were installed in boreholes BGC02-04 and 02-05 and a vibrating wire piezometer was installed in borehole BGC03-35.

The flowing sands encountered in borehole BGC02-07 prevented instrumentation from being installed in the borehole. Borehole BGC02-07 was attempted two additional times in areas within 5 m of the original 02–07 location. Each attempt had similar results with no instrumentation being installed.

Area 2

Area 2 is located between the historical location of the deep water storage pond and the West Twin Dike. A total of eight boreholes (BGC02-06, 02-11, 03-01, 03-02, 03-12, 03-13, 03-14, 03-15, 03-33, 03-34 and TC #35) were drilled in this area. Prior to discussing the results of the drilling conducted in this area, it is important to review the following facts regarding the history of the tailings deposition at this location:

- Prior to the construction of the West Twin Dike, tailings deposited in this area had a shallow water cover.
- After construction of the West Twin Dike, tailings deposited in this area generally had a shallow water cover or were aerially exposed due to its proximity to the West Twin Dike.

The following is a list of the principal observations made during the drilling completed in this area:

- Boreholes BGC02-06, 02-11, 03-01, 03-02, 03-13 and 03-14 encountered 8 to 11 m of frozen tailings overlying thawed tailings.
- Artesian pore pressures were observed in the thawed tailings encountered in boreholes 03-01 and 03-02.
- Borehole BGC03-15, the borehole closest to the dike, encountered frozen ground throughout most of borehole profile. A small zone between 13.4 and 14 m exhibited artesian pore pressures and thawed ground conditions.
- BGC03-34 was drilled to a depth of 13.7 m and was terminated in frozen tailings.
- BGC03-33 was drilled to a depth of 22.9 m and encountered thawed tailings at a depth of 18.3 m and lake bed sediments at a depth of 22.4 m.

Thermistors were installed in boreholes BGC02-06, 03-13, 03-15, 03-33 and 03-34. Thermocouples were installed in boreholes BGC02-11 and TC 35. The thermistor installed in borehole BGC02-06 was damaged in late 2002 and is no longer operational. A standpipe piezometer with heat trace tape was installed in borehole BGC03-14. A vibrating wire piezometer was installed in BGC03-12 and 03-14 during a subsequent investigation in September 2003 to provide piezometric monitoring capabilities during winter months. The artesian pore pressures encountered in boreholes BGC03-01 and 03-02, coupled with a malfunctioning water pump on the drill rig, resulted in difficult drilling and prevented installation of any instrumentation in these boreholes. Considering the proximity of thermistors BGC03-33 and 03-34 to each other, monitoring results from these instruments are thought to reflect a continuous geothermal profile between the ground surface and 22.9 m depth.

Area 3

Area 3 is located north of the historical location of the deep water storage pond, between the West Twin Dike and the retained pond. A total of ten boreholes (02-13, 03-07, 03-08, 03-09, 03-10, 03-31, 03-32 and 03-37) were drilled in this area. This area was generally covered in shallow water cover or aerially exposed during tailings deposition after the construction of the West Twin Dike. This is due to its proximity to the dike and the on-land tailings discharge point in the northwest corner of the Surface Cell.

The following is a list of the principal observations made during the drilling completed in this area:

- Boreholes BGC02-13, 03-09 and 03-10 encountered frozen tailings from surface to a depth of approximately 11 m.
- Thawed tailings were generally encountered below 11 m.
- Occasional occurrences of artesian pore pressures in the thawed tailings below 11 m were observed.
- Boreholes BGC03-31 and 03-32 encountered frozen tailings and ice to a slightly deeper depth, between 13.7 and 17.7 m and thawed tailings below. The boreholes were terminated in thawed tailings.
- Borehole BGC03-07 was drilled near the north end of the dike and encountered frozen tailings and ice lenses. A zone of tailings between 19.5 and 22.5 m was inferred to exhibit artesian pore pressures and thawed conditions based on response of the drilling equipment while advancing the borehole.
- Borehole BGC03-08 encountered 6.4 m of frozen tailings before encountering a piece of metal debris, at which point a decision was made to terminate the borehole.
- Borehole BGC03-37 was drilled into the shale pad located near the north abutment of the West Twin Dike to a depth of approximately 7.3 m. The borehole encountered approximately 2.7 m of shale fill overlying frozen tailings to a depth of 7.3 m.

Thermistors were installed in boreholes BGC03-07, 03-09, 03-10 and 03-37 and a thermocouple was installed in borehole BGC02-13. Vibrating wire piezometers have been installed in BGC03-31 and 03-32 to provide piezometric monitoring capabilities during winter months.

Area 4

Area 4 is located on the west side of the Surface Cell between the two retained ponds. A total of five boreholes (BGC03-20, 03-21, 03-38, 03-39 and BH 10 [TC #37]) were drilled in this area. Prior to discussing the results of the drilling conducted in this area, the following facts are important:

- Tailings deposited in the northern portion of this area after construction of the West Twin Dike were generally covered in shallow water cover or aerially exposed. This is due to

the proximity of this area to the on-land tailings discharge point in the northwest corner of the West Twin Dike.
- Tailings deposited in the southern portion of this area after construction of the West Twin Dike generally had a water cover.
- The north and south extremes of this area are located over the original shoreline of West Twin Lake.

The following is a list of the principal observations made during the drilling completed in this area:
- Borehole BGC03-20, located about 175 m west of the former water storage pond, encountered approximately 11.9 m of frozen tailings and ice layers from surface.
- Below the frozen tailings, thawed tailings were encountered to a depth of approximately 19 m. Artesian pore pressures were encountered in this borehole at a depth of approximately 14 m.
- Borehole BGC03-21 encountered frozen tailings and ice layers from surface to a depth of approximately 16 m at which point the shale surfacing layer of the original east/west causeway was encountered.
- Borehole BGC03-38 encountered frozen tailings and ice from surface to a depth of approximately 13.7 m where frozen till was encountered.
- Borehole BGC03-39 encountered approximately 1 m of shale at surface overlying frozen tailings and ice to a depth of 10.6 m where till was encountered.
- Borehole BH-10 (TC37) encountered tailings to a depth of 30 m. No core was recovered between 9 and 11 m and 24 and 27 m.

Thermistors were installed in boreholes BGC03-20 and 03-21 and thermocouples were installed in BGC03-38, 03-39 and BH-10 (TC37).

4.1.2.2 Toe of West Twin Dike

As discussed in Section 3.0, a tailings beach was formed at the toe of the West Twin Dike in the Test Cell and Reservoir during tailings deposition in 2001 and 2002. A total of four boreholes (BGC02-08, 02-09, 03-18 and 03-19) were drilled in this area.

Each borehole drilled at the toe of the West Twin Dike encountered frozen conditions throughout the entire depth of the borehole. A frozen layer of shale was encountered overlying frozen tailings and lake bed sediments. Frost shattered bedrock was encountered in two of the boreholes at a depth of approximately 7 m. Visible ice lenses were observed in the tailings in some of the boreholes.

4.1.2.3 Test Cell Dike

As discussed in Section 3.0, a shale dike was constructed into the Reservoir in 2001 and 2002. The dike was constructed using two tailings causeways that had previously formed in the Reservoir. A total of three boreholes (BGC02-09 and 03-22) were drilled in this area.

The boreholes encountered frozen shale (dike construction material) overlying frozen tailings overlying thawed tailings. Lake bed sediments were encountered only in borehole BGC02-09. No visible ice was observed in the frozen tailings samples.

4.1.3 Lab Testing

Various samples of tailings collected from the boreholes were selected for laboratory testing. Grain size, moisture content and bulk density testing were performed at the mine laboratory. Additional samples were then sent to Almor Testing Services Ltd. and EBA Engineering Consultants Ltd. for further testing. The test results were used to confirm the field classification of the soils. The laboratory testing results are summarized in Table 4.

4.1.3.1 Grain Size Distribution

The grain size distribution of 120 samples of tailings from the WTDA was determined and the results are illustrated on Figure 9. Ten tailings samples were tested twice, with and without hydrometer testing, resulting in the 130 grain size distributions illustrated on Figure 9. The test results indicate the composition of the tailings varies between primarily sand (98.9%) and primarily silt (95.7%). Clay size particles were observed in the tailings samples to a maximum of 3.0%. The range in observed particle size is likely related to segregation of particles as they settle out on the tailings beach and also under water.

4.1.3.2 Specific Gravity

The specific gravity (G_s) of 9 samples of tailings from the Surface Cell was derived according to ASTM Standard D422. Using this method, the specific gravity was observed to range between 3.89 and 4.46. The average specific gravity was calculated to be 4.08. This is lower than the previous estimate of the specific gravity (G_s = 4.6) of the tailings by Golder (1999) which was based on the specific gravity of the individual elements within the tailings.

4.1.3.3 Frozen Bulk Density

The frozen bulk density of 33 samples of tailings from the WTDA was calculated and the results are illustrated on Figure 10. The frozen bulk density was observed to range between 2128 and 3526 kg/m^3 (average 2800 kg/m^3). The lowest frozen bulk densities correspond to the samples with the highest amount of ice content.

4.1.3.4 Moisture Content and Saturation

The moisture content of 114 samples of tailings from the WTDA was calculated and the results are illustrated on Figure 11. This figure indicates that the moisture content ranges between 4.9 and 70%. The average moisture content of the samples not containing visible ice, was calculated to be 16.5%. The relationship between moisture content and grain size distribution is illustrated on Figure 12. This figure indicates that the moisture content increases with increasing fines content.

In order to evaluate the degree of saturation of the tailings, the specific gravity values derived from the grain size testing and the calculated frozen bulk densities were used to approximate the void ratio. Using the average moisture content of 16.5%, the average frozen bulk density of 2800 kg/m^3 and the maximum and minimum specific gravities, 3.89 and 4.46, the void ratio was calculated to range between 0.6 and 0.9. This is similar to the range of 0.6 to 1.0 noted for lead-zinc tailings by Vick (1983). The range of void ratios and specific gravities was then used to determine the water content required for saturation which was calculated to range between 16 and 19%. Considering that 58% of the moisture content values were greater that 16%, it is apparent that the majority of the tailings in the Surface Cell and Test Cell are saturated.

4.1.3.5 Thermal Conductivity

A composite sample of tailings from the Surface Cell was sent to EBA Engineering Consultants Ltd. in Edmonton, AB for thermal properties testing. The scope of work included the following tasks:

1. Determine the thermal conductivity in both frozen and unfrozen states;
2. Determine the relationship between unfrozen water content and freezing temperature; and,
3. Conduct index tests to measure the frozen bulk density, water content, particle size distribution and specific gravity of the composite sample.

The thermal conductivity was measured using the thermal needle probe method in accordance with the ASTM test procedure D5334-92. The unfrozen water content curve was determined using the Time Domain Reflectometry technique. The sample was blended and the resulting grain size distribution was observed to consist of 57% silt, 42% sand and 1% clay sized particles. The sample was moisture conditioned to a gravimetric water content of 15.5%. The results of the testing indicate that the thermal conductivity of the tailings is approximately 1.9 W/(m·°C) at room temperature and 3.2 W/(m·°C) at -15°C. The volumetric unfrozen water content of the sample ranged between 37.2% at 0°C to 4.9% at -11.9°C. The frozen bulk density was measured to be 2730 kg/m^3 and the specific gravity was measured to be 3.92.

4.1.4 Water Quality Testing

In order to obtain groundwater samples from the talik, four monitoring wells were installed during the geotechnical investigation of the Surface Cell in 2002 and 2003. Boreholes BGC02-04, 02-05, 03-12 and 03-14 were drilled within the tailings and a standpipe piezometer, screened in the talik, was installed in each of the boreholes. Wells BGC02-04 and 02-05 were developed using a Waterra pump system and a 1.9 cm diameter disposable bailer. Wells 03-12 and 03-14 displayed artesian conditions during drilling and were allowed to develop while free flowing.

Prior to discussing the results of the sampling program, it should be noted that the drilling procedure required the use of chilled brine as a drilling fluid. The chilled brine provided a means of retrieving frozen core samples where possible, and helped maintain a stable borehole within the frozen zone. This approach was commonly used at the mine site while operating. The preparation of brine for the 2003 drill program generally followed these procedures. For the 2003 drill program, the brine was prepared by mixing water collected from the town water supply (i.e. East Twin Lake) with calcium chloride in an open tank located near the Industrial Complex. The use of this brine likely had a significant impact on the water quality results obtained, as discussed in later sections.

4.1.4.1 Field Parameters

Monitoring wells BGC02-04 and 02-05 were sampled soon after installation in September 2002. Monitoring wells BGC03-12 and 03-14 have been sampled twice since installation, once in May 2003 and once in August 2003. Conductivity and pH values were obtained from the samples collected in September 2002 and August 2003 using the pH/conductivity meter in the mine laboratory. The data obtained from the pH and conductivity testing is summarised in Table 5. The data collected in September 2002 is not considered representative of groundwater quality due to presence of chilled brine in the sample water. The data collected in August 2003 indicates that the pore water is basic and slightly conductive. Table 5 also contains the pH and conductivity values obtained from samples of the seepage water discharged from the dike in 1999 and the water samples collected from monitoring station 159-2 in 2001. Seepage water was collected from the dike face from two sources in 1999, one near TC #13 and one near TC #18. Water quality monitoring station 159-2 is located at the Reclaim Pump House in the Reservoir. The measured conductivities from all sources were observed to be similar. The measured pH levels of the pore water samples collected in 2003 were observed to be higher than the pH of the seepage water collected in 1999, but within the range of the pH levels recorded from samples collected at station 159-2 in 2001.

4.1.4.2 Water Quality Results

In May 2003, water samples collected from wells BGC03-12 and 03-14 and ice core samples recovered from boreholes BGC03-06, 03-20 and 03-21 were sent to Accutest Laboratories Ltd. in Ottawa, ON for analytical testing. Additional water samples collected from wells BGC03-12 and 03-14 in August 2003 were sent to Maxxam Analytics Inc. in Mississauga, ON for analytical testing. The lab data sheets are included in Appendix II.

The following observations can be drawn from the groundwater quality results:

1. The initial (May 2003) samples from boreholes BGC03-12 and BGC03-14 contained elevated chloride concentration as compared to the August 2003 samples (2820/10300 mg/L in May versus 413/1380 mg/L in August).
2. There is a general decrease in concentrations of many (total) parameters from May to August 2003 including SO_4,, Ca, Cu, Fe, Pb and Zn (Zn in BGC03-12 only).
3. Many parameter concentrations were substantially less in dissolved form than in total form in August 2003 including Cu, Fe, Pb and Zn.
4. Many parameter concentrations were substantially greater in ice than in water in May 2003 including Cu, Fe, Pb and Zn; SO_4 did not follow this trend and was substantially less in ice than in water.

The concentrations of several key indicators in water and ice for all sample results from May and August 2003 are summarized in Table 6.

4.1.4.3 Summary of Water Quality Testing

In general, the initial sample results from any newly installed groundwater well should be evaluated in the context of possible residual effects of the drilling procedures, even in light of the purging of substantial well volumes during well development. In this case, it appears that the May 2003 samples were affected by the use of chilled brine in the drilling procedure as evidenced primarily by substantially elevated concentrations of chloride and calcium and also by elevated concentrations of some metals. The procedures for preparation of the brine would allow for the introduction not only of chloride and calcium but also metals (via possible contamination of the open agitation and transport tanks) into the groundwater. Therefore, the May 2003 results for boreholes BGC03-12 and 03-14 should be viewed as not representative of groundwater quality because of residual effects of the drilling procedure.

Concentrations of dissolved metals are typically lower than total metals in groundwater samples. This is because of the downhole agitation that typically occurs during purging and sampling of groundwater wells, which can mobilize suspended solids from the annulus around the screened well intake or within the bottom of the well. Therefore, the August 2003 results for boreholes BGC03-12 and 03-14 are as expected with regard to the difference between dissolved and total metal concentrations.

The generally higher metal concentrations observed in ice as compared to water could be suggestive of the effects of an unavoidable minor amount of tailings that were entrained into the ice sampled and, thereby, incorporated into the water sample. Therefore, the May 2003 results for ice should be viewed as not representative of groundwater or porewater quality because of the entrained tailings in the sample.

The groundwater quality data collected to date from the 2003 boreholes is preliminary and will be supplemented over time with additional sampling results. Nonetheless, several overall preliminary conclusions can be drawn from the data that is available as follows:

1. Thawed groundwater is present within the tailings under artesian conditions in some locations.
2. Groundwater quality is neutral to alkaline due, presumably, to the residual effects of lime deposited with the tailings (i.e., tailings slurry did typically have pH of approximately 10 as a result of the need for alkaline pH for flotation of mineral concentrates).
3. Tailings groundwater quality shows no indications of the occurrence of acid rock drainage in the tailings (i.e., dissolved metal concentrations are generally low), which is attributed to the alkaline pH and rapid freezing and/or saturation of the tailings upon deposition.

4.1.5 Results of Piezometric Monitoring

The monitoring wells and vibrating wire piezometers installed in 2003, BGC03-12, 03-14, 03-31, 03-32 and 03-35 have been monitored to determine static water levels several times since installation. Graphical plots illustrating the results of the static water level monitoring have been included in Appendix III. All piezometric monitoring instruments, with the exception of BGC03-35, indicate artesian piezometric conditions at depth. The pore pressures recorded from instruments 03-12, 03-12, 03-31 and 03-32 indicate a piezometric elevation approximately 0.5 to 2.0 m above ground surface. The pore pressures recorded from instrument 03-35, located approximately 175 m upstream of the crest of the dike, indicates a piezometric elevation of approximately 1.5 m below ground surface. Monitoring results recorded since May 2003 indicate that the piezometric levels fluctuate with time. Levels were initially highest during the spring. Reduced levels were recorded during summer months and increased levels have been observed during the winter months.

Recovery tests were performed in monitoring wells BGC03-12 and 03-14 in May, 2003. The tests measured the change in water level with time, following the removal of the heat trace wire. Graphical plots illustrating the results of the recovery tests have been included in Appendix III. Based on these tests, the calculated hydraulic conductivity of the tailings is in the range of $1x10^{-6}$ to $1x10^{-7}$ m/s.

4.1.6 Freezing Point Depression

The conductivity of the water samples was used to estimate the freezing point depression of the pore water within the tailings. This was done by using the conductivity values to estimate the amount of soluble salts present in the water sample. Using a conductivity of 6230 μS, and the approximate correlation of 1 part per thousand (ppt) salinity equating to approximately 1560 μS conductivity, the salinity was calculated to be approximately 4 ppt. The freezing point depression was determined to be approximately 0.2°C using the relationship between salinity and freezing point depression developed by Ono (1966).

Freezing point depression of talik pore water was further assessed by comparing ground conditions observed during drilling with measured ground temperatures from thermistors installed within the Surface Cell talik. This approach has been mentioned in McKay (1998). This method indicates that the freezing point depression of the talik pore water may be as much as -1.2°C. The remainder of this section discusses the information supporting this assessment of freezing point depression.

The supporting information is best examined by considering firstly the information collected proximal to the West Twin Dike and secondly, the information collected proximal to the Surface Cell pond.

Proximal to the West Twin Dike

The information from this area is derived from thermistor BGC03-13 and 03-09 installed into the Surface Cell talik and the thermistors incorporated into the vibrating wire piezometers at BGC03-12, 03-14, 03-31 and 03-32.

- The vibrating wire piezometers within the Surface Cell talik contain an internal thermistor to measure the temperature at the tip. Four vibrating wire piezometers are installed at locations proximal to the West Twin Dike (BGC03-12, 03-14, 03-31 and 03-32). These instruments indicate water temperatures of approximately -0.4° to -0.5°C in the areas where thawed conditions were observed in the Surface Cell talik.
- A thawed zone was encountered in borehole BGC03-13 between 11 and 14 m. Subsequent geothermal monitoring data collected from a thermistor installed in this zone indicates a ground temperature of approximately -0.4°C.
- A thawed zone was encountered in borehole BGC03-09 between 11 and 27 m. Subsequent geothermal monitoring data collected from a thermistor installed in this zone indicates ground temperatures between -0.1° and -0.2°C.

Proximal to the Retained Pond

- The vibrating wire piezometer installed in the Surface Cell talik proximal to the retained pond (BGC03-35) indicates water temperatures of approximately -1.2°C at the tip where thawed conditions had been encountered during drilling.
- A thawed zone and artesian pressures were encountered in borehole BGC03-11 between 11 and 18 m below ground surface. Subsequent geothermal monitoring data collected from thermistors installed in this zone indicates ground temperatures between -0.2° and -0.9°C.
- A thawed zone and artesian pressures were encountered in borehole BGC03-09 between 11 and 27 m below ground surface. Subsequent geothermal monitoring data collected from thermistors installed in this zone indicates ground temperatures between -0.1° and -0.7°C.

This information would suggest that the amount of freezing point depression increases with distance away from the West Twin Dike. Alternatively, the magnitude of freezing point depression increases in magnitude with distance from the freezing front. This is likely related to pore water expulsion, solute rejection and cryoconcentration at the freezing front.

4.2 Geothermal Characterization

Regular monitoring of the geothermal instruments, both pre- and post-2002/2003 geotechnical investigations, has been ongoing. The results of the monitoring are illustrated graphically on the plots included in Appendix IV. The following sections review the results of the monitoring data collected from geothermal instruments located in the Surface Cell, within the West Twin Dike, at the toe of the West Twin Dike and within the Test Cell Dike.

4.2.1 Surface Cell

The data collected from the geothermal monitoring instruments verifies the observations made during the 2002 and 2003 field investigations that many areas of the Surface Cell contain thawed tailings. As discussed in Section 4.1.2.1, the Surface Cell boreholes and instruments can be divided into four main areas. The following sections review the geothermal information collected from each area.

Area 1

Area 1 is located approximately where the historical location of the former Surface Cell deep water storage pond was located. The depression was infilled with tailings in the summer of 2002. The geothermal monitoring instruments located in Area 1 include:

- thermistors BGC02-01, 02-02, 02-03, 03-11; and,
- thermocouples 02-12 and 03-36.

In general, the geothermal monitoring data indicates that the subsurface has cooled since September 2002. Permafrost has aggraded into the top 4 to 5 m of the tailings and the tailings below this depth have generally cooled to within +/-0.2° of 0°C.

