REPORT ON

DETAILED DESIGN OF THE WEST TWIN DYKE SPILLWAY (WATER LICENSE PART G ITEM 7) NANISIVIK MINE CLOSURE STUDIES

Submitted to:

Breakwater Resources Ltd.
Suite 2000, 95 Wellington Street West
Toronto, Ontario
M5J 2N7

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Nanisivik Mine
A Division of CanZinco Ltd.
c/o Breakwater Resources Ltd.
Suite 2000, 95 Wellington Street West
Toronto, ON
M5J 2N7

Attention: Mr. Bob Carreau, CCEP
Corporate Manager, Environmental Affairs

RE: DETAILED DESIGN OF WEST TWIN DYKE SPILLWAY

Dear Mr. Carreau:

Enclosed are six (6) copies of our Report *Detailed Design of the West Twin Dyke Spillway*. The previous report dated February 6, 2004 has been updated to reflect the revised Reservoir water level. We trust you will issue the required number of copies directly to Nunavut Water Board and DIAND directly.

Please do not hesitate to contact us if we can be of any further assistance.

Yours truly,

GOLDER ASSOCIATES LTD.

Ken A. Bocking, P.Eng.
Principal

DGR/KAB/dh
Att.

EXECUTIVE SUMMARY

Breakwater Resources Ltd. (Breakwater) has retained Golder Associates Ltd. (Golder) to provide geotechnical and hydrological design support for the Nanisivik Mine Closure. This report provides part of the required documentation to support the Reclamation and Closure Plan (RCP) proposed by CanZinco Ltd., the operators of the mine.

Production of lead and zinc concentrates took place at the Nanisivik Mine between 1976 and 2002, when production ceased. The current owner of the mine, CanZinco Ltd., has been in possession of the mine since 1996. Since 2002, Nanisivik Mine has been operating under Nunavut Water Board Licence No. NWB1NAN0208, which provides for the continuation of on-site environmental protection activities during the development and submission, for approval, of a Final 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. This report specifically addresses Part G - Item 7 of the Water Licence, the design of the West Twin Dyke Spillway.

The principle closure objectives for the West Twin Disposal Area (WTDA) are to mitigate the potential long term environmental impacts and to return the land to a condition similar to pre-mining development. The closure concept is to restrict the transfer of oxygen to the tailings, and also to minimize the transport of any available metals. At closure, drainage from the WTDA will occur passively, without the need of manpower to operate siphons. For this reason, a dyke spillway and drainage channel will be constructed at closure to drain seasonal runoff from the Surface Cell to the Reservoir. The spillway has been designed to safely pass severe storm events. Tailings, which are currently exposed in the Reservoir will either be covered or re-located below a minimum water cover depth of 1.0 m. This will limit the potential for scouring of tailings at the outlet of the spillway.

The location of the spillway inlet, the spillway invert elevation and the tailings cover are intrinsically linked. Several options were considered before selecting the proposed configuration. Subsurface investigations done at the proposed location included boreholes, test pits, thermocouple installation, and laboratory testing of soil and bedrock samples. The geologic conditions are inferred to comprise till overlying frost-shattered bedrock overlying competent bedrock.

The spillway will convey run-off from the Surface Cell to the Reservoir. The Surface Cell will be graded and covered to direct runoff to the spillway. The spillway invert will be Elev. 384.0 m, and the normal water level in the Reservoir will be 370.2 m. The spillway has been designed to convey a 24-hour Probable Maximum Precipitation (PMP) storm event estimated to produce 140 mm of rainfall in 24 hours. The extreme daily snowmelt is estimated to produce 155 mm of runoff but the peak flow would be less due to a more even distribution over the event period. The

estimated peak flow over the Surface Cell Spillway is approximately 5.2 m^3/s, resulting in an peak water depth of approximately 0.52 m at the spillway inlet. The flow depth would decrease to about 0.31 m in steeper portions of the spillway.