Area 2

Area 2 is located proximal to the West Twin Dike between the crest of the dike and the historical area of the storage pond. The geothermal monitoring instruments located in Area 2 include:

- thermistors BGC02-06, 03-13, 03-15, 03-33, 03-34; and ,
- thermocouples BGC02-11 and TC35.

In general, the geothermal monitoring data indicates that permafrost has aggraded into the top 8 to 18 m of the tailings. The tailings below this depth have generally cooled to within +/-0.2° of 0°C. The thickness of permafrost appears to increase towards the West Twin Dike. For example, thermistor BGC03-13, located approximately 50 m upstream of the crest of the West Twin Dike, indicates permafrost to approximately 11 m below surface while thermistor BGC03-33, located at the downstream crest of the West Twin Dike, indicates permafrost to a depth of 18 m.

Area 3

Area 3 is located north of Areas 1 and 2 between the West Twin Dike and the retained pond in the Surface Cell. The geothermal monitoring instruments located in Area 3 include:

- thermistors BGC03-09, 03-10, 03-07, 03-37; and,
- thermocouples BGC02-13, TC #38, TC #39, TC #40 and TC #41.

Thermocouples TC #38 through TC #41 are shallow (within 3 m of surface) and were installed by the mine to monitor the performance of a shale test pad located near the north abutment of the dike. Since the data has no implications relating to talik characterization, the data from these instruments is not discussed in this report.

In general, the geothermal monitoring data indicates that permafrost has aggraded into the top 8 to 18 m of the tailings. The tailings below this depth have generally cooled to within +/-0.2° of 0°C. The thickness of permafrost appears to increase towards the West Twin Dike. For example, thermistor BGC03-10, located approximately 75 m upstream of the crest of the West Twin Dike, indicates permafrost to approximately 6 m below surface while thermistor BGC03-09, located approximately 75 m upstream of the crest of the West Twin Dike, indicates permafrost to a depth of 11 m.

Area 4

Area 4 is near the west side of the Surface Cell in between the two remnant retained ponds. The instruments within Area 4 are approximately 250 m upstream of the crest of the West Twin Dike. The geothermal monitoring instruments located in Area 4 include:

- thermistors BGC03-20 and 03-21; and,
- thermocouples BGC03-38, 03-39 and TC #37.

In general, the geothermal monitoring data indicates that permafrost has aggraded into the top 11 to 16 m of the tailings.

4.2.2 West Twin Dike

As discussed in Section 3.0, the West Twin Dike was constructed in stages throughout the 1990's to increase tailings storage capacity in the Surface Cell. The dike has been instrumented with thermocouples and frost gauges at various times and locations since it was constructed. Currently, the operational instruments in the dike include:

- thermocouples – TC #1, TC #2, TC #12, TC #13, TC #14, TC #15, TC #16, TC #18, TC #28, TC #29, TC #30, TC #31, TC #32, TC #33, and TC #34.

The monitoring data collected from the West Twin Dike in 2003 indicate that, for the most part, the dike and the foundation of the dike are frozen to an elevation of at least 359 m (see data from TC #16A). A potentially thawed zone has been indicated by the data from thermocouple TC #33. The data from thermocouple TC #33 indicates that ground temperatures approach 0°C at a depth of 16 m. The tailings deposit extends to 19 m at this location; however, the deepest thermocouple node is located at 16 m depth.

Readings collected at areas where thaw was previously observed in 1999 at TC #13 and TC #18, indicate that the dike and foundation are currently frozen at these locations.

4.2.3 Toe of West Twin Dike

As discussed in Section 3.0, a tailings beach was formed at the toe of the West Twin Dike in the Test Cell and Reservoir during tailings deposition on 2001 and 2002. The instruments located at the toe of West Twin Dike include:

- thermistor – BGC03-19 and
- thermocouples – BGC02-08, 02-10 and 03-18.

In general, the geothermal monitoring data indicates that permafrost has aggraded into the entire thickness of tailings which ranges between 4 and 9 m. Ground temperatures at depth in permafrost have been measured to fluctuate around -7°C.

4.2.4 Test Cell Dike

As discussed in Section 3.0, a shale dike was constructed into the Reservoir in 2001 and 2002. The dike was constructed using two tailings causeways that had previously formed in the Reservoir. The instruments located in the Test Cell Dike include:

- thermistors – BGC02-09, 3-22, and;
- thermocouples – TC 36.

The monitoring data verifies the observations made during the 2002 and 2003 field investigations that permafrost has aggraded into the tailings beneath the Test Cell Dike to a depth of between 14 and 18 m. The monitoring data indicates that the tailings beneath this depth exhibit temperatures near 0°C. It should be noted that the tailings below 14 to 18 m depth were interpreted to be thawed during drilling.

4.3 Surface Cell and Test Cell Talik Characterization

The results of the geotechnical investigation programs completed in 2002 and 2003 and the geothermal monitoring, during the same time, indicate that taliks exist within the tailings in the Surface Cell and the Test Cell. This information, along with the historical tailings deposition practices within the WTDA previously discussed in Section 3.2, was reviewed and interpreted in order to characterize the extent and magnitude of the Surface Cell and Test Cell taliks.

4.3.1 Geotechnical/ Geothermal Information

The geotechnical and geothermal information collected during the 2002/2003 investigations was discussed in Section 4.1 and 4.2. The following list summarizes the information that is pertinent to characterization of the Surface Cell and Test Cell taliks:

- The thickness of tailings contained within the Surface Cell ranges to a maximum of approximately 34 m;
- The thickness of tailings contained within the Test Cell ranges to a maximum of approximately 20 m;
- The tailings consist of sand and silt size particles that exhibit a relatively high specific gravity of between 3.9 and 4.5 (typical value for sand = 2.65);
- The tailings are generally saturated and some frozen samples were observed to contain ice lenses;
- Ground ice was encountered in several boreholes above and below the thawed tailings;
- The Surface Cell talik exhibits artesian pore pressures near the West Twin Dike and near hydrostatic pore pressures near the retained pond;
- Geothermal data collected from thermistors in the Surface Cell indicates sub-zero temperatures in the range of -0.1 to -1.0°C within the depth ranges where high pore pressures were encountered during drilling.

4.3.2 Talik Magnitude and Extent

Figure 13 illustrates six cross sections constructed through the Surface Cell. The significant geotechnical observations from the boreholes located along each alignment are indicated on the sections. Where possible, the estimated original lake bottom elevations have been plotted on the simplified logs provided on the sections.

Figure 14 illustrates the extent of the Surface Cell talik. These figures were developed based on information collected during the various site investigations and subsequent monitoring. The figures indicate the Surface Cell talik thins towards the upstream face of the dike and thins outwards from the middle of the Surface Cell. Figure 14 shows the approximate thickness and extent of the talik, its relation to the location of the original West Twin Lake and the former location of the deep water storage pond. Based on the information presented on Figure 14, it is estimated that the Surface Cell contains approximately 2,000,000 m^3 of thawed tailings.

Figure 15 illustrates two cross sections constructed through the Reservoir and Test Cell. The significant geotechnical observations from each borehole located along each alignment is indicated on the cross sections. Geotechnical information regarding the tailings in the Test Cell has been derived from boreholes drilled into the Test Cell Dike and Test Cell Test Covers. The deposition of tailings into the Test Cell has also been reconstructed from bathymetry maps as discussed in Section 3.2. The thickness of tailings within the Test Cell ranges between 2 and 14 m. The boreholes drilled into the Test Cell Dike indicate that frozen tailings extend down to a depth of between 14 to 18 m. Tailings beneath this depth were presumed to be thawed based on observations made during the geotechnical drilling conducted in the 2003 investigation.

Figure 16 illustrates the interpreted extent and magnitude of thawed tailings in the Test Cell. This drawing was derived using the limited geotechnical information that exists concerning the Test Cell and the historical bathymetry of the area. Based on the information presented on Figure 16, it is estimated that the Test Cell contains approximately 1,000,000 m^3 of thawed tailings.

No detailed geotechnical investigation of the Reservoir was undertaken. The deposition of tailings into the Reservoir was studied and discussed in Section 3.2. The thickness of tailings in the Reservoir likely ranges to a maximum of 18 m and the tailings that are currently under water have been under water cover since deposited. The tailings in the Reservoir will remain thawed. Since the present closure plan for the tailings in the Reservoir includes isolation by water cover and does not include permafrost aggradation, no further study of the tailings in the Reservoir is required.

4.4 Hydraulic Connectivity between Taliks and Reservoir

Previous drilling investigations (Golder 1999) and more recent work in 2002 and 2003 confirm the frozen nature of the tailings and bedrock directly beneath the West Twin Dike to an elevation of approximately 359 m. Hence, it is unlikely that any hydraulic connection exists across the foundation zone that was confirmed to be frozen. It is possible that hydraulic connection may exist at a greater depth.

Considering the geotechnical investigation results indicate that thawed tailings exist beneath the Test Cell Dike, hydraulic connectivity will exist between the Test Cell Area and the Reservoir.

4.5 Geochemical Characterization

4.5.1 Acid Generation Potential

4.5.1.1 1999 and 2001 Studies

The acid generation potential for tailings has been characterized by several studies completed by Lorax Environmental. Lorax (1999) describes metals analysis, acid base accounting, mineralogical examination, grain size analysis and humidity cell testing of a sample collected from the WTDA Surface Cell. Lorax (2001) presents acid base accounting of three additional samples of tailings from the WTDA Surface Cell. The results of these reports are summarized below.

The determination of total metal concentrations in tailings were: 25.2% iron; 2,610 ppm zinc; 406 ppm lead; 4.5 ppm cadmium; and 6 ppm silver.

The acid base accounting analyses confirmed that the tailings are potentially acid generating. The tailings samples contained high concentrations of total sulphur (39.5 to 48.0%) of which most was in the form of sulphides (39.3 to 45.5%). The samples contained some neutralizing potential ("NP") from 79 to 237 kg$CaCO_3$/t that was primarily in the form of carbonates (72 to 232 kg$CaCO_3$/t). However, the resulting net neutralization potential for tailings was negative (–997 to –1419 kg$CaCO_3$/t) and the resulting ratio of NP to acid potential was much less than one for all samples (0.05 to 0.19), which demonstrates their classification as potentially acid generating.

The mineralogical analysis indicated that sulphides in the tailings sample consisted primarily of pyrite (80%) with trace amounts of marcasite and sphalerite. Calcite (crystallized carbonate) comprised the majority of the remainder of the sample with minor quartz grains.

The sample analysed for grain size was generally within the size ranges for silt and sand with approximately 81% finer than 0.2 mm (i.e., fine sand and silt) and 100% finer than 2 mm (i.e., coarse sand and finer).

A humidity cell test was conducted for 35 weeks. The test was run at 20°C for 26 weeks and then at 2°C for the remainder of the test to determine the effect of colder temperatures. The primary observations of the tests are as follows:

1. Leachate remained neutral (pH>7.5) throughout the test;
2. Sulphate production rates were generally steady throughout the test but became reduced by approximately 50% at the time of switching to the colder temperature; the calculated rate of carbonate depletion did not vary with temperature;
3. Calcium and magnesium were released at rates of at least two orders of magnitude greater than other metals; and
4. Zinc was released at a rate of approximately two orders of magnitude greater than other heavy metals.

The general conclusions of the humidity cell tests were:

1. The tailings sample was confirmed to be potentially acid generating (neutralization depletion rate faster than acidity depletion rate);
2. A reduction in temperature reduced the rate of sulphate production;
3. The total sulphide concentration was 39% to 46% with pyrite being the dominant sulphide mineral present; pyrrhotite was not identified as being present;
4. Zinc is the most mobile metal in the tailings; and
5. Neutralization potential is present primarily in the form of carbonates.

4.5.1.2 2003 Study

Geochemical analyses were conducted on a sample of tailings collected from the surface of the Surface Cell as part of a study conducted for the Department of Indian Affairs and Northern Development, Nunavut, in 2003. The geochemical data generated for these studies has been made available for use in the Nanisivik 2004 RCP. The tailings sample was separated into a “sand” sized fraction and a “silt” sized fraction in the field such that two sets of duplicate analyses were conducted.

The study included ICP metals concentrations, acid base accounting and leach extraction tests and a mineralogical analysis.

The determinations of total metals concentrations agreed with the previous results with: >15% each iron; 2,926/2,262 ppm zinc; 856/1,912 ppm lead; 6/5 ppm cadmium; and 3.8/4 ppm silver for the sand/silt samples, respectively. The metals concentrations do not indicate a consistent bias of higher concentrations in one of the size fractions. For example, lead and copper are greater in the silt sized fraction while zinc and chromium are greater in the sand sized fraction.

The acid base accounting analyses further confirmed that the tailings are potentially acid generating and the data agreed with the previous analyses. Tailings pH was 8.5 and 8.6. The concentrations of total sulphur (32.8 and 34.1%) were slightly lower than the previous analyses but most was, again, in the form of sulphides (32.3 and 33.8%). The neutralizing potential ("NP") (321.3 kg$CaCO_3$/t each) was slightly higher than previous analyses but most was, again, in the form of carbonates (324.2 and 319.2 kg$CaCO_3$/t). The resulting net neutralization potential was again negative (–703.8 and –744.4 kg$CaCO_3$/t) but higher than previous analyses. The resulting ratio of NP to acid potential was again much less than one (0.3 each) but higher than previous analyses.

The mineralogical analysis reinforced the previous observation that pyrite is the dominant sulphide present in concentrations of approximately 56% and 62%. Pyrrhotite was not observed. Dolomite was the dominant carbonate mineral present at concentrations of approximately 33% and 29%. Comments on textures were also provided by the petrographic analyst that indicated "no rims or encapsulation" in either the silt or sand samples.

Leach tests with distilled water were run at a 3:1 mix ratio. The tests produced leachate containing 1,796/1,392 mg/L sulphate, <0.03 mg/L iron each and 0.62/1.98 mg/L zinc for the sand/silt samples, respectively, at neutral pH of 7.5. The leachate concentrations do not indicate a consistent bias of higher concentrations in one of the size fractions. For example, zinc and cadmium are greater in the silt sized fraction while sulphate and cobalt are greater in the sand sized fraction.

4.5.1.3 Conclusions

The geochemical tests clearly demonstrate that the tailings are potentially acid generating and will contain a high sulphide content, as expected. Nonetheless, the tailings do contain abundant neutralization potential and leachate pH remained neutral during both humidity cell and leach extraction tests.

The absence of pyrrhotite from the tailings is noteworthy. Pyrite is less geochemically reactive than pyrrhotite and, therefore, reduces the risk of acid rock drainage slightly as compared to cases where pyrrhotite is abundant. Similarly, the petrographic observation that no rims or encapsulation are present is noteworthy as it suggests that the acid generation process has not been initiated in the tailings since the oxidation process typically results in mineralogical changes in the surfaces of some minerals.

Tailings paste pH was neutral to slightly alkaline, including the grab sample collected from the pond surface in 2003. This is further indication that the tailings have not oxidized (to 2003), even at surface under maximum exposure to oxygen.

The leaching of metals and sulphate in the humidity cell test and the leachate extraction test provides an indication of the production rate of these contaminants. The humidity cell test also provides further indications of the beneficial effects of reduced temperature through reduced contaminant production rates.

Tailings groundwater quality shows no indications of the occurrence of acid rock drainage in the tailings (i.e., dissolved metal concentrations are generally low), which is attributed to the alkaline pH and rapid freezing and/or saturation of the tailings upon deposition.

4.5.2 Rate of Oxidation

Given that the tailings are confirmed to be potentially acid generating, the rate of oxidation is an important consideration. The rate of oxidation in Nanisivik tailings will be controlled by two key factors: temperature and oxygen availability.

A common reference regarding sulphide oxidation in the north is Dawson and Morin, 1996, which demonstrates that the rate of sulphide oxidation decreases at low temperatures but may not be eliminated completely at freezing temperatures. This text also confirms that the cold arctic climate substantially limits the rate of oxidation by maintaining frozen conditions through most of the year. This is especially applicable to the Nanisivik site because of the extremely cold daily temperatures (average –15°C).

The site specific effects of temperature on the rate of sulphide oxidation in Nanisivik tailings were measured by Dr. Bo Elberling of the University of Copenhagen in 1998 as part of a multi-year research program. The tests were conducted on columns loaded with undisturbed tailings. One of the test columns consisted of older, well-drained tailings from the Surface Cell. The measurements from that column confirmed that the rate of oxidation decreased with decreasing temperature. At 2°C, the rate of oxidation was reduced by a factor of 30% relative to the rate at 20°C. Oxidation was observed to continue at a decreasing rate to as cold as 0°C. The temperature at which oxidation would cease completely was not determined and could not be accurately extrapolated.

Dr. Elberling's testwork also demonstrated that the sulphide oxidation rate was controlled by oxygen diffusivity and not by reaction kinetics. The testwork went on to verify that, as is generally acknowledged, application of a saturated diffusive barrier layer would reduce oxygen diffusivity and, therefore, the oxidation rate by several orders of magnitude.

The reference to an oxygen diffusive barrier is analogous to both the proposed shale cover on the Surface Cell/Test Cell areas and the proposed water cover on the Reservoir. The shale cover on the Surface Cell is anticipated to form a frozen, saturated zone at the base of the cover. The frozen, saturated zone will provide the same beneficial effect of a water saturated diffusive barrier layer due to the reduced oxygen diffusivity in water and ice relative to air. The water cover on the Reservoir will provide this same benefit.

Therefore, the rate of oxidation of tailings after reclamation in the Surface Cell, the Test Cell area and the Reservoir is anticipated to be reduced to a negligible level due to the effects of cold temperatures/freezing and the presence of oxygen diffusivity barriers.

4.5.3 Transport Mechanism

The MEND study referenced above (Dawson and Morin, 1996) confirms that the cold arctic climate substantially restricts the primary contaminant transport mechanism by maintaining frozen conditions through most of the year when surface water is immobile.

This is directly applicable to the Nanisivik site because of the extremely cold temperatures (average mean annual temperature of -15°C). The ground (including the covers on the Surface Cell and Test Cell areas) at Nanisivik will be frozen to the surface during the winter season and any transport of contaminants will be prevented. The cover is designed to maintain frozen conditions within the tailings and the base of the cover even during the short summer season. It is anticipated that a frozen saturated zone will form at the base of the active layer (within the cover) as a result of the recurring infiltration of snow melt, runoff and precipitation water. This zone will prevent the contact of surface run off water with tailings and, therefore, will prevent the transport of contaminants to surface water.

4.5.4 Summary

There are two primary conclusions regarding the geochemistry of the tailings that stem from the information described above:

1. The tailings are potentially acid generating.
2. Acid generation has not yet become established in the tailings.
3. The rate of oxidation of tailings after reclamation in the Surface Cell, the Test Cell area the Reservoir is anticipated to be reduced to a negligible level due to the effects of cold temperatures/freezing and the presence of oxygen diffusivity barriers.
4. The frozen saturated zone in the base of the cover on the Surface Cell and Test Cell areas will prevent the contact of surface run off water with tailings and, therefore, will prevent the transport of contaminants to surface water.

The first conclusion is as expected given that the Nanisivik mine processed sulphidic ore. The confirmation that the tailings are acid generating creates the requirement for reclamation of the tailings with the objective of preventing, controlling or mitigating acid generation and transport of oxidation products.

The second conclusion is reasonable for the Nanisivik site given the relatively high inherent neutralization potential and high pH of the tailings and given that tailings were frozen and/or saturated in place shortly after deposition. This conclusion suggests that reclamation measures at this site should be focussed on the prevention of acid generation rather than on control or mitigation strategies.

The remaining conclusions are also as expected given that they mirror the widely accepted conclusions of previous research on the effects of the Arctic climate on acid rock drainage.

5.0 CLOSURE OBJECTIVES

5.1 Specific Reclamation Objectives

CanZinco's approach to closure and reclamation of the Nanisivik Mine site follows the "Mine Site Reclamation Policy for Nunavut" published by the Department of Indian Affairs and Northern Development in July 2002. The primary objectives of the Reclamation and Closure Plan are in accordance with the Mines Reclamation Policy to "ensure the impact of mining on the environment and human health and safety is minimized".

The specific reclamation objectives for the WTDA follow from this policy. Further, the specific reclamation objectives follow from the "Guidelines for Abandonment and Restoration Planning for Mines in the Northwest Territories" (1990), which states that the objective of a mine reclamation plan should be to "prevent progressive degradation and to enhance natural recovery in areas affected by mining."

Given that the primary risks posed by the WTDA facilities are related to the potential for acid rock drainage, the potential for the physical movement of tailings to the environment and the potential loss of surface land use values, the specific reclamation objectives for the WTDA reclamation plan are as follows:

1. Isolate potentially acid generating tailings from the atmosphere to minimize the risk of acid rock drainage;
2. Minimize the risk of physical movement of tailings to the environment; and
3. Provide a safe and useable surface environment that corresponds to the natural surroundings.

The reclamation measures described in this report are proposed as an efficient and practical means of achieving these objectives. The reclamation measures are described in detail Section 6 and are summarized in an overview format, for ease of reference, in Section 5.2.

5.2 Specific Measures

5.2.1 Minimization of Oxygen Exchange in Surface Cell and Test Cell Tailings

The closure plan for Surface Cell and Test Cell tailings is based on minimizing oxygen exchange by placing a cover of shale and sand and gravel over the exposed tailings. The cover will provide thermal insulation, to maintain frozen conditions and allow for permafrost aggradation, with a durable surface cap of local natural material. The cover will be thick enough to maintain continuous frozen conditions within the underlying tailings during mean annual and warmer conditions, even in light of a worst case climate change predictions over the next 100 years.

5.2.2 Minimization of Oxygen Exchange in Reservoir Tailings

The tailings contained within the Reservoir will remain under a minimum of 1 m of water cover. Erosion protection will be applied to the tailings within a zone just above and just below the pond water level to minimize the risk of re-suspension due to wave and ice action. The final water level in the Reservoir pond and Polishing Pond will be the same and will be returned to the original, pre-mining elevation of West Twin Lake (370.2 m). The re-establishment of previously existing water levels hence negates the need for a water retention dam.

5.2.3 Transfer of Water Flow from Surface Cell to Reservoir

A spillway and outlet channel will be constructed to safely pass seasonal run-off and severe storm events around the West Twin Dike, to minimize the risks to the tailings solids retention dike. The spillway will be constructed near the south abutment of the dike and will convey water from the Surface Cell to the Reservoir without the aid of pumps and/or siphons.

5.2.4 West Twin Dike

The West Twin Dike will remain in place during closure for permanent retention of Surface Cell tailings. The dike will be graded smooth to a slope shallower than previously its approved closure design and will be covered with Twin Lakes sand and gravel to prevent erosion.

5.2.5 Test Cell Dike

The Test Cell Dike will partially remain to retain tailings solids. The crest of the Test Cell Dike will be graded as a portion of the grading plan for the entire cell.

5.2.6 Polishing Pond and West Twin Lake Outlet

The water control outlet structure that was used to release water in a controlled manner during mine operations will be removed and replaced with an open outlet channel and overflow weir that is approximately 7 m wide at the base. The invert level will be set at the original elevation of the outlet of West Twin Lake (i.e. 370.2 m). Water passing through the outlet channel will join with water flowing from East Twin Lake and flow into Twin Lakes Creek.

5.2.7 Reclamation and Post-Closure Period Monitoring Programs

A comprehensive program for determining the long-term performance of the proposed reclamation measures is to be implemented. The monitoring information will also provide information that identifies areas where maintenance, repairs or contingencies may be required in the short term.

The monitoring information to be collected includes:

- ground temperatures within the cover, the tailings and the natural ground;
- subsurface water pressures related to freezing of the taliks;
- quality of water entering the environment;
- climate data;
- regular inspections of surface conditions by trained technical staff; and
- professional inspections of surface conditions by a professional geotechnical engineer.

This information will be reviewed by CanZinco, in concert with other technical experts as required, and CanZinco will undertake appropriate actions where required. Additionally, this information will be forwarded to the Nunavut Water Board for their review and public posting.

If some of the reclamation measures are found to be substantively underperforming, then a contingency action would be performed. The specific actions to be implemented would depend on the circumstances but would include consideration of the following:

- Increased frequency of monitoring;
- Blading of the cover, erosion protection and spillways to fill/close cracks or settlement depressions;
- Placement of additional material or different material (i.e. larger rip rap);
- Clearing of debris for water flow paths;
- Application to the Nunavut Water Board for modification of works; and

- Water treatment with lime.

In 2010, a comprehensive, all-encompassing assessment of the monitoring information is to be conducted that will determine whether the reclamation objectives have been achieved and whether the WTDA is considered to be successfully reclaimed.

6.0 CLOSURE PLAN

6.1 Surface Cell and Test Cell Reclamation Covers

The design of the Surface Cell and Test Cell closure covers are discussed in detail in the component closure plan report "Engineering Design of Surface Reclamation Covers" (Water License Part G, Item 4). The following sections review the design aspects of the covers as requested in water license clause Part G, Item 15, requirement iv.

6.1.1 Design Criteria

The closure concept for the Surface Cell and Test Cell is a cover of natural materials that will be constructed to mitigate the potential long term environmental impacts from exposed tailings. This cover will prevent wind and water erosion of the tailings. In addition, the cover will limit oxygen exchange between the air and tailings and will maintain frozen conditions within the tailings throughout the year.

Mine waste covers constructed of natural materials in cold regions generally incorporate an insulating layer of material overlain by a layer of coarse-grained armouring material. The insulating material is generally saturated and frozen to provide a low permeability barrier to infiltration by water and oxygen. The insulating layer generally exhibits a low thermal conductivity that does not allow the full penetration of active layer thaw. The purpose of the armouring layer is to prevent erosion of the underlying insulating material. A light coloured armouring material may provide additional geothermal benefits by reflecting sunlight, thereby reducing heat absorption.

6.1.2 Materials

The reclamation cover for the Surface Cell, Test Cell and tailings at the toe of the West Twin Dike will be constructed of an insulating layer of shale overlain by an armouring layer of Twin Lakes sand and gravel. For a detailed description of the cover materials, their availability and geotechnical and geochemical characteristics, refer to the Reclamation Covers Report (Water License Part G, Item 4).

6.1.3 Design

The reclamation cover of the tailings in the Surface Cell, Test Cell and at the toe of the West Twin Dike will be constructed of a layer of 0.25 m (minimum thickness) of Twin Lakes sand and gravel (for erosion protection) overlying a 1.0 m (minimum thickness) layer of quarried shale (for insulation and containment of infiltration water). The gradation specifications for each of these materials is included in the Reclamation Covers Report (Water License Part G, Item 4). The Twin Lakes sand and gravel will be quarried from the deltaic deposit located between East Twin Lake and West Twin Lake. The shale will be quarried from several sites around the mine site. The majority of the shale is expected to be derived from the Mt. Fuji, East Twin and West Twin quarries.

Topographic high points on the tailings surface will be graded prior to construction of the cover to limit the thickness of cover required in the topographic low spots. In addition, where fill thicknesses are greatly in excess of 1.25 m, select native materials such as till or shale quarry strippings may be used. The proposed tailings grading plan for the Surface Cell is illustrated on Figure 17. The cover will be constructed according to the cover grading plan illustrated on Figure 18. The longitudinal grade of the main drainage swales within Surface Cell cover will be constructed to a minimum of 0.5%. The drainage swales will direct surface water to the spillway inlet location near the south edge of the Surface Cell.

The proposed tailings grading plan for the Test Cell and tailings located at the toe of the West Twin Dike is illustrated on Figure 19. The cover will be constructed according to the cover grading plan illustrated on Figure 20. The main drainage swales within the cover will be constructed to a minimum grade of 0.5%. The drainage swales will direct surface water into the Reservoir.

6.1.4 Construction Considerations

The remoteness of the site, the available options to transport equipment and supplies to site and the length of the construction season present limitations on the construction schedule for reclamation activities.

Transportation to site by sea generally occurs between July and September, although vessels have come in as early as May and as late as October. Any required heavy equipment or bulk supplies would have to be delivered during these months.

The construction season available for quarrying shale extends between April and October. Different extraction methods may be required at different times of the year. Blasting may be conducted during the early spring when ground conditions are frozen. Ripping of the thawed portion of the active layer may be completed in late spring and summer. Testing completed earlier by mine site staff on a sample collected from a test blast indicates that similar grain size characteristics can be achieved to that of normal ripping operations.

If saturated, tailings may be displaced into the shale material during cover placement due to the weight of the construction traffic. Therefore, some areas may need to be covered while the ground is frozen. The ponded water in the Surface Cell and Test Cell will be removed prior to initiation of cover placement in those areas. Currently, only the subaqueous tailings and a small “island” of tailings on the west side of the Surface Cell would be exposed at surface once the ponded water is removed. The remaining tailings in the Surface Cell are currently already covered by a layer of shale that is estimated to average 300 mm thickness. The tailings currently covered by the thin layer of shale may be covered while the tailings are thawed during the summer due to the increased trafficability provided by the shale cover. Most of the tailings in the Test Cell and toe of the West Twin Dike are currently exposed at the ground surface.

The regrading of tailings must be accomplished when the active layer thaw is minimal. It is suggested that regrading of the tailings surface occur in May/June to avoid the poor trafficability later in summer. It is anticipated that the reclamation works will proceed over the summer portions of a two year period.

The required volumes of shale and Twin Lakes sand and gravel for each portion of the WTDA are summarized in Table 7. These quantities are neat and in-place with no allowance for bulking.

6.2 Reservoir Shoreline Tailings and Reservoir Water Cover

6.2.1 Design Criteria

The Reservoir/Polishing Pond area will be reclaimed by restoring the water level to the natural elevation of 370.2 m asl (the “normal” water level - NWL). All tailings within the Reservoir will be relocated to greater than 1 m below the NWL, to as great a degree as practical. This approach continues to provide the long term benefits of water cover for prevention of acid rock drainage without the need for water retention dams or manually operated outlet structures. Additionally, the design water depth will prevent the physical remobilization of tailings due to wave action, as described in later sections.

The perimeter of the Reservoir pond has been divided into three sections, for reclamation planning: the toe of the Test Cell Dike, the toe of the West Twin Dike; and the remainder of the perimeter. The extent of these three areas are shown on Figure 21.

For the perimeter other than the toe of the dikes, it is possible that some tailings are present in some locations above or near the NWL. In these areas, tailings will be pushed, by bulldozer or backhoe, to below 369.2 m asl to provide at least 1 m water cover.

For the perimeter areas below the West Twin and Test Cell Dikes, tailings are exposed substantially above the NWL. In the case of the Test Cell Dike area, tailings rise to an elevation of approximately 371 m asl, the nominal elevation of the tailings beach upon which the Test Cell Dike is seated. In the case of the West Twin Dike, tailings are present in a relatively flat-sloped beach that extends from the pond edge to the toe of the dike, a distance of approximately 150 m. Within these two areas, it is estimated that there are more than 100,000 m^3 of tailings above the NWL in the Reservoir. It is considered impractical to remove all of these materials due to the potential long term effects on dike stability if ponded water (and resultant heat source) were allowed at the toe of the dikes.

In the long term, permafrost aggradation is anticipated to progress towards the perimeter of the Reservoir pond to the point where permafrost forms around the pond shoreline proximal or below the pond water level. This phenomenon is commonly observed in natural conditions in the north where a lake has a shallow sloping shoreline (MacKay 1962). Permafrost can form to a shoreline depth under water equivalent to approximately 2/3 of the typical thickness of ice. In the summer thaw season, the shoreline at the waters edge does not thaw and the lake bottom sediments may not thaw until a depth greater than 2/3 of the typical ice thickness is reached. Should this occur around the perimeter of the Reservoir Pond, as is expected, littoral tailings will be subject to permafrost aggradation. However, given that the timeframe for this phenomena to occur is unknown, reclamation efforts are required to protect against oxidation of tailings and transport of contaminants during the interim period, as described below.

6.2.2 Design Analyses

6.2.2.1 Reservoir Water Levels in Closure

The estimates of the maximum water level (MWL) and low water level (LWL) made in the previous studies (Golder, 2002) were updated to reflect the proposed grading plans and water levels in the Surface Cell and the Reservoir. The updated water levels as derived from Golder (2004 b,c,d) are summarized in Table 8. The following observations could be made:

- The seasonal water level fluctuations in the Reservoir (West Twin Lake) will be limited to a few centimetres because of the small contributing drainage areas, low precipitation, and large spillways
- The peak water level corresponding to the PMP storm event was estimated to 370.8 m (i.e. a flow depth of 0.6 m on the outflow weir structure invert).
- The water level decline under the drought conditions had been evaluated in the previous study. The maximum water decline under extremely dry conditions was estimated to be 200 mm.

In summary, the extreme range of water level fluctuation in the Reservoir is estimated at 0.8 m, from 370.0 m (LWL) under extreme drought conditions, to 370.8 m (MWL) under PMP storm event conditions.

6.2.2.2 Wave Effects

During the open water season, which lasts approximately four months per year at Nanisivik, wind action may cause waves on the water surface. Closure planning measures involved an estimation of the significant wave heights possible, and an assessment of the possible effects on the tailings and shoreline erosion protection design.

Because of the limited size of the Reservoir, the estimated significant wave height is only 0.4 m. Since the 1.0 m water cover provided is 2.5 times deeper than this wave height, tailings re-suspension is not expected to be problematic. In fact, a review of the water quality data submitted as part of the SNP portion of the License during operations, indicates TSS values and metal concentrations were well within the discharge criteria (despite the fact that 1.0 m of water cover was not maintained constantly during that period).

The available wind data were summarized, the design wind conditions selected, and the wave height calculated are detailed in the following paragraphs:

Design Wind Speed

Two sources of wind data were used: the site data, collected at the Nanisivik Mine meteorological station; and the regional data, collected at the Pond Inlet Airport meteorological station.

The on-site wind record is available for four years: 1993, 1995, 1996 and 1997. The data shows the highest observed wind velocity was 91 km/h. Because of the short period of record, the frequency of occurrence (or return period) cannot be assigned to this observed value.

To gain better insight into the frequency distribution of the wind speed data, the regional, long-term records at the Pond Inlet Airport were used. Pond Inlet is located approximately 220 km east from the Nanisivik Mine. The wind speed record at the Pond Inlet Airport includes 28 years of observations, from 1976 through 2003. The extreme wind speed statistics at the Pond Inlet Airport were provided by Environment Canada. The 100-yr return wind speed at the Pond Inlet Airport is estimated to be 95.6 km/h (with a 95% confidence interval from 86.1 km/hr to 105.2 km/h).

The maximum wind speed observed at the Nanisivik Mine (91 km/hr) is consistent with the estimated maximum wind velocity at the Pond Inlet Airport (95.6 km/hr).

The definition of the design wind speed for erosion protection normally involves two steps:

- Selection of the wind speed value based on the in-land meteorological records.
- Application of a correction factor, which accounts for the fact that the wind speed above the water surface is higher than that over the land surface.

For the purpose of conservative design, the upper limit of the 95% confidence interval of a 100-yr return wind speed (105.2 km/h) was used in the design. Because of the limited fetch on the Reservoir, the correction factor is small (i.e. 1.07). The correction results in a design wind speed on 112.6 km/h.

Design Wave Height

The wave height can be calculated using the following equation (Smith, 1995):

$$h_w = 0.00513\, V^{1.06} F_e^{0.47},$$

where:
h_w is the wave height in meters,
V is the design speed in km/h (112.6 km/hr in this case),
F_e is the effective fetch in km (0.3 km in this case).

Using this equation, the design wave height in the Reservoir (West Twin Lake) was calculated to be 0.4 m (Table 9).

6.2.3 Design Considerations

As shown on Figure 21, erosion protection will be provided around the perimeter of the Reservoir, wherever there is a potential for erosion of existing tailings. Erosion protection will not be provided on natural ground located along the new shoreline.

Erosion protection on remnant tailings will extend between a top elevation of 370.8 m and a bottom elevation of 369.2 m. The rationale for these elevations is as follows:

- The top elevation is equal to the MWL under PMP storm conditions (i.e. Elev. 370.8 m).
- The top elevation is equal to the NWL (370.2 m) plus 1.5 times the design wave height.
- The bottom elevation is equal to the LWL (i.e. 370.0 m predicted under extreme drought conditions) minus 1.0 m (to maintain 1 m of water cover over exposed tailings).

As a result, rip rap protection will be provided between elevation 369.4 and 370.8 m, a slope distance of 1.4 m.

6.2.3.1 Rip Rap Sizing

Rip rap is the most common means of protecting slopes against wave action. The required median rip rap size was calculated based on the wave height (h_w) and the bank slope. The median stone size is given by the following equation:

$$D_{50} = 377\ h_w / \sin(70-\alpha)$$

where:

D_{50} = median stone size (mm)
h_w is the wave height (m), and
α is the angle of the slope to be protected (degrees).

The required median stone size was calculated using parameters included in Table 9 for a slope of 4H:1V. Based on the analysis, it is recommended that rip rap with median size of 200 mm be used at the toe of the shoreline erosion protection. The recommended gradation specifications for the rip rap are included in Table 10.

Rip rap will not be provided on natural shoreline. Potential erosion of these areas would not result in the transport of tailings out of the Reservoir. If excessive erosion of the natural shoreline area is observed during routine surveillance, appropriate maintenance will be undertaken to repair the locally affected area.

6.2.3.2 Protection of Shoreline Against Moving Ice Masses

It is understood that the ice cover on the water bodies at Nanisivik can reach a thickness of approximately 2 m. During the ice break period, the ice cover can move and potentially exert forces on the shoreline causing erosion.

The following considerations apply to the ice erosion:

- In theory, the dynamic forces that could potentially be exerted by the free masses of ice are very high. Erosion protection against such theoretical forces is not practical, given the sedimentary bedrock units located at Nanisivik Mine.
- The movement of ice masses in the Reservoir will be constrained by the following structures that will remain after closure:
    - the remnant Test Cell Dike;
    - the baffle dike (lowered);
    - the access road causeway (lowered); and
    - the West Twin outlet channel and overflow weir.
- Due to the limited size of the Reservoir, the extent of ice cover movement and the impact on the shoreline within the Reservoir will be restricted. Accordingly, it is anticipated that the erosion by ice action will be limited.

In summary, the erosion and associated environmental impact caused by the ice impact are expected to be limited, but the cost of protecting the shoreline against it would be prohibitive. It is recommended, therefore, that the Reservoir shoreline erosion protection not be designed specifically to withstand the ice impact. Some later maintenance may be required to repair impacted rip rap. Additionally, water quality data collected during mine operations as part of the Surveillance Network Program (SNP) portion of the License, indicates TSS values and metal concentrations were well within the discharge criteria. Since the tailings were not protected from ice effects during this time, it is not anticipated that any negative impacts on water quality will result from tailings re-suspension related to ice effects.

6.2.3.3 Prevention of Ice-entrained Tailings Transport

Because of the winter ice cover on the Reservoir may be approximately 2 m thick, it is possible the tailings may become frozen onto the base of the ice pan around the perimeter, and in shallow area of the Reservoir. Upon ice break-up in spring, these tailings could potentially be transported by moving ice. Ice movement will be restricted however because the baffle dike and causeway will be left in place with a lowered crest elevation 369.7 m (0.5 m below the NWL). The Outlet Control Embankment and spillway will also inhibit the movement of ice out of the Reservoir. It is therefore expected that the ice will melt out within the Reservoir, dropping any adfrozen tailings back into the pond.

Records of water quality at the outlet structure (Station 159-4) indicate that the total suspended solids values were low during the period of active mine operation, in spite of active tailings deposition. This suggests that the internal structures (i.e. the baffle dike, access road causeway and Outlet Control Structure) were effective at preventing off-site transport of tailings with ice.

6.2.4 Design of Shoreline Erosion Protection

6.2.4.1 Design Elements

The erosion protection details are shown on Figure 21.

Three typical sections have been developed corresponding to the various zones:

- Section A illustrate the tailings relocation along the original West Twin Lake shoreline where a thin layer of tailings is present on the relatively steep bank. These tailings will be excavated and re-located to below Elev. 369.2 m to meet the 1.0 m minimum water cover requirement. The remaining bank will consist of native soils.
- Section B is for use in the area extending away from the toe of West Twin Dike which consists of a relatively shallow sloping tailings beach. The design involves covering the tailings with the thermal cover to Elev. 369.2 m (provides a miimum water cover of 1 m at that point) and providing erosion protection below the toe of the thermal cover to match the subgrade.

- Section C is for use on the tailings beyond the toe of the Test Cell Dike. The existing tailings in this area are relatively steeply sloping at approximately 12-15%. Again the thermal cover will be placed down to elevation 369.2 m and erosion protection will be placed below the toe.

The sections were developed to minimize tailings relocation and allow for construction on potentially saturated tailings beaches.

A bedding layer will be provided to prevent waves from "plucking" fine materials from the thermal cover beneath the rip rap. Table 10 provides gradation specifications for the bedding layer (Type 2) material.

6.2.5 Construction Considerations

To reduce difficulties associated with the requirement for traffic of construction equipment on wet tailings beaches, the water level will be lowered to 368 m prior to construction. This will allow placement of the shale cover to beach elevation 369.2 m (and slightly lower at the toe of the cover). Where the tailings are saturated and soft, the 1 m thick shale layer may have to be pushed in a single lift. By over-building and then trimming the shale, traffic directly on tailings can be avoided. The erosion protection can then be placed with the equipment sitting on the shale layer.

6.3 Dike Stability Considerations

6.3.1 Design Criteria

The West Twin Dike will remain in place during closure for permanent retention of Surface Cell tailings. The dike currently exists at a slope (3.7H:1V) shallower than its previously approved closure design and will be covered with an armouring layer of Twin Lakes sand and gravel to prevent erosion. The Test Cell Dike will be partially removed and regraded during closure. The remaining portion of the dike will merge into the shoreline reclamation cover. While this remnant dike will not retain water during closure, it is still considered important to assess the long term stability of the remnant dike and shoreline reclamation cover.

The closure design of the dikes must exhibit satisfactory stability in the short and the long term. To address this, several analyses were completed to assess the current and long term stability of the dike.

6.3.2 West Twin Dike Stability Analyses

The stability analyses completed for the West Twin Dike are reviewed in detail in Appendix V. The stability of the West Twin Dike was reviewed considering static and pseudo-static (seismic loading) conditions. The stability was assessed using two methods; one considering the dike to act as a rigid block due to its frozen state with thawed tailings and water applying pressure on its upstream face; and the second considering the dike to contain a thawed zone at depth that exhibits artesian pore pressures. The results of the analyses indicate that the dike exhibits acceptable stability considering assumed strength parameters and applying static and pseudo-static loading conditions.

6.3.3 Test Cell Dike Stability Analyses

The stability analyses completed for the Test Cell Dike is reviewed in detail in Appendix V. The stability of the Test Cell Dike and associated sloping reclamation cover were assessed considering static and pseudo-static conditions. The stability was assessed assuming piezometric pressures in the slope equivalent to the water level in the adjacent Reservoir. In addition, the downstream slope of the cover was assumed to be 3H:1V. The results of the analyses indicate that the dike and associated sloping cover exhibit acceptable stability considering assumed strength parameters and applying static and pseudo-static loading conditions. Slope angles flatter than 3:1, such as 4:1, would be considered adequate as well.

6.3.4 Liquefaction

Although the stability analyses consider pseudo-static forces that may be applied to the dike during a seismic event, the analyses do not consider the possible liquefaction of the tailings. For liquefaction to occur, the tailings material needs to be both loose and saturated. Hence, for all areas where the tailings are frozen, liquefaction is not a concern. In the Surface Cell, if the tailings in the talik were to liquefy, the pore pressures could be transferred onto the West Twin Dike structure. As shown in the detailed analysis, very high pore pressures are required to reduce the Factor of Safety to unity for this dike.

The potential consequences of a liquefaction event also needs to be considered. To review, no water will be retained by the West Twin Dike during closure. Liquefaction of the tailings may result in disturbance of the surface cover, which would have to be repaired. Pore pressures generated during liquefaction may also result in deformation of the West Twin Dike, and worst case scenario, the release of some tailings and talik pore water into the Reservoir. Remedial work would then be required to handle and treat the pond water and to clean up any released tailings.