The spillway will consist of a 6 m wide open channel with the base founded generally in intact bedrock. Where the base is located in frost-shattered bedrock or overburden, erosion protection consisting of rip rap stone with mean size 300 mm will be provided to a flow depth of 0.6 m. Rip rap bedding and filter layers will be provided beneath the rip rap to prevent the movement of soil through the coarse rip rap. The spillway outlet structure includes a plunge pool to force a hydraulic jump and a flared outlet channel so calm flows report to the Reservoir with minimal energy. Erosion protection will not be required where the channel base will be in intact bedrock.

The construction is estimated to involve about 48,800 m^3 of excavation and approximately 400 m^3 of fill for erosion protection. Construction supervision will be important to ensure compliance with design assumptions as design revisions may have to be implemented to accommodate variations in ground conditions. During construction, sampling and laboratory testing of both in situ materials and fill materials is required.

Routine surveillance is required to identify ice blockage or damming that could affect the performance of the spillway or result in a reduced freeboard to the crest of the West Twin Dyke. A service road will be provided along the full length of the spillway to allow for periodic inspection and maintenance which may include removal of soil resulting from slope movements and re-grading due to settlement from thawing.

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1.0 INTRODUCTION

Breakwater Resources Ltd. (Breakwater) has retained Golder Associates Ltd. (Golder) to provide geotechnical and hydrological design support for the Nanisivik Mine Closure.

This report provides the design basis for the proposed closure spillway between the Surface Cell and Reservoir at the West Twin Disposal Area. It forms a part of the required documentation to support the Final Reclamation and Closure Plan proposed by CanZinco Ltd., the operators of the mine.

This document and the information presented herein are provided in response to the requirements for component report G.7, the West Twin Disposal Area Surface Cell Spillway Design.

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., 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 at least 2004 or 2005. However, depressed international base-metal prices necessitated a re-evaluation of the mine production plan in fall 2001. 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 regularly by CanZinco Ltd. in response to the 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 the approaches and plans for reclamation of the mine 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 (the "Water Licence"). The license provides for the continuation of on-site environmental protection activities during the development and submission, for approval, of a 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 Part G of the Water License, as summarized in Table 1.

1.2 Specific Requirements of the Final Reclamation and Closure Plan

CanZinco's approach to closure and reclamation of the Nanisivik site includes the primary objective to "prevent progressive degradation and to enhance natural recovery in areas affected by mining". The specific requirements for the West Twin Dyke Spillway Design (the "Spillway design") come from two sources: Part G, Item 7 of the Water Licence and the needs of CanZinco.

Part G, Item 7 of the Water Licence, as excerpted below, provides the following requirements for the Spillway Design:

> *The Licensee shall submit to the Board for approval a report on the proposed spillway which shall include but not be limited to:*
>
> 1. *Overview of alternative spillway designs and justification for the preferred alternative;*
> 2. *Design hydrology;*
> 3. *Spillway geometry, with emphasis on the geometry where the spillway meets the covered tailings;*
> 4. *Geology along the centreline of the spillway;*
> 5. *Erosion protection measures;*
> 6. *A discussion on the effects of permafrost formation in shallow regions of the Reservoir portion, and of the effects of the entrainment of tailings within surface ice formation on the re-suspension of tailings and how these mechanisms will affect long term water quality; and*
> 7. *A discussion on how pore water expulsion from the freezing surface cell may affect the long-term Water quality of the Reservoir portion of the West Twin Disposal Area.*

This report addresses Items 1 through 5. Items 6 and 7 are addressed in the West Twin Disposal Area Closure Plan, as noted in Table 1.

2.0 BACKGROUND

2.1 Project Description

The Nanisivik Mine is located on the Borden Peninsula of Baffin Island in the Nunavut Territory (Figure 1). Production of zinc and lead concentrates took place at the site between 1976 and 2002 under the Nunavut Water Board License (NWB1NAN9702 dated July 31, 1997). Production at the site ceased in September 2002 due to project economics. CanZinco Ltd., has been in possession of the mine since 1998.

The Mill tailings were disposed into the West Twin Disposal Area (WTDA), which was constructed in the former Kuviku Lake. The East Twin Lake (former Quasaqtoq Lake) served as a source of potable water supply to the mill and town site that supported mining operations. Figure 2 shows the general arrangement of the site prior to construction.