Considering the case of the Test Cell Dike and associated sloping reclamation cover, it is important to note that the dike is currently underlain by 14 to 18 m of frozen tailings. Hence, liquefaction would only be a concern for thawed tailings located beneath this frozen zone beneath the dike. Liquefaction would likely result in some settlement of the dike, with resulting disturbance to surface features such as design grade and rip rap placement. But again, no water will be retained by this structure during closure, so catastrophic release of surface water is not a consideration.

Additionally, liquefaction will only be a concern when the tailings are thawed. Hence, as permafrost aggradation occurs into the tailings over the next 30 years, the liquefaction concern will be reduced and then eliminated once the tailings are completely frozen.

6.3.5 Closure Contouring of West Twin Dike

The West Twin Dike is currently graded to an overall slope of 3.7H:1V. The shale will be covered with an armouring layer of 0.25 m of Twin Lakes sand and gravel. One portion of the downstream slope of the dike still need to be covered with shale. The required volumes of shale and armouring sand and gravel is summarized in Table 7.

6.4 West Twin Dike Spillway Design

The main purpose of the spillway is to provide passive drainage of runoff out of the Surface Cell and around the West Twin Dike to prevent excessive ponding and a potential heat source to the underlying tailings. The spillway will be an open channel, generally excavated into bedrock. The details of this design are provided in the West Twin Dike Spillway Design report (Water License requirement Part G, Item 7) provided under a separate cover.

6.4.1 Design Criteria

The spillway has been designed to safely convey flows from the routed 24-hour probable maximum precipitation (PMP) storm event. The inlet invert elevation will be 384.0 m. Flows will report to the Reservoir, which will have normal water level 370.2 m.

To prevent scour of tailings in the Reservoir, energy dissipation will be provided within the spillway outlet structure.

The spillway has been designed to require minimal maintenance for mine closure. Routine surveillance and maintenance requirements are set out in Section 6.4.4.

6.4.2 Design Considerations

The estimated PMP event is 140 mm in 24 hours (Section 2.6). Under this event, the estimated (routed) peak inflow to the spillway would be approximately 5.2 m^3/s, corresponding to a water level of 384.6 m in the Surface Cell. For sloping sections of the spillway, the flow depth would be less than 0.6 m (i.e. 0.52 m for segments sloping at 1% or 0.31 m depth for segments sloping at 5%). A protected flow depth of 0.6 m minimum is provided throughout the spillway.

The spillway alignment was selected to situate the base within intact bedrock while considering the required excavation volumes. The geologic conditions were inferred from the borehole and test pits completed in the spillway area by BGC and Nanisivik Mine (Golder 2004a). In general, the stratigraphy in the area of the spillway is as follows:

- Till, overlying;
- Frost shattered bedrock, overlying;
- Competent bedrock.

Ground ice was observed in boreholes near the Reservoir shoreline (i.e. BH1 and BGC03-06).

Where the design flow depth is not entirely within intact bedrock, erosion protection will be provided. Based on flow velocity calculations, it is recommended that segments of the spillway sloping at more than 1% gradient will require rip rap with a median stone size (D_{50}) of 300 mm. For flatter segments (sloping at up to 1%), the required median stone size was estimated to be 100 mm. A bedding layer will be provided below the rip rap and a granular filter (Type 3) will be placed on overburden as required.

A service road will be provided along the full length of the spillway to allow for periodic inspection and maintenance.

6.4.3 Design Details

The spillway design is presented on Figure 22.

The spillway will consist of a 6 m wide open channel approximately 565 m in length. The vertical alignment was selected to minimize excavation, while still situating the invert of the channel within intact bedrock to the extent possible. The base width selected is a reasonable minimum to allow for excavation considering that blasting will likely be required.

The spillway is comprised of three segments (Section A-A’, Figure 22):

- The **inlet portion** of the spillway (about 120 m long) will be flat (slope of 0%) with a nominal invert of Elev. 384.0 m. The spillway inlet will be constructed to match the Surface Cell cover contours and rip rap protection will be provided.

- The **chute portion** will comprise a segment about 130 m long with 1% slope followed by a length of approximately 270 m at 5% slope.
- The **outlet portion** will consist of a plunge pool (to dissipate flow energy) and an associated outflow channel. To facilitate drainage, the plunge pool will have a floor level of Elev. 370.6 m, or 0.4 m above the normal water level in the Reservoir. An outlet channel sloping at about 1% will discharge flows to the Reservoir at Elev. 370.2 m.

A typical section of the spillway is shown on Figure 22 (Section B-B'). Excavated slopes in intact bedrock will be approximately 0.1H:1.0V. In frost shattered bedrock, flatter slopes of 1H:1V will be built. Overburden slopes will be cut at 4H:1V vertical for long term stability considerations. No erosion protection is required on these cuts slopes, except within the 0.6 m flow depth.

Side berms will be constructed to crest elevation 370.8 m for the outlet channel near the Reservoir. This is equivalent to 1.5 times the design wave height above the normal water level in the Reservoir. It is also above the maximum water level in the Reservoir predicted under extreme flood conditions. The approximate extent of rip rap protection is shown on Figure 22.

6.4.4 Construction Considerations

6.4.4.1 Diversion of Upstream Watershed

The spillway is located in a valley where ephemeral flows are produced from the upstream watershed. To prevent erosion of the spillway side slopes, the flows will be directed to an inlet channel where erosion protection will be provided (Figure 22). A series of small culverts will be provided under the service road to prevent icing and subsequent loss of trafficability.

A small deflection berm will be provided at the crest of the cut slope on the uphill (south and west) side of the spillway to prevent run-off from eroding the slope. No ditching or berming is required on the opposite side of the spillway because the collecting watershed is not large.

6.4.4.2 Excavation, Erosion Protection and Alignment

Where the design flow depth of 0.6 m is not entirely within intact bedrock, erosion protection (consisting of rip rap placed on a layer of bedding material) will be provided. Rip rap slopes within the flow depth should be 4H:1V. If overburden (soil) is exposed within the flow depth, a sand and gravel filter should be placed beneath the rip rap bedding to prevent the migration of fine materials through the bedding and rip rap.

There is some chance that ground ice may be encountered during the construction of the outflow channel. If so, the ground ice will be sub-excavated and the void will be backfilled with granular fill covered with bedding and rip rap. Alternatively, the alignment could be adjusted.

If poor rock conditions are identified in the field, the slopes will be flattened during excavation to ensure long term stability.

6.5 West Twin Lake Outlet Channel

For closure, the existing control structure at the outlet of the Polishing Pond will be removed. It will be replaced with an open channel and overflow weir structure which will passively control the Reservoir water level and minimize future maintenance requirements.

The channel will have a base width of 7 m to match the nominal width of Twin Lakes Creek receiving flows downstream. To create the channel, the existing outlet control structure will be removed and the opening in the existing embankment will be lowered and widened. A 0.3 m wide reinforced concrete cut-off wall with invert Elev. 370.2 m will be provided in the channel to control the normal water level in the Reservoir (West Twin Lake). Erosion protection will be provided in the channel upstream and downstream of the wall to prevent scour of the natural or fill materials adjacent to the channel.

Flood routing analysis indicated that the flow depth in the design storm would be 0.6 m. Consequently, the design maximum water level (MWL) in the Reservoir will be Elev. 370.8 m.

6.5.1 Design Criteria

The outlet control structure has been deigned to safely convey flows from the routed 24-hour probable maximum precipitation (PMP) storm event.

The normal water level (NWL) in the Reservoir will be controlled at 370.2 m. The MWL will be 370.8 m.

Erosion protection will be provided around the Reservoir where required to prevent the erosion of tailings. The top elevation of the erosion protection will be the NWL plus 1.5 times the significant wave height generated by a 100-year return wind velocity. Table 9 notes that the design wave height is 0.4 m so the design criteria would require erosion protection to an elevation of 370.8 m.

The outlet has been designed to require minimal maintenance. Routine surveillance and maintenance are required as set out in Section 6.5.6.

6.5.2 Design Considerations

6.5.2.1 Hydraulic Considerations

The GAWSER model (Guelph All-Weather Sequential Event Runoff model) was used for the hydrological simulation of the Nanisivik Mine tailings system. This model is widely used in Canada for various types of hydrological analyses. The following design conditions were used for the hydrological modelling:

- The Surface Cell watershed area is 127 ha. The Reservoir watershed area is 173 ha (excluding the Surface Cell watershed). The total watershed area of the outflow control structure is 300 ha (127 ha plus 173 ha).
- The elevation-discharge relationship for the Surface Cell was developed from the hydraulic calculations of flow in the Surface Cell spillway inlet. Hydraulic roughness (Manning's) coefficient of 0.035 for the spillway channel was assumed.
- The elevation-discharge relationship for the Reservoir was developed from the hydraulic calculations of flow for the West Twin Lake (Reservoir) outlet control spillway, also using a Manning's roughness of 0.035.
- The elevation-storage relationships were developed based on the Surface Cell grading plan for closure and the Reservoir grading plan for closure.
- The watershed properties were set to reflect the frozen ground conditions and simulate high surface runoff. The resulting runoff coefficient for the PMP event was 0.94.

From a hydraulic perspective, both spillway inlets will act as a broad crested weirs.

6.5.2.2 Design Precipitation Event

The extreme precipitation conditions at Nanisivik as derived from Golder (2004 b,c,d) are summarized as follows:

- The extreme daily rainfall and snowmelt amounts at Nanisivik are comparable. The daily rainfall PMP event is estimated to be approximately 140 mm. The daily snowmelt PMP event is estimated to be 155 mm. For comparison, a daily PMP event in Northern Ontario is approximately 500 mm to 700 mm.
- The amount of rainfall and snowmelt at the Nanisivik Mine is small. Therefore, a comparatively small spillway is required at the Nanisivik Mine, even to convey extreme floods.

For the mine closure, the Reservoir outflow channel was designed to convey a flood resulting from the PMP storm event (140 mm in 24 hours). The snowmelt distribution is more uniform than the rainfall distribution. Consequently, the PMP rain storm may produce greater peak flows than the snowmelt event. Hydraulic characteristics corresponding to a 100-year return period are also provided in the report for the purpose of comparison.

6.5.2.3 Design Flow Conditions

The following hydraulic conditions will be observed at the West Twin Outlet Channel under the PMP conditions (Table 11):

- The calculated peak discharge from the Reservoir is approximately 6.5 m^3/s;
- The calculated peak water depth at the Reservoir Outlet is approximately 0.6 m;. and
- The calculated peak velocity at the Reservoir Outflow Structure is 1.5 m/s.

For comparison, under the 100-yr storm conditions (Table 11):

- The calculated peak discharge over the outflow structure is estimated to be 1.7 m^3/s;
- The calculated peak water depth is estimated to be approximately 0.2 m; and
- The calculated peak flow velocity is estimated to be approximately 1.5 m/s.

6.5.2.4 Erosion Protection

The erosion protection against the flowing water was sized based on the calculated peak flow velocity and channel side slopes (Smith, 1995). As mentioned previously, the calculated peak velocity at the West Twin Outlet Channel is 1.5 m/s. To protect against these flows, the median rip rap (D_{50}) required would be less than 130 mm. For continuity with proximal toe protection, it is recommended that the channel be protected from erosion with rip rap having an average diameter D_{50} of 200 mm. The material specifications for the rip rap are included in Table 10.

6.5.3 Geotechnical Considerations

6.5.3.1 Subsurface Conditions

Information regarding the subsurface conditions at the outlet area was obtained from:

- The original design drawings for the existing Reservoir outlet structure (Kilborn, 1977) indicate bedrock at about Elev. 369.4 m;
- Borehole BGC03-23, located about 25 m from the spillway alignment, in which bedrock was encountered at Elev. 368.8 m, underlying sand and gravel sized till (inferred to be embankment fill based on the Kilborn drawing); and
- Borehole BGC03-24, located about 100 m southeast of BGC03-23. In this borehole, bedrock was encountered at Elev. 367.7 m. The upper 0.6 m of the bedrock is noted as being highly fractured.

Borehole logs for BGC03-23 and BGC03-24 are included in Appendix VI.

6.5.3.2 Configuration of Existing Outlet Structure

The existing outlet structure (Figure 23) consists of a weir and an embankment which together close the former creek section. Based on the available subsurface data, it is inferred that the existing concrete control structure is founded on a bedrock foundation on the original shoreline (which forms the left abutment of the West Twin Outlet Control Embankment).

The new outlet structure will be formed by excavating into natural ground at the north side of the existing outlet location to take advantage of the natural ground and presumably higher bedrock elevation. For closure, it is generally preferable to excavate hydraulic channels into natural ground rather than embankment fill.

Construction details of the existing West Twin Outlet Control Embankment are not certain. As-built drawings show an upstream membrane in the Diversion Dike (Kilborn, 1977) but it is not clear if this was extended into the Outlet Control Embankment (Figure 23). After construction, a cut-off trench was excavated on the upstream face and backfilled with marine clay, presumably to reduce seepage. Since that time, any potential seepage amount has been low enough to maintain a water level of approximately 371.2 to 371.5 m in the polishing pond (or about 2 m of head across the structure). In addition, visual inspections since 1998 have not identified and seepage on the downstream side of the embankment. Lowering the water level to 370.2 m at closure will further reduce the potential for seepage. Based on anecdotal information from mine site staff, no water would be impinged on the embankment at a water level of 370.2 m. Therefore, no additional seepage mitigation measures are proposed for the existing embankment. Bedrock below the overflow weir will be sealed and the concrete wall will tie into the existing embankment material.

6.5.4 Design Details

The design of the West Twin Outlet Channel is shown on Figure 24. The design incorporates the following elements:

- A 0.3 m wide reinforced concrete cutoff wall will provide a fixed non-erodible invert and prevent seepage through the rip rap and bedding zones at the sides of the channel. Below Elev. 369.0 m. To limit seepage under the wall, the bedrock foundation for the control structure will be grouted. Except under extreme low precipitation conditions, this will prevent the Reservoir water level from dropping below the NWL. As a preliminary basis, the grouting would extend to a depth of 1.5 m below the wall founding level. As a contingency, the depth of grouting could be increased in localized areas, if required.
- The wall will extend 5 m laterally into the adjacent embankment fill and into the natural ground abutment to cut-off the rip rap and bedding layers and increase the seepage flow path around the ends of the wall.
- Upstream of the concrete cut-off wall, rip rap erosion protection will be provided on the side slopes up to Elev. 370.8 m. This is 0.6 m above the invert, which corresponds to the

NWL in the Reservoir plus 1.5 times the significant wave height. The inlet channel flares to meet the Reservoir discharge flow.

- Erosion protection requirements are set out on Table 12. Downstream of the wall, rip rap erosion protection will be provided to 0.3 m above the channel floor to protect up to the flow depth calculated under PMP storm conditions. To prevent possible scour of the natural (unprotected) creek bed, the sloping outlet channel will terminate in a 3 m long stilling basin with elevation 369.0 m (approx.) to force a hydraulic jump on the constructed erosion protection. These elevations may be adjusted following the required survey of the creek.
- Rip rap will be placed against the wall to smooth the flow lines and to protect the concrete surfaces that would otherwise be exposed to the elements.

6.5.5 Construction Considerations

Construction quantities are summarized in Table 13.

The West Twin Outlet Channel has been located based on limited available information. The detailed location and construction will be verified through a test pitting investigation prior to construction. Field adjustments could be made to the position based on the bedrock foundation conditions encountered (including the depth to and extent of frost shattering of the bedrock).

The concrete cut-off wall will be founded on competent bedrock. Excavation to the bedrock surface is expected to be required to provide a suitable foundation. Based on the available borehole information (Appendix VI), it is anticipated that this excavation will extend down to approximate elevation 368 to 369 m. Deeper local excavation may be required. The wall foundation level should be approved in the field by the Field Representative.

It has been assumed that the water level in the Reservoir/Polishing Pond will be drawn down to Elev. 368 m at the time of construction. Normal local dewatering measures may be required in the excavation for the concrete wall.

Bedrock foundation grouting will extend to a depth of about 1.5 m below the wall prior to pouring the concrete. This will locally tighten the bedrock and reduce seepage through the near surface zone. The actual depth of the grouting will be adjusted based on observations and response tests done in the grout holes during drilling.

The cut-off wall will be doweled into bedrock. The actual depth of the dowels will be based on the conditions encountered in the field and approved by the Field Representative. In general, dowels will extend a minimum of 0.5 m into intact bedrock (i.e. non frost shattered).

For the efficient hydraulic operation of the West Twin Outlet Channel, it is important that the creek downstream of the structure flows relatively freely under flood conditions. A pre-construction survey of Twin Lakes Creek is required to verify this assumption.

6.5.6 Maintenance

Visual observation of the Outlet Control Structure should be included as part of the routine surveillance program. Surveillance items should include inspection for the presence of upstream or downstream blockages and the condition of the concrete wall and rip rap.

6.6 East Twin Diversion Channel Upgrade

When the mine was constructed, the flow from East Twin Lake was diverted from its natural course, which became part of the Polishing Pond. A Diversion Dike was constructed to redirect the flow through a constructed Diversion Channel (Figure 25). Based on hydrologic considerations (Golder, 2004 b,c,d), it was determined that the diversion should be maintained after closure.

There is evidence of erosion on the upstream face of the East Twin Lake Diversion Dike and channel in the area indicated on Figure 25. Should the dam be eroded away, runoff from the large East Twin Lake watershed would report to the Reservoir, potentially eroding tailings or compromising the integrity of West Twin Dike and/or the West Twin Outlet Channel.

There is also evidence of erosion within the channel itself. Such erosion is of little potential consequence, except for sediment discharge to the channel.

6.6.1 Design Criteria

Measures are required to protect the East Twin Diversion Dike against erosion under flows from the PMP storm event.

It is not practical or necessary to line the entire diversion channel with erosion protection because the consequences of erosion are not severe. Any channel erosion would not be classified as a catastrophic consequence.

6.6.2 Design Considerations

The East Twin Lake drainage area (34.6 km^2) is much greater than that of the Reservoir (West Twin Lake) drainage area (3 km^2). The diversion will remain in place for closure. Erosion noted in the channel is not of consequence, but the dam itself will be upgraded.

The peak flows from the East Twin Lake watershed were evaluated in the hydrological study (Golder 2004 b,c,d). The peak flows from the East Twin Lake watershed were estimated to be approximately 13.2 m^3/s for the 100-yr return storm and approximately 50 m^3/s under the PMP conditions.

Based on the design drawing (Kilborn, 1977), the diversion channel base is approximately 5.5 m wide and it has a base gradient of 0.7%.

Design flows conditions are summarized on Table 14. The calculated flow depth under the PMP conditions is 2.2 m and the estimated flow velocity is 2.9 m/s.

6.6.3 Design Details

6.6.3.1 Dike Protection Measures

Considering the design flow conditions described in Section 6.6.2, it is recommended that the Diversion Dike be protected with rip rap with mean particle size of 300 mm. Construction volumes for the East Twin Lake Diversion Channel Upgrade are provided in Table 15.

A radius is provided to ensure uniform flow near the dam. Protection beyond the ends of the dam is required. As a preliminary basis, a 20 m length is assumed. This will have to be confirmed in the field.

6.6.3.2 Repair of Existing Channel Erosion

Where signs of erosion are evident on the outside (west) Diversion Channel slope and east slope of the groin at the channel outlet, it is recommended that the side slopes be flattened to 3H:1V and the existing bank protection be restored. This assumes the median particle diameter on the existing banks along the channel is 100 mm and will offer protection under modest storm flows.

If during routine surveillance, erosion of the channel slope is evident, maintenance will be required.

6.7 Discharge Water Quality

The outflow from the Polishing Pond (Station 159-4) is a defined “Final Discharge Point” in the Water Licence to which the effluent discharge criteria apply. This location is included in the Reclamation and Closure Monitoring Plan and will continue to be sampled throughout the closure monitoring period.

The outflow from the Polishing Pond immediately mixes with outflow from East Twin Lake, which is unaffected by mine activities. The combined water, Twin Lakes Creek, flows approximately 8 km to Strathcona Sound, receiving runoff from natural sulphide outcrops, the

West Adit area, the town site, the Industrial Complex and the concentrate haulage road along the flow path. It is well documented that Twin Lakes Creek typically carries high concentrations and high loads of heavy metals introduced from these source areas downstream of the WTDA. Further, this was the case both pre-mining and during mine operations.

A study completed prior to mining by B.C. Research (1975) states that Twin Lakes Creek does not support fish due to the poor habitat characteristics.

A water quality projection has been developed to identify the anticipated post-reclamation water quality at the outflow from the Polishing Pond.

The intent of the reclamation measures for the Surface Cell, the Test Cell area, the Reservoir and the Polishing Pond is, in part, to protect the environment. Water quality at the outflow from the Polishing Pond is one of the means of assessing the level of environmental protection that is being provided. Therefore, the water quality projection provided herein is of interest in providing an indication of the anticipated success of the reclamation measures in this regard.

The Water Quality Projection uses the following sources of information:

- A site specific water balance developed for this project by Golder (2004d), which is included in Appendix I;
- Projections of water volumes expelled over time from the Surface Cell and Test Cell taliks developed for this project by BGC;
- 2002 and 2003 groundwater quality analyses from the Surface Cell; and
- The record of water quality in East Twin Lake and other "unaffected" locations.

The Water Quality Projection focuses on total zinc as the parameter of interest. This is because experience at this site demonstrates that zinc is the most mobile metal and that control of zinc will provide control of other metals. For example, kinetic testing conducted on Nanisivik tailings, as described in Section 4 of this report, specifically demonstrated that zinc was mobilized at an order of magnitude greater than other metals. This approach is consistent with other base metal mines.

The potential for diffusive transport of contaminants from submerged tailings into the Reservoir Pond was considered in the context of the current state-of-the-art technical understanding of this topic. The review of literature on diffusive transport under water cover is documented by Lorax Environmental in Appendix VII. The review indicates that, in this case, the potential for diffusive transport that would have an observable effect on pond water quality is extremely low. References for current research in this topic are provided in Appendix VII.