2.2 West Twin Disposal Area (WTDA)

The WTDA which was built in 1976 in the location of the former Kuviku and Quasaqtoq Lakes by diverting the Twin Lakes Creek (at the outlet of carries flows from Twin Lake) into a diversion channel using an embankment dam.

An embankment and outlet weir were constructed to isolate the WTDA and allow control of the effluent outflow from the WDTA.

2.2.1 Existing Conditions at the WTDA

The existing conditions at the WDTA are shown on Figure 3.

WTDA is comprised on two cells: the Surface Cell which is contained by the West Twin Dyke, and the Reservoir, or West Twin Lake, which sits at the toe of the West Twin Dyke.

Currently, water which accumulates in the Surface Cell is siphoned via pipeline into the lower Reservoir (West Twin Lake) receives water siphoned out of from the Surface Cell. Outflow from the Reservoir to the adjacent Polishing Pond is via two submerged 1.2 m diameter culverts that pass through a rockfill causeway (constructed to carry the potable water pipeline) .

The Polishing Pond outflow and level is controlled by a stop log structure (weir). Discharge reports to Twin Lakes Creek which drains in the north-westerly direction and eventually discharges to the Strathcona Sound.

During operations, a reclaim pumphouse returned process water to the mill.

2.2.2 WTDA Closure Concept

The principle closure objectives for the Surface Cell are to mitigate the potential long term impacts and to return the land to its natural state. The closure concept is to restrict the transfer of oxygen to the tailings, and also to minimize the transport of any available metals. At closure, drainage from the WTDA will occur passively, without the need of manpower to operate siphons or pumps. For this reason, a dyke spillway and drainage channel will be constructed at closure to drain seasonal runoff. The spillway will also be designed to safely pass severe storm events.

Tailings, which are currently exposed in the Reservoir will either be covered or re-located below a minimum water cover thickness of 1.0 m. A normal water level (NWL) of 370.2 m will be maintained in the Reservoir, which is equivalent to the pre-mining level for West Twin Lake.

2.3 Previous Hydrologic Studies

Two hydrological studies for the Nanisivik Mine were carried out previously (Golder, 2002, and Golder, 2004). Those studies addressed design rainfall and snowmelt conditions, preliminary hydraulic design of the spillway from the Surface Cell, and hydraulic aspects of the various options of the post-closure tailings basin configurations.

The objectives of the hydrological modelling were to simulate peak discharges and water levels in the Surface Cell and Reservoir. The previously developed hydrological model has been updated to simulate the closure design conditions (Section 6.0).

3.0 SPILLWAY CONCEPT

The location of the spillway, the spillway invert elevation and the tailings cover are intrinsically linked. Issues related to the selection are:

- a higher spillway inlet elevation would likely result in a thicker cover material being placed. This occurs because of the irregular topography of the tailings surface in the Surface Cell and the need to grade the cover in a way that conveys water to the spillway inlet;
- a lower inlet elevation would require sub-excavation and grade control of frozen (and some possibly thawed) tailings which would be difficult; and
- a lower inlet elevation would result in excavations into the tailings that would have to be at a gentle slope for stability when thawed.

A spillway inlet options analysis was completed in order to determine an appropriate inlet location and approximate elevation considering the concerns listed above.

Four possible spillway inlet locations and the cover grading requirements for each were assessed. The four locations are summarised in Table 1 and are illustrated on Figure 4 and compared in Table 2.

Considerations and alternatives for the options are discussed below.

3.1 Alternatives Considered

Location 1 comprises a spillway that would pass over the crest of the existing West Twin Dyke. Two options were considered at this location, Option 1a with a spillway invert elevation of 388 m, (the current crest elevation of the dyke) and Option 1b with a spillway invert elevation of 386 m. Option 1b would require excavating into the dyke and the tailings upstream of the dyke. Both options would require significant regrading of the Surface Cell to direct surface water to the correct location. Both options would also result in a significantly larger quantity of shale for the cover because all topographic low spots between the dyke and the back of the Surface Cell would need to be completely filled in to prevent ponding of water.

If the water discharging from the Surface Cell was allowed to simply run down the face of the dyke, the gradient of the spillway from the crest of the dyke to the edge of the Reservoir would be between 20 and 25% and would require major rip rap application to prevent erosion into the dyke during the design storm event. The gradient could be reduced to 6% if a long spillway could be constructed into the Reservoir. However, this would require between 50,000 and 75,000 m^3 shale to construct and a would still require a significant amount of relatively large rip rap to prevent erosion during the design storm event.

Location 1 is considered to be unfeasible, because of the regrading requirements, the possible requirement of over-excavating tailings, increased cover thickness, high spillway grades and amount of shale fill required to reduce the spillway grade. Insulation would also be required at the spillway base to protect frozen tailings.

Location 2 would comprise a spillway passing through the dolostone outcrop near the south abutment of the dyke. Two options were considered; Option 2a would have a spillway invert elevation of 388, while Option 2b would have a spillway invert elevation of 386 m. Option 2a would require a significant excess of shale material required of the cover due to the high invert elevation and the need to maintain a gradient within the Surface Cell to direct water to the inlet location. Option 2b would likely require sub-excavating 1 to 2.5 m of tailings for an approximate length of 150 m to provide the minimum cover thickness and to maintain drainage gradients. Depending on the option, the highest point of the cut into the bedrock knoll would be between 12.5 and 17.5 m. The gradient of the spillway would range between 4.5 and 6.5%. Both gradients would require major rip rap application at the outlet as this would be constructed in soil. The rock in the knoll is frost shattered and of generally low quality. Because of the low rock quality, the slope would have to be excavated back and benched to prevent rockfalls into the spillway.

Due to the low quality of the rock, the high grade of the channel and the amount of blasting required, this location is considered to be unfeasible.

Location 3 is located at the natural drainage swale located near the south abutment of the dyke. The advantage of this option is that the natural swale connects the Surface Cell to the Reservoir from this location and the spillway gradient would range between 2 and 3%. Two options were considered: Option 3a would involve a spillway invert elevation of 387 m, while Option 3b would involve a spillway invert elevation of 385 m. Option 3a would require a significant excess of shale material required of the cover due to the high invert elevation and the need to maintain a gradient within the Surface Cell to direct water to the inlet location. Option 3b would likely require sub-excavating 1 to 2.5 m of tailings for an approximate length of 150 m to provide the minimum cover thickness while maintaining drainage gradients within the Surface Cell.

Due to the excess shale requirements and the need to sub-excavate tailings, Location 3 is considered to be unfeasible.

Location 4 is located at the south shore line of the Surface Cell. The main advantage of this option is that the inlet would be located in natural ground at a location towards which the current topography/ bathymetry naturally conveys water. The spillway invert elevation could be as low as 384.0 m. This would provide the minimum thickness of cover at this location and would not require over-excavation of tailings. This elevation and location would also permit a grading plan to be developed to convey runoff. This could generally be done with the minimum required cover thickness (i.e. the minimum quantity of cover material). The initial 100 to 200 m of the spillway would be built into a 7 to 14° side slope with an overall spillway grade of 2.0 to 2.5%. Due to the low grades, erosion protection requirements would be minimized.

3.2 Selection of the Preferred Alternative

Considering the advantages and disadvantages of each option, Location 4 was selected as the preferred spillway inlet location. A detailed grading plan was developed that would convey surface water towards this location via a number of drainage swales. The minimum grade of the drainage swales was set at 0.4%. This should be sufficient to maintain drainage in spite of heaving or frost settlement, which may occur during the first few years after construction as would be anticipated. It is anticipated that some initial maintenance and grading of the swales will be required following construction.