6.7.1 Inputs to the Water Quality Projection

6.7.1.1 Natural Runoff

Quantity

The development of a water balance for the Polishing Pond, in its proposed reclaimed state, is described by Golder Associates in Appendix I. In summary, the water balance provides estimates for average monthly flows at the outflow of the Polishing Pond, which includes consideration of rainfall, snowfall, snow melt, evaporation and runoff coefficient.

For the purposes of the Water Quality Projection, flows are considered only for a six-month period from May through October. The average monthly flows used, in m^3/s, are as follows:

- May: 0.01
- June: 0.10
- July: 0.04
- August: 0.02
- Sept.: 0.01
- October: 0.005
- Annual: 0.015

Quality

The concentration of total zinc applied to runoff through the reclaimed WTDA is 0.056 mg/L. This is the average concentration of total zinc reported for stations NML-23, East Twin Lake, and NML-24, tributary to East Twin Lake, from 1996 to 2001, which are unaffected by mine activities. This concentration is greater than the federal guideline for the protection of Fresh Water Aquatic Life, 0.03 mg/L, which is attributed to the general abundance of natural sulphide mineralization in the area.

A listing of the water quality record for total zinc is provided in Appendix VII.

6.7.1.2 Water Expelled During Freezeback of Taliks

Quantity

The Talik Report describes the anticipated progression of the freezeback of the two taliks under the Surface Cell and Test Cell areas. As the ground freezes, the expansion of water into ice results in increased pore pressures that can include migration of water, if a hydraulic gradient and flowpath are present.

Water within the Surface Cell talik may migrate into the Reservoir Pond if a hydraulic connection is developed. This possibility is not anticipated but is applied to the Water Quality Projection to provide an additional degree of conservatism to the estimation. Water from the Test Cell talik is anticipated to migrate into the Reservoir pond through a thawed zone in the tailings base of the Test Cell dike. This anticipated occurrence has been applied in the Water Quality Projection.

The rate of water migration from a freezing talik decreases with time as a result of the progressive reduction in unfrozen water content as water freezes to ice. The rate of water migration from the Surface Cell and Test Cell taliks has been estimated based on an average volumetric water content of 40% and an expulsion rate of 9%, as listed in Appendix VII.

The volume of water, in m^3, anticipated to be expelled from the Surface Cell talik is estimated to be:

- Years 1 – 5: 32,400 m^3
- Years 5 – 10: 12,960 m^3
- Years 10 – 15: 11,160 m^3
- Years 15 – 20: 10,440 m^3
- Years 20 – 25: 2,844 m^3
- Years 25 – 30: 2,196 m^3

The volume of water, in m^3, anticipated to be expelled from the Test Cell talik is estimated to be:

- Years 1 – 5: 18,000 m^3
- Years 5 – 10: 9,900 m^3
- Years 10 – 15: 3,600 m^3
- Years 15 – 20: 0 m^3

It should be noted that the difference between the freezeback time in the Surface Cell Talik (25-30 years) and the Test Cell Talik (approximately 15 years) is attributed to the relative difference between the size of the two taliks.

The Water Quality Projection recognizes that these small inflows may enter into the Reservoir pond on a continuous basis. However, winter inflows are assumed to be stored as ice and released from the pond during the 6-month flow season. For the Water Quality Projection, the 6-month winter inflows are considered to be released according to the normal monthly distribution of snowmelt.

Quality

The initial concentration of zinc within the talik, 0.025 mg/L, is the higher of two groundwater samples collected from the Surface Cell talik in August 2003 during the Phase 3 Environmental Site Assessment. Other groundwater samples were analysed, but these were not used for the reasons described in Section 4.1.4. The concentration used is the concentration of dissolved zinc since total metal analyses of groundwater samples are generally not considered valid due to agitation in the well during purging and sampling that can bring suspended sediment from around the well into the sample.

Cryoconcentration is a process where the concentrations of parameters in the pore water increase over time as a talik freezes. This is a result of the preferential migration of solutes to the water rather than ice. The ice that forms within a talik is consequently likely to contain lower concentrations of solutes than the water. The exact mechanisms and rates of how cryoconcentration occurs will vary according to many site-specific circumstances. For example, the rate of frost penetration, which will vary with time and soil conditions, will affect the rate of water freezing and the rate of cryoconcentration into water.

For this Water Quality Projection, cryoconcentration was included based on an approach, suggested as being conservative in nature, wherein fully 90% of the zinc that was present in the water at the beginning of each 5-year time step was retained in the water. That is, only 10% of the zinc is assumed to be captured into ice.

This approach resulted in the 5-year time step concentrations for the Surface Cell talik listed below:

- Years 0 – 5: 0.025 mg/L
- Years 5 – 10: 0.033 mg/L
- Years 10 – 15: 0.051 mg/L
- Years 15 – 20: 0.141 mg/L
- Years 20 –25: 0.291 mg/L

This approach resulted in the 5-year time step concentrations for the Test Cell talik listed below:

- Years 0 – 5: 0.025 mg/L
- Years 5 – 10: 0.041 mg/L
- Years 10 – 15: 0.101 mg/L

6.7.1.3 Perimeter of the Reservoir Pond

Quantity

The perimeter of the Reservoir pond is proposed to be reclaimed in three zones, as described in Section 6.2.4.

Zone 1 is generally the southern and eastern sides of the pond where any residual tailings will be relocated to at least 1 m below the ultimate water level. This zone does not receive any further consideration in the Water Quality Projection since the water cover will protect the tailings from oxidation and physical effects.

Zones 2 and 3 are the sides of the pond located directly below the West Twin Dike and the Test Cell Dike, respectively. In these areas, the pond perimeter represents a transition from water cover to thermal barrier cover. Over the long term (possibly decades), permafrost is anticipated to aggrade to, and just under, the shoreline such that the tailings in the transition zone will be permanently isolated from the environment. Nonetheless, reclamation measures are included to account for the interim timeframe to promote the aggradation of permafrost and to ensure that tailings in the transition zone do not affect the environment while they freeze.

These reclamation measures will reduce any contact of tailings with the environment to a narrow “strip” located immediately above the water level. These tailings will lie beneath the 1.25 m thermal barrier cover and rip rap rock but may take some time to freeze because of the heat effects of the adjacent pond.

The Water Quality Projection includes recognition that, in the interim period while permafrost aggrades completely into the transition area tailings, it is possible that some of the runoff water that passes through the base of the cover materials in the area immediately above Zones 2 and 3 may contact the tailings. The quantity of water that could contact the tailings is restricted by:

- The small “catchment” area above these zones, which restricts the quantity of water that could possibly be directed in this direction;
- The grade of the “catchment areas” which is steep enough to promote surface flow of runoff water; and
- The thermal barrier cover materials, which will promote freezing of the underlying tailings and which will restrict water contact with tailings to a small portion of the runoff flow that lies at the base of the 1.25 m thickness.

The Water Balance Projection incorporates average precipitation and snow melt during the 6-month flow season over catchment areas of 2 ha for each of Zones 2 and 3 with a contact factor of 0.15.

Quality

Several sources of information were available to assess the possible interim influence of tailings in the transition zone. These include:

- Acid rock drainage characterization static and testing reported by Lorax Environmental (Lorax 2000);
- Humidity cell kinetic testing for acid rock drainage potential reported by Lorax (Lorax 2001);
- Column leach testing of Nanisivik tailings reported by Dr. Elberling of the University of Copenhagen (Elberling et al. 2002); and
- A review of the above, in the context of this study, by Lorax as provided in Appendix VII.

All of the above sources of information, plus site specific knowledge suggests the following observations that are incorporated into the Water Quality Projection:

- Tailings in the transition zone will be undergoing progressive permafrost aggradation and this, combined with the generally cold climate, will maintain a consistent or decreasing environmental influence during the interim freezing timeframe;
- Neutral-pH data suggested by Lorax (Appendix VII) is appropriate for use because of the following: tailings pH was neutral throughout the humidity cell test, permafrost aggradation and the generally cold climate will inhibit oxidation and tailings that have been locally exposed on surface for more than a decade remain at neutral pH; and
- The influence of tailings in the transition zone would be expected to vary throughout the summer season from a minimum during the near-frozen freshet to a maximum during the low flow, maximum thaw depth conditions at the end of the summer season.

Therefore, the site specific conditions regarding climate, the operational experience with managing tailings exposed on surface and the results of site specific testwork suggest the following influence for total zinc released to water that contacts tailings in the transition areas, that has been incorporated into the Water Quality Projection:

- May 0.3 mg/L
- June 0.3 mg/L
- July: 6.0 mg/L
- August 12.0 mg/L
- September 18.0 mg/L
- October 24.0 mg/L

6.7.2 Results

The Water Quality Projection is in the form of a spreadsheet that can provide projections for the Polishing Pond outflow based on various input values. Table 16 shows summary results from the Water Quality Projection for input values that are based on the inputs described above. The summary results are presented in 5-year increments to capture the influence of changing rates and concentrations of porewater expelled from the Surface Cell and Test Cell taliks. The detailed results of the Water Quality Projection for each 5-year time step are provided in Appendix VII.

The following observations are drawn from the results summarized in Table 16:

1. Natural runoff is projected to represent a dominant controlling factor (80% of Zn load) for Polishing Pond outflow.
2. Porewater expulsion is projected to represent a small contributing factor (less than 1% of Zn load) to Polishing Pond outflow.
3. Runoff over the Reservoir Perimeter during the interim period of permafrost aggradation is projected to represent a moderate contributing factor (20% of Zn load) to Polishing Pond outflow.

4. The quality of Polishing Pond outflow is not anticipated to vary on an annual basis.
5. The volume of water passing through the Polishing Pond is projected to decrease slightly over time due to the decreasing rate of porewater expulsion.

In summary, the annual quality of water passing through the Polishing Pond (0.07 mg/L Zn) is projected to remain similar to natural conditions (0.056 mg/L).

7.0 CONSTRUCTION PLAN

7.1 Construction Materials

7.1.1 Quantities

At closure, shale cover will be required for the following facilities around the WTDA:
- Surface Cell tailings and crest of West Twin Dike.
- Downstream face of West Twin Dike (including completion of shale cover for consistent grade over the currently exposed beaches).
- Tailings at the toe of West Twin Dike (including transition zone at shoreline).
- Test Cell tailings and Test Cell Dike (including transition zone at shoreline).

In addition to the shale cover at these locations, a top layer of sand and gravel armouring will also be required. This material will be obtained from the Twin Lakes sand and gravel deposit, located between West Twin and East Twin Lakes. The total in-place volume of shale required for the reclamation covers at the WTDA is estimated to be about 590,000 m^3. The total in-place volume of sand and gravel armouring required at the WTDA is estimated to be about 139,500 m^3.

7.1.2 Borrow Development Areas

The quantities of shale in each quarry were estimated on the basis of the exposures of shale in the existing working faces, supplemented by several shallow drill holes to confirm the depth of cover and lateral extent of the deposit. More drilling will be carried out during quarrying operations to help delineate the final quarry limits. The quarry development plans provide for 1,350,000 m^3 (in-situ) of shale cover material, which is about 1.5 times the estimated volume required for all of the reclamation covers around the Nanisivik Mine. If the cover quantity needs to be increased, additional volumes are available from the other quarries at the mine.

The Mt. Fuji Quarry development plan has been designed to provide approximately 350,000 m^3 (in-situ) of shale cover material. This volume was estimated from visual reconnaissance, available published geological information for the Nanisivik area and survey data of the quarry area. The Mt. Fuji Quarry is considered to be the most viable source of shale cover material for the WTDA, due to it’s close proximity.

The West Twin Quarry development plan has been designed to provide approximately 150,000 m^3 (in-situ) of shale cover material. This volume was determined through visual reconnaissance, available published geological information for the Nanisivik area, geological information from borehole data and survey data.

The East Twin Quarry is designed to provide approximately 750,000 m^3 (in-situ) of shale cover material. This volume was determined through visual reconnaissance, geological information of Nanisivik, borehole data, and survey data.

Approximately 139,500 m^3 of armouring sand and gravel material are required for surface reclamation covers at the WTDA. The Twin Lakes sand and gravel deposit contains a sufficient volume of protective armour material to satisfy the requirements of the surface reclamation covers. This was determined utilizing topographic information and available borehole data. Three boreholes were drilled in the deposit as part of the hydraulic connectivity assessment by Tordon (1998). The location of the boreholes is illustrated on Figure 8. These boreholes (TC-22, TC-23, TC-24) ranged in depth from 10.2 m to 13.3 m and the sand and gravel material was encountered throughout much of the borehole profile. On the basis of this work, the deposit it estimated to contain a minimum thickness of 10 m of sand and gravel. Due to its aerial extent, it is not necessary to develop the entire thickness of the deposit to recover the required volumes. The required volume of material will be obtained by excavating into the top 2 m of the deposit. The bottom of the quarry will remain at least 2 m above the level of West Twin Lake (elevation 370 m) to ensure that hydraulic connectivity does not develop between the two water bodies.

7.2 Construction Outline

At the time of preparation of this report, CanZinco is exploring three options with respect to construction approaches, as summarized below:

1) Use the available Nanisivik Mine fleet of equipment and manpower to undertake the reclamation work over two construction seasons.
2) Contract a fleet of equipment from outside contractors and operators to undertake the reclamation work in one construction season (or possibly two).
3) Transfer the mine fleet of equipment to the Government of Nunavut (GN) and have the reclamation work undertaken as a training and employment exercise over several construction seasons.

No final decision on the contracting approach or duration of operations has yet been made. As a result of this uncertainty, no detailed schedule for reclamation works at the WTDA has been prepared. Within the monitoring requirements noted in other design reports, the reclamation period has been assumed to occur over two years. This assumption is a reasonable one for the magnitude of the reclamation work required.

Construction of the West Twin Dike spillway will be somewhat dependent upon placement of the cover in the Surface Cell, coupled with considerations with regard to control of existing surface water in the existing surface water drainage channel. The inlet to the new spillway would not be operational until the Surface Cell cover is completed but portions downstream from the inlet could be constructed before the inlet is completed. The discharge portion of the spillway would be constructed when the Reservoir level is lowered.

In order to undertake a number of the reclamation tasks for the WTDA, the Reservoir water level needs to be drained down to approximately Elevation 368 m. Given that this level is lower than the natural channel grade in Twin Lakes Creek, pumps will be required to discharge water from both the Reservoir and the Polishing Pond. The pumping and water level lowering rates will have to be synchronized to ensure no major head difference develops across the access road dike. Following lowering of the Reservoir level, the following reclamation tasks will need to be undertaken:

- Relocation of any exposed tailings around the shoreline (Zone 1) to lower than Elev. 369.2 m.
- Lowering of any high points in the exposed sub-aqueous tailings located in the Reservoir.
- Placement of thermal cover, bedding and rip rap at the toes of the West Twin Dike and the Test Cell Dike.
- Breaching of the baffle dike.
- Construction of the discharge end of the West Twin Dike spillway should also be undertaken.

As noted later in the contingency planning section, in the unlikely event that Reservoir water quality does not meet discharge criteria, it may be necessary to both retain and treat water within the Reservoir and possibly the polishing pond. Therefore, both the access road causeway and the current outlet discharge structure will remain in-place until water quality is within discharge limits and appears stable. Then the water level will be reduced once again by pumping and the following three tasks will be undertaken:

- Breaching of the access road causeway and removal of the culverts.
- Removal of the existing outlet discharge structure.
- Construction of the new outlet channel and overflow weir. The weir construction requires the use of concrete and grout which are both typically undertaken during warm weather conditions.

Since these three tasks may be completed several years later (when water quality trends are stable), it is likely that construction materials such as rip rap and bedding may be stockpiled near the outlet during the main construction work on-going at the WTDA.

The proposed rip rap rehabilitation of the upstream face of the East Twin Diversion Dike and Channel could be undertaken at any time when appropriate material is available and access is possible.

7.3 Quality Assurance/ Quality Control Requirements

A quality assurance/quality control (QA/QC) program is proposed to provide a means to ensure the reclamation works are constructed to the design specifications and intent. At the current time, the proposed contractual details on who will be undertaking the reclamation work, has not been determined. As a result, the exact components of the QA/QC program have not been finalized. The following sections outline the general components of the QA/QC program to be used during the construction of the required reclamation works at the WTDA.

7.3.1 General

It is assumed that a qualified technical representative responsible for the execution of QA and QC program will include a suitably-trained Field Representative working under the direction of a professional geotechnical engineer. The Field Representative will be on-site daily throughout the construction. This continuous attendance is a requirement in order to produce the as-built report required later. The QA/QC program will be the responsibility of the Field Representative, in concert with the Site Supervisor. The Field Representative will ensure that the constructed works conform to contract documents (construction drawings) and the design intent of the reclamation works described in this document. The Field Representative will have the authority to reject any substandard work and order the contractor to redo the work such that it meets the requirements and the intent of the contract and drawings. The Field Representative will provide daily inspection reports summarizing work undertaken, methodology used, manpower and equipment utilized and written confirmation regarding field decisions made and design alterations permitted.

The Site Supervisor, representing the contractor, will be required to maintain accurate records of all construction operations and shall provide the Field Representative with a copy of the daily record at the end of each shift. The following information will be recorded on the fill placement summary sheets:

- Location and elevation at the start and end of all excavation operations, fill placement, for each major individual area (e.g. Surface Cell) for each shift;
- Tailings volume relocated and concrete/ grout placed;
- Quantity of concrete/ grout placed;
- Estimated quantity of materials placed during the shift.
- Location and elevation of material sources from the borrow pits placed in each shift.
- Quality control results on fill, concrete and grout materials.
- Number of workers and equipment engaged during the shift.
- Unusual occurrences during reclamation operations such as unstable soil conditions, extreme precipitation events or variations in fill quality.

These reports will supplement the daily reports to be filled out by the Field Representative.

As such, a geotechnical and materials testing laboratory will be required on-site during the reclamation operations.

7.3.2 Fill Placement and QC Testing

Placement of fill materials for the various layers of the cover, toe protection and spillway lining shall be subject to the following conditions:

- No fill material shall be placed until the substrate have been appropriately prepared and graded (if required) and has been approved by the Field Representative.
- Fill materials shall be placed in accordance with lines, slopes, grades and elevations as provided on final construction drawings.
- The placement, handling, spreading and compaction of the fill materials shall be performed in such a manner that the fill material is free of particle segregation, lumps, sizeable lenses, pockets and layers of material that are substantially different in gradation and texture from surrounding materials.
- Any fill material not meeting the noted requirements will be removed, remixed, blended or otherwise reworked to meet the specified requirements.
- Fill materials shall be placed and spread in continuous and approximately horizontal layers of uniform thickness.
- Hauling and placement equipment shall be routed over the fill surface such that they do not follow the same path but that the equipment track loads are spread consistently across the upper surface.

The compaction process will be performance based and will consist of a number of passes from the proposed construction equipment (e.g., dozer, loaded hauled trucks and possibly loaded water trucks, if required). Other criteria for the compaction operation, subject to verification in the test fill, will consist of the following general guidelines:

- Compaction of each layer of fill shall proceed in a systematic and continuous manner so that each portion of the layer receives an equal amount of compactive effort.
- The method of changing direction of the equipment shall result in uniform compaction.
- Overlap should occur between the various passes of the construction equipment.
- It is expected that the upper surface will be free from ruts. If any are noted, re-levelling and/or additional passes will be required.
- Any oversized particle sizes will be removed from the fill before proceeding with compactive effort.

Alternatively, no compaction specification, other than a firm and tight final surface, is required for the Twin Lakes sand and gravel. Table 17 provides recommended testing frequencies for the shale cover and Twin Lakes sand and gravel materials for grain size and moisture content determinations. Additional testing requirements for the materials in the spillway and toe erosion protection elements will be detailed later during the preparation of final drawings and technical specifications.

Test pits will be excavated by the contractor in locations determined by the Field Representative. Spot thickness of the reclamation cover, bedding and rip rap layers will be measured in these test pits. This information will be recorded by the Field Representative to ensure the minimum thickness requirements for each material are being met.

7.3.3 Excavation

Prior to starting work, the Field Representative will request from the contractor details of the proposed excavation methods, including, blasting, ripping equipment, method of excavation, details and locations of weigh scales, schedules and sequence of operations to complete the work.

7.3.3.1 Soil Excavation

Soil excavation is required in the spillway and outlet channel. In general, excavated soil may be used as common fill throughout the WTDA, to fill in low spots and general site re-contouring, possibly including access roads

Soil excavation may involve ripping and blasting to loosen and excavate permafrost-affected soils. Materials are expected to include a variety of soils of glacial origin, including till, sand and gravel with cobble and boulder fragments as well. Excavations within the Surface Cell and Reservoir may encounter a minor amount of tailings. At the outlet end of the spillway, massive ground ice has been observed. Close to the bedrock surface, detached blocks of weathered bedrock may be included. There is no topsoil component that needs to be segregated and stockpiled.

Within the spillway area, soil excavation will be almost entirely in the dry, except for some wet conditions at the outlet (which will be undertaken when the Reservoir is lowered). Unsuitable materials, such as massive ice lenses will be sub-excavated and backfilled with granular fill, then covered with bedding and rip rap. The excavated ice-rich soils, which are not suitable as backfill elsewhere, will be placed in the Reservoir to thaw or in low areas of the tailings cover. If placed in the Reservoir, these materials will thaw and any ice-entrained sediments will settle into the Reservoir. The volume of unsuitable materials from the required excavations is expected to be minor, and no spoil disposal area has been designated.

Any soil excavation undertaken within, or adjacent to, existing water courses will need to be cognizant of water management issues to ensure that flows can either be retained within the system (upstream of an excavation) or be passed around an excavation. It will be critical to ensure that no high sediment water is discharged to the environment. Proper pumping procedures and possibly settling ponds, rockfill check dams and silt curtains may be required to control construction-affected surface water.

The excavation requirements and limits will be shown on the construction drawings, based on the results of subsurface explorations. The Field Representative will examine the conditions exposed at the required excavation lines and, if the conditions are deemed to be unacceptable, will require excavation to be continued locally beyond the lines, slopes and elevations shown on the drawings, to remove unacceptable material. The Field Representative will ensure that the contractor takes the necessary precautions to preserve, in an undisturbed condition, all material outside of the required excavation lines and that the stability of the excavated slopes is not impaired. The Field Representative will ensure that no construction traffic is routed over the excavated surfaces, unless they have been suitably protected to prevent damage.