4.0 SUBSURFACE CONDITIONS IN SPILLWAY AREA

4.1 Drilling and Test Pitting Investigations

Fifteen (15) boreholes were drilled and eight (8) test pits were excavated in the proposed spillway area by BGC Engineering Ltd. (BGC) in 2002 and 2003. Boreholes were drilled to depths between 6 m and 10 m. Test pits were excavated to between 0.8 m and 1.4 m depth and were terminated generally when hard frozen ground was encountered. Both disturbed (thawed) samples and frozen core samples were collected and logged for lithology and geotechnical permafrost properties. Select samples were sent to the mine laboratory for index testing including moisture content, frozen bulk density and grain size distribution. Additional samples were selected for transport to Calgary and sent to a soil laboratory for additional lab testing. Thermocouples were installed in six of the boreholes.

Appendix A contains a summary of the BGC investigation progress in the spillway area. Summaries of the laboratory testing done on soil and rock samples are provided.

4.2 Interpreted Geologic Conditions

The geologic conditions are inferred from the drilling and test pitting investigations as summarized in Appendix A. In general, the stratigraphic section contains the following units:

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

At or near the Reservoir shoreline, massive ground ice has been observed.

The grain size distributions determined for overburden samples collected in the spillway area are shown on Figure 5. The overburden is generally composed of coarse-grained material, containing an average of 80% sand and gravel sized particles. Larger particles such as cobbles and boulders were observed in situ, but were not collected as part of the sampling program. Silt sizes are prominent in several locations. Visible ice was observed within the overburden as lenses, typically surrounding the larger clasts.

The bedrock encountered in the proposed spillway alignment was dolostone. The depth to bedrock ranged from 7.3 m in borehole BGC03-17, near the proposed outlet at the lake, to 0.6 m in borehole BH-7. The Rock Quality Designation (RQD) was observed to range between 0% for the frost shattered bedrock to 100% for the competent bedrock. The unconfined compressive strength of select core samples was found to vary between 68 and 186 MPa in axial point load tests.

4.3 Geothermal Monitoring

Regular monitoring of the geothermal instruments has been ongoing since May 2003. The results of the monitoring are illustrated graphically on the plots included in Appendix A. The monitoring data verifies the observations in the 2002 and 2003 drilling and test pitting programs that fully frozen conditions exist along the proposed spillway alignment. Ground temperatures at depth in the permafrost are approximately -12°C. The monitoring data indicates that the depth of the active layer is approximately 1.0 to 2.5 m below ground surface at this location.

5.0 HYDROLOGIC DESIGN CRITERIA

The spillway will convey run-off from the Surface Cell to the Reservoir. The Surface Cell will be graded and covered to direct runoff to the spillway. The spillway invert will be Elev. 384.0 m, and the normal water level in the Reservoir will be 370.2 m.

Key water levels for the Surface Cell and Reservoir and provided on Tables 3 and 4 respectively.

5.1 Method of Analysis

The GAWSER model (Guelph All-Weather Sequential Event Runoff model) was used for the hydrological simulation of the Nanisivik Mine tailings basin. 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: 127 ha.
- The elevation-discharge relationship for the tailings basin was developed from the hydraulic calculations of flow in the spillway channel. A hydraulic roughness coefficient (Manning's n) of 0.035 for the spillway channel was assumed.
- The elevation-storage relationship was developed based on the Surface Cell grading plan for closure (Figure 6).
- 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.

5.2 Design Precipitation Event

The extreme precipitation conditions at Nanisivik are summarized as follows: (Golder, 2002; Golder, 2003):

- The amounts of rainfall and snowmelt at the Nanisivik Mine are small. Therefore, a comparatively small spillway is required at the Nanisivik Mine, even to convey extreme floods (Golder, 2002).
- 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 event is estimated to be 155 mm. For comparison, a daily PMP event in Northern Ontario is approximately 500 mm to 700 mm (Golder, 2002).

A recommendation was made in the earlier hydrological study that the Surface Cell spillway be designed to convey a Probable Maximum Precipitation (PMP) event (Golder, 2002).

For the mine closure, the Surface Cell Spillway was designed to convey a flood resulting from the PMP storm event. The snowmelt distribution is more uniform than the rainfall distribution. Consequently, the PMP rain storm may produce a greater peak flow then the snowmelt event. The estimated PMP event at Nanisivik is 140 mm in 24 hours.

Hydraulic characteristics corresponding to a 100-year return period are also provided in the subsequent section for comparison. The estimated 100-yr return storm event is 41 mm in 24 hours.

5.3 Design Flow Conditions

The following hydraulic conditions will be observed at the West Twin Dyke Spillway under the PMP conditions (Table 5):

- The estimated peak flow over the West Twin Dyke Spillway is approximately 5.2 m^3/s.
- The estimated peak water depth is approximately 0.52 m at the 1% slope section of the spillway and 0.31 m at the 5% slope section of the spillway (Table 5).
- The estimated peak flow velocity is approximately 1.5 m/s at the 1% slope section of the spillway and 2.5 m/s at the 5% slope section of the spillway (Table 5).

For comparison, under the 100-yr storm conditions, the peak discharge over the spillway is estimated to be 1.35 m^3/s; the peak water depth is estimated to be approximately 0.23 m at the 1% slope section of the spillway and 0.14 m at the 5% slope section of the spillway; and the peak flow velocity is estimated to be approximately 0.9 m/s at the 1% slope section of the spillway and 1.5 m/s at the 5% slope section of the spillway (Table 5).

6.0 DESIGN CONSIDERATIONS

The main purpose of the spillway is to provide passive drainage of runoff out of the Surface Cell to prevent excessive ponding and a potential heat source to the underlying tailings. The spillway will be an open channel generally in bedrock. Precise control of the water level in the Surface Cell is not a requirement so a cut-off wall is not required. Seepage though fractured rock at the base of the spillway (below Elev. 384 m) is of no consequence.

The spillway design is presented on Figures 6 and 7. Figure 6 shows the spillway in plan view while Figure 7 shows a longitudinal section and cross-sections at various locations.

6.1 Spillway Geometry

The spillway will consist consists of a 6 m wide open channel approximately 574 m in length. The vertical alignment was selected to minimize excavation, while still situating the invert of the channel within non-shattered bedrock to the extent possible. The base width selected is a reasonable minimum to allow for excavation using blasting.

The spillway inlet has an invert elevation of 384 m as required by re-grading considerations. The outlet of the spillway is designed to match the planned normal water level of 369.0 m in the Reservoir. Scour of the tailings near the outlet is not a concern because tailings will be re-located to below Elev. 368.0 m. Erosion protection will be placed on native soil at the end of the spillway.

A service road will be provided along the full length of the spillway to allow for periodic inspection and maintenance. The access road will be located on the uphill (west) side of the spillway because of the potential for slope movement due to solifluction is the greatest on the uphill side. This will allow for removal of slumped material before it enters the spillway. A road crossing through the spillway will be provided at Ch 0.045 (Dwg. No.2) for access to the West Twin Dyke. A turnaround will be provided at the outlet end of the spillway.

6.2 Geotechnical Considerations

The spillway is shown in plan on Figure 6 and in section on Figure 7. Boreholes and test pits put down previously in the area are shown along with the inferred bedrock surface on Figure 7.

Based on the available information, the base of the spillway will be in bedrock. Where possible, the 0.6 m (maximum) flow depth will be located in intact bedrock to enhance physical stability (and thus maintenance requirements). Rip rap erosion protection will be provided wherever frost shattered rock or overburden is encountered within a 0.6 m flow depth above the spillway invert. Riprap will not be required where the channel base will be in intact bedrock.

Excavated slopes in intact bedrock will be approximately 0.1 horizontal:1 vertical. In frost shattered bedrock, flatter slopes of 1 horizontal:1 vertical will be built. Overburden slopes will be cut at 4 horizontal:1 vertical for long term stability considerations. No erosion protection is required on these cuts slopes, except within the 0.6 m flow depth.

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

The outlet channel will have rip rap to Elev. 370.8 m. This is equivalent to one design wave height times 1.5 above the normal water level in the Reservoir. It is also equivalent to the maximum water level in the Reservoir predicted under extreme flood conditions.

6.3 Alignment

The alignment is dictated by subsurface conditions. For the most part, the base of the spillway will be located in intact rock. Where the flow depth is not contained within intact rock then rip rap erosion protection will be placed. Where overburden is encountered in the invert, layers of rip rap bedding and a filter layer will be placed to prevent fines from moving through the bedding.