7.3.3.2 Rock Excavation

Rock excavation includes the excavation of weathered and unweathered, in-situ bedrock and individual rock fragments each having a volume greater than one cubic metre. Rock excavation will only be required at the West Twin Dike Spillway and West Twin Outlet Channel. The excavation will encounter dolostone bedrock, which has been frost-shattered near the surface and along the bedrock-overburden contact.

Excavation will be done by a combination of drilling and blasting and ripping, depending on the quality of the rock and the extent of frozen conditions at the time. The materials excavated from the frost-shattered and intact bedrock zones are expected to be suitable as backfill for re-contouring or as rip rap for other areas of the WTDA, subject to screening requirements. The intact dolostone bedrock however, must be excavated and transported to a designated stockpile area where it can be used for backfill and/ or rip rap.

The Field Representative will be responsible for designating which material will go to the stockpile and which can be used for backfill.

In general, all of the bedrock excavation will be done in the dry. The construction drawings will show the required excavation limits based on the results of the subsurface explorations. The contractor will be responsible for ensuring that the structural integrity of the rock is preserved. The excavation methods must produce smooth and sound rock surfaces, with a minimum of fracturing of the rock outside the excavation. The Field Representative and the contractor will develop controlled blasting techniques that satisfy the specified excavation requirements. The contractor's initial blasts will be performed as trials. The burden, drillhole pattern, hole depth, explosive type and quantity, blasting sequence and delay pattern will be modified to achieve the specified results.

Prior to the start of drilling and blasting, the contractor will submit complete details of the blast to the Field Representative, including the following:

- The location, depth and area of each blast.
- The diameter, spacing, depth, pattern and inclination of blast holes.
- The type, strength, quantity, column load and distribution of explosives to be used per hole, per delay and per blast.
- The sequence and pattern of delay.
- The description and purposes of any special methods to be adopted.

Excavation cannot proceed without the review and approval of the contractor's plan by the Field Representative. The Field Representative will inspect all excavated rock faces and assess the degree of disturbance outside the required lines of excavation. Depending on the conditions observed, the Field Representative may require blasting practices to be modified in order to minimize blast induced damages. Excavated surfaces will not contain overhanging rock. The Field Representative will direct the contractor to scale and remove all loosened rock from the excavated slopes, even if this requires enlarging the excavation beyond the required excavation lines.

The Field Representative will map the exposed geological conditions during construction to document "as-built" conditions. This information may be used during construction to aid in design modifications that may be required due to unexpected conditions. The contractor may be required by the Field Representative to expose a fresh, undisturbed surface to permit inspection and mapping.

7.3.4 Drilling and Grouting

The Field Representative will be responsible for monitoring the drilling and grouting program for the new overflow weir. As a part of the final engineering and design of the structure, a detailed grouting program will be developed that will provide guidance to the contractor and the Field

Representative on critical items such as:

- Drill location, size, orientation and depth.
- Pressure testing criteria.
- Grout mixes, pressures, pumping rates, locations and sequences.
- Criteria for selection of additional grout holes.

7.3.5 Concrete

The Field Representative will be responsible for monitoring the placement of concrete for the overflow weir structure. This work will include acceptance of the bearing surface, confirmation of the final weir elevation, climatic conditions for the concrete placement and review of the forming arrangements and placement methodology.

Quality control testing will be required to validate the concrete placed for the weir. Information to be reviewed and tests to be undertaken include the following:

- Review of concrete mix design, including water/ cement ratio, aggregate suitability and use of special additives, if required;
- Slump cone to assess placement consistency;
- Air entrainment; and,
- Moulds of placed concrete to assess compressive strength (if critical).

7.3.6 Rip Rap

Within the WTDA reclamation works, rip rap will be placed as erosion protection along the spillway channel, the toe of the Test Cell Dike, adjacent to the new overflow weir, the existing Diversion Dike and portions of the outlet channel upstream and downstream of hydraulic structures.

Rip rap will be sourced from the following areas:

- bedrock from excavated channels (i.e. West Twin Dike Spillway);
- bedrock from specific quarries (possibly); and,
- cobbles and boulders from the Twin Lakes sand and gravel deposit.

The material will consist of clean, well graded, hard, bedrock fragments or coarse-grained particles. Screening of the host material will be required to produce the required particle size gradation.

Inspection of the processing of this material will be done by the Field Representative to ensure that the source material meets the required grain size and quality.

The Field Representative will check that the rip rap fill is placed to the lines, slopes and elevations shown on the construction drawings. Erosion protection layers will be shaped with a wide-bucket excavator to produce a planar surface with consistent grade. Rip rap should be placed within seven days of completion of the slope excavation or foundation preparation on which it is to be placed. In general, fill materials may be placed on frozen ground, provided they are free of ice and snow. The rip rap and bedding can be placed and spread in a single lift. Nominal compaction will result from the systematic routing of the spreading equipment only.

The rip rap placed on the bedding layer will be levelled and dressed in such a manner that segregation of the material into zones of uniform particle size does not occur and that the completed layer is stable, with no tendency to move or slide. The larger particle sizes should be evenly scattered throughout the zone, with the smaller particles filling the voids between the larger particles to form a uniform zone of interlocked particles with no voids through which the underlying material is visible. The finished surface should be uniform and free from undulations and depressions.

The Field Representative will take samples of the rip rap materials both before and after placement and undertake required testing (to be detailed in the technical specifications) to confirm that the material meets the specifications. Control of rip rap gradation will generally be undertaken by visual inspection.

7.3.7 Surveying

Surveyors will be required to undertake the following tasks with respect to the construction of the reclamation work:

- Layout of survey control points to be used for later construction work;
- Cut stakes for spillway excavations and other channels;
- Grade stakes layout for the various layers within the cover, spillways and for surface swales;
- Validation of various material thicknesses;
- Topographic surveys and the calculation of quantities placed for the various materials;
- Coordinates and elevations of all instrumentation installed; and,
- Production of final as-built drawings in both plan and section views.

The as-built drawings will be included in the As-built Report as reviewed in Section 7.3.9.

7.3.8 Instrumentation

As discussed in more detail in Section 8, some instrumentation will be installed into and through the various reclamation works in order to provide performance monitoring. Instruments that are currently proposed include the following:

- Settlement monitoring points;

- Thermistors;
- Frost Gauges; and,
- Shallow monitoring wells.

The Field Representative, in combination with the Site Supervisor, will be required to coordinate the installation of these instruments, where and when practical around the construction activities. Site surveyors will then be required to record coordinates and elevations for the instruments installed.

7.3.9 As-Built Reports

As required in the Water License, an As-built Report will be produced the major elements of the reclamation plan for the WTDA. These reports will contain the following information:

- Summary of construction schedule;
- Summary of quantities and test results on materials placed;
- Summary of technical decisions made as they may deviate from original design specifications and/or intent; and
- A selection of construction photos.

The objective of the as-built report is to confirm that the reclamation works have been constructed in accordance with their design intent. Any deviations from the original design basis, and the associated rationale for the deviation, will also be included. This report will be stamped by a professional engineer, registered to practice in Nunavut.

8.0 PERFORMANCE MONITORING

In accordance with the “Guidelines for Abandonment and Restoration of Mines in the NWT” (1990), a performance monitoring program has been developed to provide a means of measuring the effectiveness of the reclamation works at the WTDA. The monitoring requirements during reclamation and closure periods are fully detailed in the Monitoring Requirements Report (Water License requirement Part G, Item 9).

In general, the monitoring program provides for performance monitoring during the 2 year Reclamation Period and for a subsequent 5 year Closure Period. During the Reclamation Period, worker presence at the mine site is anticipated for construction monitoring and general reclamation activities. This presence will enable the proposed monitoring programs to be carried out by the on-site personnel under the direction of an Environmental Coordinator and geotechnical engineer. During the Closure Period, performance monitoring will be conducted to determine the success of reclamation measures. Continuous worker presence at the mine site is not planned during the closure period and environmental monitoring programs will be carried out during regular site visits and possibly utilizing trained, local field assistants and staff hired from nearby Arctic Bay.

Table 18 outlines the proposed monitoring methods and components for the surface structures and reclamation works. Figure 26 provides the location of existing instrumentation and new instrument that will be installed at the WTDA.

Surface water released from the reclaimed WTDA into Twin Lakes Creek is currently monitored at Station 159-4 (Figure 27), as required by the Water Licence. Monitoring at this location will continue through the Reclamation and Closure Periods in order to assess any influences that the reclaimed facilities may have on water quality, beyond those assessed in Appendix VII, provided herein.

As described in the Monitoring Requirements Report, the monitoring schedules will be assessed on an on-going basis and modified as appropriate to ensure that site conditions are adequately evaluated.

8.1 Reclamation Period Monitoring

The majority of the monitoring completed during the reclamation period associated with the reclamation works will involve quality assurance/quality control of construction of the reclamation covers for tailings, excavation of the spillway channel, excavation of the outlet channel and cover and relocating Reservoir shoreline tailings. Detailed construction information will include the following information:

- cover thickness, compaction, moisture content and grain size distribution;
- grades and elevations for the spillway, channel and drainage swales;
- side slopes of spillway and channel excavations;
- material present in base of spillway and channel excavation (bedrock versus overburden); and,
- bedding and rip rap materials placed for erosion protection.

Monitoring of instrumentation will be conducted at the required frequency noted in Table 19, subject to their date of installation.

The schedule for water quality monitoring at Station 159-4 (outflow for the Polishing Pond) through the Reclamation Period will remain the same as is required under the current Water Licence. Hence, daily monitoring of flow and field parameters (pH, temperature, conductivity) plus weekly monitoring of chemical parameters (total metals, sulphate and total suspended solids), will be undertaken. Analyses for total metals and sulphate will be performed at an accredited, off-site laboratory since the on-site assay laboratory will no longer be operational.

8.2 Closure Period Monitoring

Monitoring during the closure period will focus on collecting necessary information to evaluate the performance and effectiveness of reclamation works. This will include the collection of ground temperature and water quality information, as well as observing the physical condition of the reclamation works.

The monitoring schedule for the Closure Period is detailed in Table 20. The monitoring schedule is planned to be reduced through the Closure Period in anticipation of the data verifying the effectiveness and satisfactory performance of the reclamation works. All thermistors will be monitored quarterly during the five year period. Frost gauges will be read every two weeks for years 1 to 3. Geotechnical inspections will occur in the spring and fall for year 1 reducing to once per year for years 2 through 5. During the geotechnical inspection, visual observations of cover deformation, performance of spillway and channel excavation side slopes will be recorded. In-situ samples of the armour sand and gravel, shale and underlying tailings will be collected to assess the performance of the reclamation cover. The results of the performance monitoring program will be documented and submitted to the Nunavut Water Board as a component of the annual environmental report.

The schedule for water quality monitoring at Station 159-4 through years 1 and 2 of the Closure Period is reduced in recognition of the completion of reclamation activities. The schedule calls for weekly monitoring of flow and field parameters (pH, temperature, conductivity) plus monitoring every two weeks for chemical parameters (total metals, sulphate and total suspended solids). Analyses for total metals and sulphate will be performed at an accredited, off-site laboratory since the on-site assay laboratory will no longer be operational.

The schedule for water quality monitoring at Station 159-4 through years 3 to 5 of the Closure Period is reduced slightly in anticipation of good water quality. The schedule calls for monitoring of flow and field parameters (pH, temperature, conductivity) plus monitoring for chemical parameters (total metals, sulphate and total suspended solids) every two weeks. Analyses for total metals and sulphate will be performed at an accredited, off-site laboratory since the on-site assay laboratory will no longer be operational.

Implicitly included in the monitoring program is the climatic data recorded at the Nanisivik AES Station. Yearly records of temperature and precipitation values will be assessed as context for evaluating the performance of the reclamation works.

9.0 CONTINGENCY PLANS

Several contingency plans have been developed in order to address performance issues that may be identified during the reclamation and closure monitoring periods. Potential issues with the following reclamation works have been developed as outlined below:

- Talik freeze-back;
- Physical performance of reclamation covers;
- Physical condition of West Twin Dike;
- Water quality;
- Physical performance of West Twin Dike Spillway;
- Physical performance of West Twin Outlet Channel and overflow weir; and
- Physical performance of Reservoir shoreline erosion protection.

The consequences of each issue and suggested mitigation approaches are identified in Table 21. Common to all suggested mitigation measures is identification of the root cause and appropriate reaction to limit the environmental consequences of each issue. The mitigation measures range between performing localized maintenance of the covers to treatment of Reservoir water.

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TABLES

Table 1: Summary of Component Closure Plan Reports

Water License Reference	Report
Part G, Item 3	Final Closure and Reclamation Plan
Part G, Item 4	Reclamation Cover Designs
Part G, Item 5	West Twin Disposal Area Talik Investigation
Part G, Item 6	Borrow Areas Development and Closure Plan
Part G, Item 7	West Twin Disposal Area Surface Cell Spillway Design
Part G, Item 8	Waste Rock and Open Pit Closure Plan
Part G, Item 9	Reclamation and Closure Monitoring Plan
Part G, Item 12	Annual Review of Reports G3 to G9 and Submission, for Approval, of Proposed Modifications
Part G, Item 13	Report on Environmental Site Assessment (ESA) Program
Part G, Item 14	Human Health and Ecological Risk Assessment (HHERA)
Part G, Item 15	West Twin Disposal Area Closure Plan
Part G, Item 16	Underground Mine Solid Waste Disposal Plan
Part G, Item 17	Landfill Closure Plan
Part G, Item 20	Annual Review of Reports G15 to G17 and Submission, for Approval, of Proposed Modifications
Part G, Item 21	Annual Reclamation Liability Cost Update
Part G, Item 22	2007 Terms of Reference for Comprehensive Assessment of Mine Site Remediation

Table 2: Summary of Nanisivik Climate Data

Climate Variable	Parameter	Value	Source of Data
Air Temperature	Mean Annual Air Temperature (MAAT)	-15.1°C	Nanisivik AES
	Highest Recorded MAAT	-13.9°C	Nanisivik AES
	Lowest Recorded MAAT	-16.8°C	Nanisivik AES
	Mean Air Freezing Index (AFI)	5824°C-days	Nanisivik AES
	Mean Air Thawing Index (ATI)	275°C-days	Nanisivik AES
	1:100 Warm Annual Air Temperature	-13.3°C	Estimated by BGC Based on Nanisivik AES
Precipitation	Total Annual Precipitation	240 mm/year	Nanisivik AES
	Extreme Annual Precipitation (1:100 Return Period)	380 mm	Estimated (Golder 1998)
	Short Duration Extreme Rainfall (24 Hour 1:100 Year Precipitation Event)	36 mm	Estimated (Golder 1998)
	Probable Maximum Precipitation	140 mm to 210 mm	Estimated (Golder 1998)
Evaporation	Mean Annual Evaporation	200 mm/year	Estimated by DIAND (WTDA Monitoring Station)
Global Warming	Best Estimate MAAT Increase to Year 2100	2.8°C	Estimated by BGC Based on EC (1998)
	High Sensitivity MAAT Increase to Year 2100	5.0°C	Estimated by BGC Based on EC (1998)

Table 3: Summary of Borehole and Instrumentation Information

Borehole #	Elevation (m)	Surveyed (Y or N)	Location	Depth of Borehole (m)	Instrumentation Installed
BH-01	372.63	Y	Spillway	10	None
BH-02	379.36	Y	Spillway	10	None
BH-03	384.05	Y	Spillway	10	None
BH-04	386.73	Y	Spillway	10	None
BH-05	385.91	Y	Spillway	10	None
BH-06	388.71	Y	Spillway	10	None
BH-07	392.01	Y	Spillway	10	None
BH-08	386.27	Y	Spillway	10	None
BH-09			Spillway	10	None
BH10 (TC37)	387.5	Y	Surface Cell	30.5	Thermocouple
TP02-01	372.5	N	Spillway	1.4	None
TP02-02	377	N	Spillway	1.6	None
TP02-03	382	N	Spillway	0.8	None
TP02-04	385	N	Spillway	1.1	None
TP02-05	386	N	Spillway	1.4	None
TP02-06	390	N	Spillway	1.4	None
TP02-07	387	N	Spillway	1.3	None
TP02-08	390	N	Spillway	1.1	None
BGC02-01	386.9	Y	Surface Cell	19.2	Thermistor
BGC02-02	386.7	Y	Surface Cell	19.2	Thermistor
BGC02-03	386.4	Y	Surface Cell	19.2	Thermistor
BGC02-04	386.8	Y	Surface Cell	13.7	Monitoring Well
BGC02-05	386.5	Y	Surface Cell	12.2	Monitoring Well
BGC02-06	386.7	Y	Surface Cell	18.0	Thermistor
BGC02-07	386.5	Y	Surface Cell	24.7	None
BGC02-08	372.8	Y	Toe of West Twin Dike	7.9	Thermocouple
BGC02-09	375.9	Y	Test Cell Dike	30.0	Thermistor
BGC02-10	374.3	Y	Toe of West Twin Dike	8.7	Thermocouple
BGC02-11	386.5	Y	Surface Cell	26.8	Thermocouple
BGC02-12	386.3	Y	Surface Cell	25.3	Thermocouple
BGC02-13	387.3	Y	Surface Cell	30.0	Thermocouple
BGC03-01	386.5	N	Surface Cell	10.1	None
BGC03-02	386.5	N	Surface Cell	16.2	None
BGC03-03			Surface Cell		Thermocouple
BGC03-04	387.0	Y	Spillway	8.9	Thermocouple
BGC03-05	383.4	Y	Spillway	9.0	Thermocouple
BGC03-06	372.9	Y	Spillway	8.8	Thermocouple
BGC03-07	387.3	Y	Surface Cell	25.6	Thermistor
BGC03-08	387	N	Surface Cell	7.6	None
BGC03-09	387.2	Y	Surface Cell	27.4	Thermistor
BGC03-10	386.9	Y	Surface Cell	28.4	Thermistor
BGC03-11	386.0	Y	Surface Cell	24.5	Thermistor

Table 3: Summary of Borehole and Instrumentation Information Cont.

Borehole #	Elevation (m)	Surveyed (Y or N)	Location	Depth of Borehole (m)	Instrumentation Installed
BGC03-12	386.7	Y	Surface Cell	19.2	Monitoring Well/ VW Piezometer
BGC03-13	386.2	Y	Surface Cell	17.7	Thermistor
BGC03-14	386.7	Y	Surface Cell	11.9	Monitoring Well/ VW Piezometer
BGC03-15	387.3	Y	Surface Cell	16.8	Thermistor
BGC03-16	372.4	Y	Spillway	7.0	Thermocouple
BGC03-17	373.0	Y	Spillway	7.6	Thermocouple
BGC03-18	373	N	Toe of West Twin Dike	8.5	Thermocouple
BGC03-19	373	N	Toe of West Twin Dike	9.8	Thermistor
BGC03-20	388.2	Y	Surface Cell	20.1	Thermistor
BGC03-21	388.9	Y	Surface Cell	16.2	Thermistor
BGC03-22	374.5	N	Test Cell Dike	27.4	Thermistor
BGC03-23	372.0	Y	WTL Outlet Dam	3.7	Thermocouple
BGC03-24	371.4	Y	WTL Outlet Dam	4.4	Thermocouple
BGC03-31	387.6	Y	Surface Cell	21.3	VW Piezometer
BGC03-32	387.2	Y	Surface Cell	22.3	VW Piezometer
BGC03-33	387.5	Y	Surface Cell	22.9	Thermistor
BGC03-34	387.5	Y	Surface Cell	13.7	Thermistor
BGC03-35	386.8	Y	Surface Cell	14.6	VW Piezometer
BGC03-36	386.8	Y	Surface Cell	14.5	Thermocouple
BGC03-37	388.8	Y	Surface Cell	7.3	Thermistor
BGC03-38	387.7	Y	Surface Cell	15.1	Thermocouple
BGC03-39	386.5	Y	Surface Cell	10.6	Thermocouple

Table 4: Summary of Lab Test Results - Surface Cell

				Nanisivik Mine Lab				Almor Laboratory Testing							
Borehole	Location	Depth Below Grade (m)	Soil Type	Bulk Density (kg/m^3)	Moisture Content (%)	% Sand and Gravel	% Silt and Clay	Moisture Content (%)	% Gravel	% Sand	% Silt	% Clay	Gs	$Is_{(50)}$ Diametral/ Axial (MPa)	Atterberg Limits
BGC02-01	Surface Cell	10.0	Tailings		19.6	39.0	61.0		0.0	9.3	90.7	0.0	4.02		
		11.3	Tailings		20.1	44.5	55.5								
		15.2	Tailings		23.8	9.8	90.2								
BGC02-02		13.1	Tailings		17.8	78.7	21.3		0.0	13.5	86.5	0.0	4.02		
		14.6	Tailings		16.1	29.3	70.7								
		15.2	Lake Bed Sediments		16.3	92.4	7.6								
		15.8	Dolostone											5.2/ 9.9	
BGC02-03		16.2	Tailings		22.0	28.3	71.7								
		16.5	Dolostone											4.9/ 9.2	
BGC02-06		3.8	Tailings	2675	24.7	14.6	85.4								
		4.0	Tailings	2525	20.8	48.2	51.8								
		14.6	Tailings	2542	19.5	8.0	92.0		0.0	4.3	95.7	0.0	4.10		
		17.4	Dolostone											4.6/ 9.5	
BGC02-07		24.4	Lake Bed Sediments		20.7	93.0	7.0								
BGC02-11		2.1	Tailings	2795	18.5	41.1	58.9								
		6.1	Tailings	2887	18.9	25.9	74.1		0.0	30.1	69.9	0.0	4.15		
		7.9	Tailings	3224	16.0	83.3	16.7								
		9.4	Tailings	2861	19.6	13.4	86.6		0.0	6.6	90.4	3.0	3.99		
		24.7	Dolostone											5.7/ 8.6	
BGC02-12		24.1	Lake Bed Sediments		23.8	87.7	12.3		0.0	64.8	28.1	7.1	2.69		
		24.7	Dolostone											5.4/ 10.4	
BGC02-13		1.8	Tailings	3526	19.6	42.0	58.0								
		4.0	Tailings	3208	17.8	72.7	27.3		0.0	74.6	25.4	0.0	4.46		
		6.4	Tailings	2673	17.5	65.8	34.2								
		26.0	Tailings		13.2	91.4	8.6								
BGC03-01		2.3	Tailings		17.5										
		6.9	Tailings		17.2										
		9.9	Tailings		20.0										
BGC03-02		0.8	Tailings		13.7	91.9	8.1								
		1.1	Tailings		12.4	84.1	15.9								
		3.8	Tailings		14.4	77.6	22.4								
		5.0	Tailings		16.4	76.3	23.7								
		5.3	Tailings		13.4	85.0	15.1								
		8.4	Tailings		13.3	68.3	31.7								
BGC03-07		1.4	Tailings		11.6	84.3	15.7								
		2.9	Tailings		15.2	59.1	40.9								
		3.2	Tailings		19.7	86.8	13.2								
		5.9	Tailings	2558	26.1	35.5	64.5								
		9.0	Tailings		38.7	96.7	3.4								
		11.7	Tailings		32.7	61.6	38.4								
		13.3	Tailings		20.2	75.7	24.3								
		16.4	Tailings	2337	21.1	58.5	41.5								
		17.5	Tailings	2724	21.4	33.3	66.7								
		23.9	Lake Bed Sediments					10.3	18.9	57.2	19.8	4.1	2.83		
		24.5	FSB												
BGC03-08		1.3	Tailings		14.2	79.8	20.2								
		2.9	Tailings		16.5	78.3	21.8								
		3.6	Tailings		18.2	49.8	50.2								
		5.6	Tailings		15.6	81.9	18.1								
BGC03-09		1.4	Tailings		14.0	86.6	13.4								
		2.3	Tailings		15.8	77.3	22.7								
		4.4	Tailings		18.3	81.2	18.8								
		5.9	Tailings		17.4	30.3	69.7								
		8.1	Tailings		19.8	33.8	66.3								
		10.5	Tailings		19.2	45.7	54.3								
BGC03-10		1.7	Tailings	2849	13.4	57.2	42.9								
		3.2	Tailings	2581	15.6	32.2	67.8								
		4.4	Tailings	3075	14.8	73.4	26.6								
		27.6	Tailings		12.3	82.3	17.7								
		28.2	Tailings		11.7	68.7	31.3								
BGC03-11		1.7	Tailings		11.2	92.5	7.5								
		2.8	Tailings		12.8	67.2	32.8								
		4.1	Tailings		15.5	83.7	16.3								
		19.7	Tailings		8.0	87.4	12.6								
		20.6	Tailings		13.4	92.4	7.6								
		20.6	Tailings		11.8	92.0	8.0								
		24.4	Bedrock												
BGC03-12		0.7	Tailings		4.9	80.0	20.0								
		4.4	Tailings	2743	17.2	72.0	28.0								
		5.0	Tailings		15.7	57.0	43.0								
		5.9	Tailings	2823	17.3	60.5	39.6								
		8.2	Tailings		17.0	43.7	56.3								
		10.2	Tailings	2809	14.1	75.4	24.6								
		12.0	Tailings	2790	15.8	28.8	71.2								

Table 4 Cont.: Summary of Lab Test Results - Surface Cell

				Nanisivik Mine Lab				Almor Laboratory Testing							
Borehole	Location	Depth Below Grade (m)	Soil Type	Bulk Density (kg/m^3)	Moisture Content (%)	% Sand and Gravel	% Silt and Clay	Moisture Content (%)	% Gravel	% Sand	% Silt	% Clay	Gs	$Is_{(50)}$ Diametral/ Axial (MPa)	Atterberg Limits
BGC03-13	Surface Cell	0.8	Tailings	2462	15.7	76.3	23.7								
		2.9	Tailings		11.4	89.2	10.8								
		3.5	Tailings		16.3	81.9	18.1								
		5.0	Tailings		13.3	81.1	18.9								
		7.2	Tailings					17.8							
		16.1	Tailings	2562	23.0	31.3	68.8								
		16.3	Lake Bed Sediments					10.9							
		16.4	Dolostone												
BGC03-14		1.1	Tailings		13.8	41.9	58.1								
		2.9	Tailings		15.1	83.5	16.5								
		4.7	Tailings		14.7	91.2	8.8								
		8.1	Tailings			75.9	24.1								
		13.0	Tailings		18.6	86.5	13.6								
BGC03-15		2.3	Tailings		14.6	85.3	14.7								
		2.9	Tailings		14.5	88.5	11.5								
		10.6	Tailings		16.3	84.6	15.4								
		11.4	Tailings	2744	19.6	44.2	55.8								
		14.3	Tailings		19.0	53.0	47.0								
		16.3	Tailings	2608	18.8	25.8	74.2								
		16.5	Dolostone												
BGC03-20		1.8	Tailings			80.5	19.6								
		2.1	Tailings		17.9	57.5	42.5								
		4.5	Tailings		21.2	31.9	68.1								
		5.3	Tailings			36.3	63.7								
		5.6	Tailings					14.0							
		8.7	Tailings		24.3	16.0	84.0								
		8.4	Tailings		21.9	16.1	83.9								
		10.6	Tailings					15.7	0.0	25.0	73.4	1.6	3.9		
		19.7	Lake Bed Sediments		21.1	92.8	7.2								
		20.0	Lake Bed Sediments		19.0	83.7	16.3								
BGC03-21		1.5	Ice and Tailings			67.5	32.5								
		1.7	Ice and Tailings		69.9	17.1	82.9								
		4.1	Tailings		15.1	89.2	10.8								
		5.0	Tailings		17.4	69.1	30.9								
		5.6	Tailings		19.3	66.2	33.8								
		8.8	Tailings		30.4	30.2	69.9								
		10.3	Tailings					18.5	0.0	7.5	90.1	2.4	3.9		
		14.5	Tailings					12.0	81.6		18.4				
BH 10		4.75	Tailings			30.3	69.7								
		14.4	Tailings			81.2	18.8								
		30.2	Lake Bed Sediments			96.4	3.6								
BGC02-08	Toe of WT Dike	1.5	Tailings	3366	18.5	34.2	65.8								
		3.0	Tailings	2748	31.3	18.6	81.4		0.0	5.3	92.7	2.0	4.14		
		4.0	Tailings		17.1	15.9	84.1								
		4.6	Lake Bed Sediments		9.0	85.6	14.4								
		6.1	Lake Bed Sediments		9.9	84.7	15.3								
BGC02-10		4.9	Tailings	3102	23.9	10.1	89.9								
		5.2	Tailings	3190	21.5	28.7	71.3								
BGC03-18		1.7	Tailings		16.8	83.5	16.5								
		2.9	Tailings		26.2	89.1	10.9								
		4.1	Tailings		16.7	84.6	15.4								
		6.0	Tailings		22.9	45.5	54.5								
		6.9	Lake Bed Sediments		31.6	81.1	18.9								
		8.4	Dolostone												
BGC03-19		1.4	Tailings			29.8	70.2								
		2.7	Tailings		17.1	81.0	19.0								
		3.5	Tailings		19.2	83.2	16.8								
		4.2	Tailings		23.4	98.9	1.1								
		5.1	Tailings		24.2	30.8	69.2								
		5.9	Tailings		16.5	84.2	15.8								
		7.5	Tailings		15.8	80.9	19.1								
		9.3	Tailings		16.3	47.6	52.4								
		9.6	Lake Bed Sediments					36.7							
BGC02-09	TC Dike	7.0	Tailings	2988	15.0	93.9	6.1								
		8.8	Tailings	3062	14.9	95.9	4.1								
		10.1	Tailings	3192	15.6	95.6	4.4								
		13.7	Tailings	2141	15.4	68.2	31.8								
		14.3	Tailings	2128	17.2	16.7	83.3								
		29.9	Tailings		13.3	67.3	32.7								
BGC03-22		4.1	Tailings		10.4	88.7	11.3								
		4.4	Tailings		5.2	93.0	7.0								
		5.3	Tailings		12.0	71.5	28.5								
		7.0	Tailings			83.3	16.7								
		9.0	Tailings		11.0	92.3	7.8								
		11.7	Tailings		11.0	84.6	15.4								
		16.3	Tailings												
		17.8	Tailings		14.5	65.4	34.6								
		27.4	Lake Bed Sediments		50.8	77.4	22.7								

Table 5: Results of pH/Conductivity Testing on Pore Water samples from Surface Cell

	Jul-99		2001		September 2002		19-Aug-03	
Monitoring Well/ Station	pH	Conductivity (μS/cm)	pH	Conductivity (μS/cm)	pH	Conductivity (μS/cm)	pH	Conductivity (μS/cm)
BGC02-04					10.7	> 20,000		
BGC02-05					10.4	>20,000		
BGC03-12							9.2	4110
BGC03-14							10.4	6230
TC13 (seepage)	6.7	3700						
TC18 (seepage)	6.6	5350						
Station 159-2 (Reclaim Pump House)			8.8 to 12.2	3450 to 5500				

Table 6: Summary of Water Quality Testing

	Water	Ice
pH	8.73 to 10.7	-
SO_4	575 mg/L to 1390 mg/L	<1 mg/L to 14 mg/L
$Cu_{(T)}$	0.014 mg/L to 0.09 mg/L	0.842 mg/L
$Cu_{(D)}$	<0.006 mg/L	-
$Fe_{(T)}$	1.68 mg/L to 30 mg/L	38.3 mg/L
$Fe_{(D)}$	<0.006 mg/L to 0.013 mg/L	-
$Pb_{(T)}$	0.517 mg/L to 3.32 mg/L	14.0 mg/L
$Pb_{(D)}$	<0.025 mg/L	-
$Zn_{(T)}$	0.250 mg/L to 4.75 mg/L	5.36 mg/L
$Zn_{(D)}$	0.013 mg/L to 0.025 mg/L	-

Note: **(D) – Dissolved Value**
(T) – Total Value

Table 7: Summary of Material Volumes Required for WTDA Reclamation Covers

Area	Shale (Cover) (m^3)	Twin Lakes Sand and Gravel (Armour) (m^3)	Rip Rap (m^3)	Bedding (m^3)
Surface Cell tailings and crest of West Twin Dike	400,000	95,000	-	-
Downstream face of West Twin Dike	5,000	11,000	-	-
Tailings at the toe of West Twin Dike	33,500	7,500	-	-
Transition zone tailings at the toe of West Twin Dike	2,500	-	750	200
Test Cell tailings and Test Cell Dike	144,000	26,000	-	-
Transition zone tailings at toe of Test Cell Dike	5,000	-	3250	550

Note: All quantities are neat and in-place.

Table 8: Key Water Levels – West Twin Lake After Closure

Water Level during PMP conditions (MWL):	Elev. 370.8 m
Water Level during 100-year Storm:	Elev. 370.4 m
Normal Water Level (NWL):	Elev. 370.2 m
Minimum (Low) Water Level (LWL):	Elev. 370.0 m

Table 9: Wave Height Calculations and Shoreline Erosion Protection

Parameters	**Reservoir (West Twin Lake)**
Effective Fetch (km)	0.32
Design Wind Speed (km/h)	112.6
Design Wave Height (m)	0.40
Bank Slope	**Required Mean Stone Size (mm)**
4H:1V	200

Notes:
1. Constructed slopes steeper than 4H:1V are not recommended for long-term stability. If in the field, it is understood that a steeper slope is required locally, appropriate design details will have to be developed for the specific application.

2. For slopes flatter than 4H:1V somewhat smaller rip rap could be used. However, for simplicity, a single gradation is recommended.

3. Design wind speed source: Environment Canada.
4. Wind speed correction factor is 1.07, due to fetch length.
5. Median stone size calculated considering specific gravity = 2.65.

Reference: Smith (1995).

Table 10: Rip Rap and Bedding Material Grain Size Specifications

Particle Diameter (mm)	Material Type and Percent Passing			
	Type 1 Rip Rap (D_{50} – 300 mm)	**Type 1A Rip Rap (D_{50} – 200 mm)**	**Type 2 Bedding /Erosion Protection (D_{50} – 100 mm)**	**Type 3 Filter Sand and Gravel (D_{50} – 25 mm)**
Maximum Particle Size (mm)	600	450	300	300
Median Particle Size, D_{50} (mm)	300 min.	200 min.	100 min.	See below
15% Finer than, D_{15} (mm)	450 max.	150 max.	75 max.	

Note:
1. The fraction finer than 75 mm, shall have less than 5% passing the US No. 200 sieve (0.075mm).
2. The gradation specification for the shale and Twin Lakes sand and gravel is provided in the Reclamation Covers Design report.

Type 3 Filter	
Sieve Size (mm)	Percent Passing
75	100-75
25.4	70-40
19	67-40
12.5	55-25
9.5	45-12
4.75	35-0
2.36	30-0
1.18	23-0
0.300	13-0
0.150	8-0
0.075	5-0

Table 11: Peak Flow Conditions - West Twin Outlet Channel

Parameter	Precipitation Condition	
	100-yr	**PMP**
Inputs:		
Peak flow (m^3/s)	*1.7*	*6.5*
Channel Slope (%)	*0%*	*0%*
Width (m)	*7*	*7*
Peak Flow Depth (m)	0.20	0.60
Peak Flow Velocity (m/s)	1.5	1.5

Table 12: Erosion Protection Requirements – West Twin Outlet Channel

Material Exposed	Erosion Protection Layers (Thickness)
Intact Rock	none
Frost Shattered Rock	Type 1A (0.45 m) ON Type 2 (0.30 m)
Overburden	Type 1A (0.45 m) ON Type 2 (0.30 m) ON Type 3 (0.15 m)

Notes:

1. Thicknesses are measured perpendicular to slope
2. Material Descriptions
   - Type 1A – Rip Rap, D_{50} = 200 mm
   - Type 2 – Bedding Material, D_{50} = 100 mm
   - Type 3 – Filter – Sandy Gravel
3. D_{50} – mean particle size

Table 13: Estimated Construction Quantities - West Twin Outlet Channel

Excavation Volumes (m^3)		Fill Volumes (m^3)		
Overburden Excavation	Intact Bedrock	Rip rap (Type 1A)	Erosion Protection (Type 2)	Bedding (Type 3)
1,300	10	300	200	100
TOTAL 1,310 m^3		**TOTAL 600 m^3**		

Table 14: Design Flow Conditions – East Twin Diversion Channel

Design Flow and Channel Geometry		
	100-yr event	PMP event
Design Flow, m^3/s	13.2	50
Channel Slope	0.686%	0.686%
Side Slope, V:H	1:1.5	1:1.5
Bottom width, m	5.5	5.5
Manning cofficient	0.035	0.035
Hydraulic Modelling Results (Manning's Equation)		
Channel Depth, m	1.05	2.20
Velocity, m/s	1.95	2.89
Median Stone Size Calculation		
D_{50} mm	130	284
Notes:		
Hydraulic Calculations by FlowMaster v.5.17 (Haestad Methods, Inc.)		
D_{50} - mean particle diameter		
Median stone size calculated considering specific gravity = 2.65		
Reference: Smith (1995).		

Table 15: Estimated Construction Quantities East Twin Diversion Channel Upgrading

Excavation Volumes (m^3)	Fill Volumes (m^3)		
Overburden Excavation	**Rip rap (Type 1)**	**Bedding (Type 2)**	**General Fill**
60	360	240	1,000
TOTAL 60 m^3	**TOTAL 1,600 m^3**		

Table 16: Summary of Water Quality Projections for Zinc

Timeframe	Porewater Expulsion					Natural Runoff		Reservoir Perimeter		Polishing Pond Outflow			
	Surface Cell talik		Test Cell talik		% of	Conc.	% of	Conc.	% of	Conc.	Volume	Load	% of
	Volume	Conc.	Volume	Conc.	annual load		Annual load		Annual load		(m3)	(kg)	annual load
Years 1-5	32400	0.025	18000	0.025	0.6%	0.056	79%	0.3 – 24	20%	0.072	496,000	35	100%
Years 5-10	12960	0.033	9900	0.041	0.3%	0.056	80%	0.3 – 24	20%	0.073	491,000	35	100%
Years 10-15	11160	0.051	3600	0.101	0.4%	0.056	80%	0.3 – 24	20%	0.073	490,000	35	100%
Years 15-20	10440	0.141	0	-	0.9%	0.056	79%	0.3 – 24	20%	0.073	489,500	35	100%
Years 20-25	2844	0.291	0	-	0.5%	0.056	80%	0.3 – 24	20%	0.073	488,000	35	100%

Notes: all concentrations in mg/L.
Volumes in m^3.

Table 17: Testing Frequencies for Cover Materials

Item	Minimum Test Frequency	
	Shale	**Twin Lakes sand and gravel**
Grain Size Distribution	Every 10,000 m^3	Every 10,000 m^3
Moisture Content	Every 5,000 m^3	Not Required

Table 18: Performance Monitoring of Reclamation Works

COMPONENT	MONITORING COMPONENT	MONITORING METHODS	DETERMINATOR	PURPOSE/ IMPACT
Talik Freezeback	Surface Cell and Test Cell Taliks	Thermistors and Thermocouples	Ground Temperatures	Slower (or faster) than anticipated talik freezeback.
		Vibrating wire piezometers	Pore Pressures	Could lead to pingo formation, dike instability or transport of tailings or pore water to surface.
		Monitoring wells	Water Quality	Pore water expulsion and cryoconcentration of solutes could have impact on Reservoir water quality
	Surface Deformation	Visual monitoring Topographic surveys	Frost Heave and Pingo/Frost Mound Formation	Cracking and deformation of cover would allow for release of talik pore water, release of tailings and damage to the cover.
	West Twin Dike	Visual monitoring Topographic surveys Vibrating wire piezometers Thermistors and thermocouples	Dike Instability • Heave deformations • Cracking • Pore pressures • Thawing at depth	Dike instability may release talik pore water and tailings into the Reservoir.
Reclamation Covers	Armour Layer	Visual monitoring	Erosion	Could lead to erosion of underlying shale
			Settlement	Standing water on surface
			Frost effects	Displaced surface fragments
			Break down of particles	More erosion of underlying shale
	Shale Layer	Visual monitoring	Erosion	Exposure of underlying tailings Reduction of insulating thickness
		Visual monitoring and survey monuments	Settlement	Interrupts surface drainage and may results in standing water Reduction of insulating thickness
		Thermistors and frost gauges	Insulation for permafrost aggradation	Thaw into tailings may result in poor quality of runoff water, or initiate oxidation of tailings
		Drilling, sampling and testing for bulk density and moisture content of shale and underlying tailings	Moisture content retained in void space (controls oxygen entry)	Must check that moisture retention is being achieved to verify limiting of oxygen entry into underlying tailings
		Shallow monitoring wells	Poor water quality from surface drainage	May have negative impact on water quality in Reservoir
		Visual monitoring Shallow monitoring wells	Tailings displaced into shale	May result in poor quality surface drainage
	Drainage Swales	Visual monitoring	Erosion	Exposure of underlying tailings Reduction of insulating thickness
		Visual monitoring and survey of swale grades	Settlement/ heave of the swale	Standing water

Table 18: Performance Monitoring of Reclamation Works Continued

COMPONENT	MONITORING COMPONENT	MONITORING METHODS	DETERMINATOR	PURPOSE/ IMPACT
West Twin Dike Spillway	Spillway Inlet	Visual Inspection	Frost Heave/ Thaw Settlement Ice and Snow Blockage	May reduce discharge capacity and/or create ponding
	Channel	Visual Inspection	Channel erosion due to flow and/ or slumping and/or settlement of channel side walls due to flow erosion or thawing of permafrost.	May reduce discharge capacity. Potential for progressive degradation.
	Spillway Outlet	Visual Inspection	Permafrost degradation. Erosion of tailings in Reservoir.	Assess if plunge pool and discharge element are appropriate.
	Rip Rap	Visual Inspection	Integrity of rip rap	Assess rip rap integrity for the long-term.
	Spillway Water Level	Visual Inspection	Water depth in channel during high flow conditions	Confirm depth of rip rap protection.
Reservoir Shoreline Erosion Protection	Natural Shoreline	Visual Inspection	Erosion Suspended sediment Thermal degradation	Assess wave action during open water season. Assess stability of surfical slopes.
	Rip Rap	Visual Inspection	Displacement, erosion, physical deterioration due to freeze/ thaw and wetting/ drying effects	Assess effects of wave and ice action following ice break up an open water season.
	Ice Cover	Visual Inspection	Plucking and displacement of rip rap Tailings entrainment Ice debris blocking outlet	Check during ice break-up period to assess conditions.
	Water Quality	Visual/ Sampling and Testing	Suspended sediment Re-suspension of tailings	Check that water cover and shoreline protection elements are sufficient to maintain low levels of suspended sediment in the water to be discharged.

Table 18: Performance Monitoring of Reclamation Works Continued

COMPONENT	MONITORING COMPONENT	MONITORING METHODS	DETERMINATOR	PURPOSE/ IMPACT
Polishing Pond Outlet Channel And Overflow Weir	Concrete weir wall	Visual Inspection	Deformation cracking Spalling Chemical reactions	Ensure physical stability of structure and concrete integrity.
	Seepage	Visual Inspection	Water seeping around or under the weir wall	Assess effectiveness of foundation grouting and abutment seal.
	Rip Rap	Visual Inspection	Displaced or deteriorated rip rap	Erosion of channel and abutments may affect integrity of concrete weir wall.
	Blockages	Visual Inspection	Ice or debris in channel upstream or downstream of structure	Ensure that design channel capacity is maintained during open water period.
East Twin Lake Diversion Dam And Channel	Diversion Dam	Visual Inspection	Erosion/ degradation of rip rap Settlement Cracking	Integrity of Diversion Dam is required to prevent runoff from East Twin Lake from entering the Reservoir.
	Channel erosion protection	Visual Inspection	Displaced or degraded rip rap	Erosion protection is required to ensure uniform flow in vicinity of dam.
	Diversion channel slopes	Visual Inspection	Erosion of slope or channel bottom Displaced or deteriorated rip rap	Ensure stability of channel during flood flow events.

Table 19: Reclamation Period Monitoring Schedule

Monitoring Method	Year of Reclamation Activities	
	1	2
Thermistors and Thermocouples	Monthly	Monthly
Vibrating Wire Piezometers	Monthly	Monthly
Monitoring Wells (Water Quality Samples)	Twice in Summer Period	Twice in Summer Period
Visual Inspections (by site or local staff)	Weekly (June - October) Every Two Weeks (November - May)	Weekly (June - October) Every Two Weeks (November - May)
Geotechnical Inspections	Twice/year (Spring/ Fall)	Twice/year (Spring/ Fall)
Topographic Survey	Twice/year (of selected points)	Twice/year (of selected points)

Table 20: Closure Period Monitoring Schedule

Monitoring Method	Year Following Completion of Reclamation Activities				
	1	2	3	4	5
Thermistors and Thermocouples	Quarterly	Quarterly	Quarterly	Quarterly	Quarterly
Frost Gauges	Every Two Weeks (June – September)	Every Two Weeks (June – September)	Every Two Weeks (June – September)	Only if Required	Only if Required
Vibrating Wire Piezometers	Quarterly until Frozen	Quarterly until Frozen	Quarterly until Frozen	Quarterly until Frozen	Quarterly until Frozen
Monitoring Wells (Water Quality Samples)	Once per Summer	Once per Summer	Once per Summer	Once per Summer	Once per Summer
Visual Inspections (by site or local staff)	Every two weeks (June - October) Monthly (November - May)	Every two weeks (June - October) Monthly (November - May)	Every two weeks (June - October) Monthly (November - May)	None	None
Geotechnical Inspections	Twice/ year (Spring/ Fall)	Once/year pending suitable performance	Once/year pending suitable performance	Once/year pending suitable performance	Once/year pending suitable performance
Topographic Survey of selected points	Once/year	Once/year	Once/year	None	None

Table 21: Contingency Plans for Undesirable Performance of Reclamation Works

COMPONENT	ISSUES	CONSEQUENCES	MITIGATION APPROACH
Talik Freezeback	Slower than anticipated freezeback of talik	• Freezeback has not progressed at rate predicted by geothermal modelling.	• Review of physical conditions and geothermal data by geotechnical engineer to recalibrate the model. • Continue to monitor freezeback rate beyond current anticipated closure monitoring period, with consideration of potential impacts.
	Elevated pore pressures in talik	• Could lead to pingo formation, dike instability or transport of tailings or tailings to surface.	• Increase frequency of data collection and review. • Review of physical conditions and piezometric data by geotechnical engineer to identify root cause of pressures. • Install relief wells into high pore pressure areas.
	Poor water quality talik that impacts Reservoir water quality	• Cryoconcentration of solutes related to pore water expulsion during freezeback of the taliks may have a negative impact on the Reservoir water quality.	• Increase water quality monitoring frequency. • Review of physical conditions and geothermal and water quality data to quantify volumes, water quality and expected duration. • Treat Reservoir water prior to release to environment.
	Formation of pingo or frost mound	• Cracking or deformation of cover possibly allowing release of tailings or water. • Possible interference with surface drainage grades.	• Review of physical conditions and geothermal data by geotechnical engineer to identify root cause of deformation. • Pressure relief wells into pingo or frost mound to reduce surface deformations. • Maintenance and repair of cover to achieve design specifications.
West Twin Dike	West Twin Dike Instability	• Potential to release talik pore water and/or tailings into the Reservoir.	• Review of physical conditions and geothermal and piezometric data to identify root cause of observations noted. • Pressure relief wells to reduce pore pressures if appropriate. • Maintenance and repair of dike crest and slope if appropriate. • Continued vigilant monitoring.
Reclamation Cover	Erosion of armouring layer	• Reduction in Twin Lakes sand and gravel and shale thickness • Reduction in insulation thickness • Potential poor quality run-off	• Enlarge the surface particle size. • Flatten surface grades. • Supplement layer with additional armour.
	Settlement of cover interrupts surface drainage	• Allows for standing water that may leave thermal impacts	• Localized repair of settled area to correct surface grade.
	Excessive depth of active layer thaw	• Sub-surface thawed tailings may cause poor surface water quality runoff	• Need to determine if cause is climatic conditions (warm) and/or thermal properties of the cover shale and/or sand and gravel armour. • If excessive thaw depth occurs in localized areas, need to determine if surface water quality impacts are significant and/or if oxidation may be initiated. • If impacts are significant, determine if cover layer can be thickened, if moisture content can be increased or if grade can be reduced to enhance moisture retention in the cover.
	Surface water runoff quality becomes poor	• Potential impacts on Reservoir water quality	• Assess and attempt to mitigate root causes with respect to the cover and the underlying tailings. • Potential treatment of water in the Reservoir prior to discharge to the environment.
	Heaving of cover results in open cracking	• Infiltration of water/oxygen • Oxidation of tailings • Poor quality surface runoff	• Localize repair of cracked areas by contouring or supplement with additional material.

Table 21: Contingency Plans for Undesirable Performance of Reclamation Works Continued

COMPONENT	ISSUES	CONSEQUENCES	MITIGATION APPROACH
West Twin Dike Spillway	Frost Heave	• May reduce discharge capacity	• Excavate heaved material, replace with non-frost heave susceptible material to design grade.
	Frost Settlement	• Creates ponding, thermal impacts	• Grade surface and/or place compacted material to re-establish grade and eliminate depression(s).
	Blockage by Ice and Snow	• Ponding on top of cover, reduces freeboard on dam, thermal impacts, reduced spillway discharge capacity	• Remove ice and snow accumulation before spring melt. • Assess causes and mitigate.
	Channel – erosion and/or settlement of slopes	• Degradation of slopes, permafrost, increased sedimentation in channel, reduced discharge capacity.	• Regrade areas due to thaw settlement. • Flatten slopes to stable angle. • Remove debris in channel. • Replace or place rip rap.
	Channel – erosion/ scour of channel bottom	• May lead to undermining of slopes • Flow concentration and accelerating erosion • Thermal degradation • Increased sedimentation downstream • Affects discharge capacity	• Check flows, gradients and velocities. • May require local weirs to dissipate energy. • Replace with rip rap sized for local velocities.
	Outlet – energy dissipation - scour	• May lead to undermining of slopes • Flow concentration and accelerating erosion • Thermal degradation • Increased sedimentation downstream • Affects discharge capacity	• Increase size/depth of plunge pool and/or outlet area. • Replace rip rap with larger sizes.
	Rip Rap Integrity	• Progressive erosion of slope • Increased sedimentation • Reduced discharge capacity • Possible thermal degradation	• Assess velocities during periods of seasonal/ extreme flow. • Replace rip rap with larger size. • Consider flattening slopes.
	Inadequate Freeboard	• Potential for overtopping of dam	• Remove blockages in channel. • Assess causes. • Mitigate condition to prevent future occurrence.
	High Spillway Water Level	• Increased potential for ponding upstream in Surface Cell • Reduced discharge capacity • Erosion of Channel	• Review hydraulic design parameters. • Inspect channel and remove blockages and constrictions. • May need to increase channel capacity by altering grade or by excavating wider or deeper.

Table 21: Contingency Plans for Undesirable Performance of Reclamation Works Continued

COMPONENT	ISSUES	CONSEQUENCES	MITIGATION APPROACH
Reservoir Shoreline Erosion Protection	Erosion of natural shoreline	• Suspended sediment • Slope instability • Thermal degradation	• Assess physical conditions. • Place rip rap and/or thawed cover over affected area.
	Rip Rap degradation	• Loss of erosion protection • Increased suspended sediment • Re-suspension of tailings • Thermal degradation • Test Cell cover instability	• Assess causes – i.e. ice plucking, wave energy, physical deterioration. • Replace with larger rip rap or more durable material.
	Water quality degradation	• Release of non-compliant water from Reservoir	• Assess sources: sedimentation, re-suspension of tailings, ARD/ metals leaching from waterline area. • Improve rip rap cover in areas subject to erosion. • Increase shale/ rip rap cover to a greater depth thickness. • Manage and treat (if required) reservoir water.
West Twin Outlet Channel and Overflow Weir	Deformation or cracking of concrete wall	• Inability to maintain reservoir at design water level • Increased gradient in outlet channel • Wall failure	• Check foundation conditions. • Assess concrete quality. • Undertake concrete repairs and reinforce structure/ foundation.
	Concrete spalling or chemical reactions	• Long term deterioration of structure • Potential lowering of Reservoir pond level	• Assess concrete quality. • Assess reservoir water quality. • Repair concrete and cover with chemical resistant concrete or other coating.
	Seepage around or under concrete wall	• Depending on location and rate of flow, may affect Reservoir levels. • May cause erosion of embankment slopes around structure.	• Assess volumes and location during open water season. • Remove, replace, and re-compact abutment materials along seepage path. • Additional grouting may be required in bedrock foundation.
	Damaged Rip Rap	• Increased erosion • Increased suspended sediment • Long term thermal degradation	• Assess causes. • Increase rock sizes and replace – may require regarding and re-sloping of bank. • May require more durable rock if rock particles are deteriorating.
	Blockages	• Reduced discharge capacity • Increased Reservoir level • Increase in shoreline erosion	• Remove blockages. • Mitigate conditions causing blockage.

Table 21: Contingency Plans for Undesirable Performance of Reclamation Works Continued

COMPONENT	ISSUES	CONSEQUENCES	MITIGATION APPROACH
East Twin Lake Diversion Dike and Channel	Erosion or degradation of rip rap on dam or channel slopes.	• Increased erosion • Degradation of dam and channel • Increased suspended sediment	• Assess causes. • Repair damaged areas with new rip rap – larger particle size and/ or better quality rock.
	Scour of channel bottom	• May affect rip rap protection on adjacent slopes	• Armour channel bed with suitable sized rip rap.

FIGURES

APPENDIX I
HYDROLOGICAL STUDIES – NANISIVIK MINE

Golder 2004b – Extended Hydrological Study
Golder 2004c– Addendum to Extended Hydrological Study
Golder 2004d – Water Balance Assessment – Nanisivik Mine Closure

APPENDIX II
WATER QUALITY TEST DATA

Accutest Laboratories – Tested water samples from:
BGC03-12 and BGC03-14
Ice Core samples from:
BGC03-16, BGC03-20, and BGC03-21. (June 2003)

Maxxam Analytical – Tested water samples from:
BGC03-12 and BGC03-14. (Sept. 2003)

APPENDIX III
PIEZOMETRIC MONITORING AND TESTING

	Borehole	Figure
Piezometric Monitoring Results	BGC03-12	III-1
	BGC03-14	III-2
	BGC03-31	III-3
	BGC03-32	III-4
	BGC03-35	III-5
Recovery Test	BGC03-12	III-6
	BGC03-14	III-7

APPENDIX IV
GEOTHERMAL MONITORING
WEST TWIN DISPOSAL AREA

APPENDIX V
STABILITY ANALYSES

1.0 INTRODUCTION

A stability analysis of the West Twin Dike and Test Cell Dike (with associated sloping reclamation cover) was conducted to evaluate the short and long-term stability of the dikes. The objectives of the stability analyses were to determine the following:

- the current stability of the dikes; and,
- how pore pressure fluctuations may impact dike stability.

It should be noted that a stability assessment was undertaken for the West Twin Dike previously (BGC 2000 and 2002a). These results were included in the Closure and Reclamation Plan (February 2002) submitted by CanZinco to the Nunavut Water Board. In addition, a failure consequence classification of low was provided in BGC (2002b). Results of the previous analyses indicated that the stability of the West Twin Dike was acceptable given the assumptions made at the time. Given the greater understanding of the geotechnical and geothermal properties gained from the 2002 and 2003 geotechnical investigations, the results of the stability models were reviewed and the results of that review are provided herein.

2.0 WEST TWIN DIKE STABILITY ANALYSES

2.1 Dam Stability Criteria

CDA (1999) defines slope stability Factor of Safety as the factor required to reduce the mobilized shear strength parameters (of the soil or rock) of a potential sliding mass into a state of limiting equilibrium. A simpler definition notes the Factor of Safety as the ratio of the resisting forces in a sliding block (e.g. shear strength of the soil) to the driving forces (e.g. soil weight). As such, for block to be considered “stable”, the resisting forces must be in excess of the driving ones. CDA (1999) provides design criteria for the required factors of safety for static analysis of embankment dams, as summarized in Table V-1.

The closure reclamation plan for the Surface Cell is to remove the retained pond and construct a reclamation cover over the tailings that conveys surface water and prevents ponding. Since a talik has been observed within the tailings beneath the dike, and pore pressures have been observed in these thawed tailings, the steady-state seepage case is considered to reflect the long-term condition for the dike. As such, a Factor of Safety of at least 1.5 is recommended for the static stability of the West Twin Dike. The rapid draw down and end of construction criteria are not considered applicable since the basin upstream of the dike is currently filled with tailings solids.

CDA (1999) also provides guidance on the design criteria for earthquake resistance of embankment dams, which states that dams (and associated components) shall be designed to resist the forces associated the Maximum Design Earthquake (MDE). The MDE is also defined in CDA (1999). For a High Consequence Category dam (failure leading to some human fatalities and/or large socio-economic, financial, environmental damages), the MDE shall be either of the following:

- 50% to 100% of a deterministically derived Maximum Credible Earthquake (MCE) or
- a return period of 1,000 to 10,000 years for a probabilistically derived earthquake.

For low consequence dams, the MDE requirements are stated as 1:100 to a 1:1000 year return event. Information provided by the Geological Survey of Canada and referenced in BGC (2000a) notes that the 1 in 476 year event is 0.076g and the 1 in 1,000 year event is 0.099g. The site is situated in the Z_a=1 (0.05g) zone, according to the Building Code of Canada. CDA (1999) does not provide any specific factors of safety required for pseudo-static analyses, but refers to some other published work.

Mitchell (1983), in his textbook on earth structure design, does provide typical safety factors for impoundment dams, as summarized in Table V-2. From this reference, Factors of Safety between 1.1 and 1.2 would be recommended for seismic analysis of low risk dams, as the West Twin Dike has been classified.

2.2 Methodology

The stability analyses was conducted utilizing two methods; a “rigid block” analysis, conducted on the West Twin Dike and a “conventional” or “thawed” analysis, conducted on both the West Twin and Test Cell Dikes. The theory and applicability of each method is summarized in the following sections.

2.2.1 Rigid Block Analysis

The rigid block analysis theory is illustrated on Figure V-1. The analysis assumes the entire West Twin Dike to be a block, consisting of shale and tailings, which are frozen. The dike block is then considered to be similar to a rigid structure, such as a concrete buttress dam, containing the tailings on the upstream side. The tailings contained behind the dike are then considered to be exerting water and soil pressures on the upstream side of the “rigid block”.

The assumptions made in order to perform this assessment include the following:

- The upstream face of the dike was considered to be vertical.
- The base of the “rigid block” was only considered to be sliding on tailings. This is considered a valid assumption since the toe of the West Twin Dike will be at approximately 374 m elevation once the reclamation cover of the tailings at the toe of the dike is constructed.

- The height of the dam was considered to be 14 m. This height was used since the reclamation cover at the toe of the dike will be constructed to approximately 374 m elevation.
- The pressure exerted on the upstream face of the dike by the thawed tailings and pore water was calculated using the coefficient of active earth pressure as calculated using the range of effective strength parameters for the tailings.
- The current water pressure acting on the upstream face of the dike was equivalent to 2 m of artesian head. This is considered a valid assumption since this is what has been observed at the piezometers installed within the talik proximal to the dike.

The rigid block analysis was completed only for static conditions, compared to the conventional analysis which was completed for both static and pseudo-static loading conditions.

2.2.2 Conventional (Thawed) Analysis

The conventional, or thawed, analysis is illustrated on Figure V-2. The conventional analysis was completed for two scenarios:

- a dry (frozen) slope exhibiting no pore pressures; and,
- a frozen slope with a thawed zone at depth which contains excess pore pressures.

The first scenario is considered to represent a long-term case should the dike thaw and the retained tailings drain. Alternatively, it represents the tailings deposit when completely frozen and hence “dry”. The second scenario is considered to represent current conditions as determined from the geotechnical characterization of the tailings deposit discussed previously.

The analyses were completed for both static and pseudo-static (seismic) loading conditions.

2.3 Material Properties

The material properties were derived from the results of the geotechnical investigations conducted in 2002 and 2003. These properties are summarized in Table V-3.

2.4 Results

2.4.1 Rigid Block Analysis

The results of the rigid block analysis of the current conditions are illustrated on Figure V-3. The results indicate that the dike currently exhibits a Factor of Safety between 2.5 and 2.7, depending on the effective friction angle of the tailings. These values are in excess of the 1.5 value recommended for long term static stability by CDA (1999).

The pore pressures required for the Factor of Safety of the dike to be reduced to 1.5, 1.2 and

1.0 were back calculated using the lower bound strength parameters for the tailings and varying the pore pressures. The results are illustrated in Figure V-4. The results indicate that pore pressures equivalent to 15 m of artesian head would be required to reduce the Factor of Safety to 1.0. It should be noted that this level is approximately 13 m higher than the pore pressures currently recorded by the piezometers installed within the talik near the West Twin Dike.

2.4.2 Conventional Analysis

The conventional stability analysis was completed using the commercially available program Slope/W, under license from Geo-Slope International, Calgary, Alberta. The Morgenstern-Price method of stability analysis was utilized to conduct the analyses. A grid and radius full search analysis was completed for each case. The analysis was completed using a minimum slip surface depth of 3 m to ensure that the slip surfaces considered were significant to the overall dike stability (i.e. below the surficial shale retention dike).

Dry Conditions

The results of the “dry” conditions analysis are illustrated on Figure V-5. The dry conditions analysis was completed assuming no pore pressures exist within the slope and the effective friction angle of the tailings ranges between 27° and 33°. The results of the analysis indicate the stability of the dike exceeds recommended CDA guidelines (FS = 1.5).

A pseudo-static analysis of the dry conditions model was completed using a horizontal acceleration of 0.10g. This value corresponds to the 1:1000 year seismic event as provided by the GSC. The results of the pseudo-static analyses are illustrated on Figure V-5. The results indicate that the seismic stability of the West Twin Dike significantly exceeds recommended guidelines (FS = 1.1 to 1.2) even when considering the lower bound strength parameters for the tailings.

An additional pseudo-static analysis was completed to determine the value required to reduce the pseudo-static Factor of Safety to 1.0. The results indicate that a value of 0.25g is required to reduce the Factor of Safety of the West Twin Dike to 1.0, considering lower bound strength parameters for the tailings.

Thawed Zone

The “thawed zone” analysis was completed assuming a thawed zone of tailings exhibiting high pore pressures exists beneath the dike. This is similar to the piezometric pressures observed in the thawed zone at piezometer BGC03-14. The analysis was first completed using a range of effective friction angles for the tailings. The results of this analysis are illustrated on Figure V-6. The results indicate that the stability of the West Twin Dike exceeds the recommended guidelines even when considering the lower bound strength parameters.
A pseudo-static analysis of the thawed conditions model was also completed using a horizontal

acceleration of 0.10g. This value corresponds to the 1:1000 year seismic event as provided by the GSC. The results of the pseudo-static analyses are also illustrated on Figure V-6. The results indicate that the seismic stability of the dike exceeds recommended guidelines even when considering the lower bound strength parameters.

An additional pseudo-static analysis was completed to determine the horizontal acceleration value required to reduce the Factor of Safety from 1.2 static to 1.0 pseudo-static. The results indicate that a value of 0.16g is required to reduce the Factor of Safety of the West Twin Dike to 1.0 considering lower bound strength parameters for the tailings.

A second set of static stability analyses were completed using the lower bound strength parameters for the tailings and ranging the pore pressures within the thawed zone to achieve certain “benchmark” Factors of Safety. The results of these analyses are illustrated on Figure V-7. The results indicate 12 m of artesian head is required to reduce the Factor of Safety to 1.0 and achieve limit equilibrium. It should be noted that this level is approximately 9 m higher than the currently observed levels in the piezometers installed within the talik.

3.0 SLOPING RECLAMATION COVER STABILITY ANALYSES

Static and pseudo-static stability analyses were completed on the sloping reclamation cover that will be constructed in the area where the Test Cell tailings transition into the Reservoir. The analyses were conducted in a similar fashion to the analyses completed for the West Twin Dike using the Slope/W software package described previously.

A conventional or “thawed” analyses was completed by varying the effective friction angle of the tailings deposit and maintaining the effective friction angle of the shale at a constant value. Piezometric pressures were applied to the slope as shown on Figure V-8. The section used in the stability analyses is illustrated in section on Figure V-8, which assumes thawed conditions in the foundation zone.

The results of the analyses are provided on Figure V-9. The results indicate that under static conditions, the Factor of Safety of the slope ranges between 1.9 and 2.3, which is in excess of the 1.5 value required. Pseudo-static analyses were undertaken considering a horizontal seismic coefficient of 0.1g. The results of the analyses indicate that the Factor of Safety ranges between 1.2 and 1.5. It should be noted that the stability analyses for the Test Cell Dike and sloping cover were completed by limiting the slip surfaces to those which pass at least 2 m below surface.

Table V-1: Factors of Safety, Static Assessment (after CDA, 1999)

Loading Conditions	Minimum Factor of Safety	Slope
Steady-state seepage with maximum pond height	1.5	Downstream
Full or partial rapid drawdown	1.2 to 1.3	Upstream
End of construction before reservoir filling	1.3	Downstream and upstream

Table V-2: Typical Safety Factors for Impoundment Dams (after Mitchell, 1983)

Case	High Risk Dam	Low Risk Dam
[a] end of construction	1.3	1.3
[b] normal operation	1.5	1.3
[c] rapid drawdown	1.3	1.1
[d] earthquake loadings	1.2	1.1
[e] earthquake loadings in combination with [a], [b] or [c]	1.1	1.0

Table V-3: Summary of Material Properties for Stability Analyses

	Specific Gravity	ϕ'	Frozen Bulk Density (kg/m^3)
Tailings	4.1	27-33°	2800
Shale	2.65	30°	2200

APPENDIX VI
WEST TWIN OUTLET CHANNEL AREA
BOREHOLE LOGS
AND
GEOTHERMAL MONITORING DATA

APPENDIX VII
WATER QUALITY PROJECTIONS

Part A – Lorax 2004a
Part B – Nanisivik Mine Water Quality Listing
Part C – Estimate of Porewater Expulsion
Part D – Lorax 2004b
Part E – Results of the Water Quality Projection