The spillway consist of three segments:

- 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 218 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 with a sloping floor will carry the flow to the Reservoir.

The alignment of the outlet portion of the spillway was selected to avoid the ground ice noted in boreholes BH1 and BGC 03-06. Section 8.1 addresses the contingency of ice being found in the excavation.

6.4 Hydraulic Considerations

The hydraulic analysis of the spillway performance demonstrated the following (Golder, 2002):

- Sub-critical flow conditions will prevail at the flat and 1% slope sections of the spillway.

- Super-critical flow conditions will prevail at the 5% slope sections of the spillway (Golder, 2002).

- The plunge pool will force a hydraulic jump resulting in sub-critical flow conditions in the outflow channel.

Table 5 provides a summary of the flow conditions for 100-year return and PMP storms.

6.4.1 Outlet Structure

Because of the super-critical flow conditions at the lower section of the spillway, a hydraulic jump will take place at the outlet of the spillway to the Reservoir. An energy dissipation structure was proposed at the spillway outflow to prevent the outlet erosion (Figures 6 and 7). The outlet channel is flared to distribute flow calmly.

Where the outflow channel terminates, erosion protection will be provided on native soils to minimize scour (Figure 7). Disturbance of tailings (relocated to below Elev. 369.2 m) is not expected.

The plunge pool floor elevation will be 370.6 m, which is 0.4 m above the normal water level in the Reservoir. It is intended that the plunge pool drain under no flow conditions to defer icing. The outflow channel floor elevation will slope from Elev. 370.9 to Elev. 370.2 m at about 1%.

The outlet channel has rip rap to Elev. 370.8 m. This is equivalent to 1.5 times the design wave height above the normal water level in the Reservoir. It is also equivalent to the maximum water level in the Reservoir predicted for extreme flood conditions (i.e. 24-hour PMP rainfall event).

6.4.2 Erosion Protection

Table 6 provides a summary of the erosion protection requirements based on the material exposed in the cut and spillway gradient at various locations.

<u>Spillway Chute Protection</u>

The spillway 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 velocities on the spillway ranged from 0.9 m/s to 2.5 m/s; the proposed channel side slopes were 1H:1V.

For the 1% slope spillway section, the estimated median stone diameters (i.e. D_{50} size) were approximately 4 cm for a 100-year precipitation event and 11 cm for the PMP event. For the 5% slope spillway section, the estimated median stone diameters were approximately 11 cm for a 100-year precipitation event and 30 cm for the PMP event.

Table 6 provides a summary of erosion protection requirements for various locations along the spillway.

Spillway Outlet Protection

As mentioned previously, super-critical flow conditions will occur in the steep reach of the Surface Cell Spillway, and hydraulic jump will occur at the plunge pool. An energy dissipation structure will be constructed at the spillway outlet to prevent the outlet erosion (Figure 7).

7.0 CONSTRUCTION CONSIDERATIONS

7.1 Alignment and Field Modifications

The spillway alignment was chosen to avoid the ground ice encountered in the boreholes near the Reservoir shoreline. Nonetheless, there is some chance that ground ice may be encountered during the construction of the outflow channel. If so, the ground ice will be subexcavated and the void will be backfilled with granular fill covered with rip rap bedding and rip rap.

Alternatively, the alignment could be adjusted as approved by the Field Representative. Frost shattered rock will be flatter laid back somewhat relative to the intact rock. (Field adjustments can be made based on actual condition encountered).

7.2 Construction Materials

Rip rap, erosion protection and filter zones (Types 1, 2 and 3 respectively) have been designed to act as a reverse filter. The gradation of these materials is shown on Table 7.

7.3 Construction Quantities

Estimated construction quantities are provided in Table 7. Work will involve about 48,800 m^3 of excavation comprising 3,400 m^3 in intact rock and 45,400 m^3 in overburden and frost shattered rock. Approximately 400 m^3 of fill will be required.

Table 8 provides a breakdown of the excavation and fill quantities.

7.4 Construction Quality Control and Quality Assurance

Construction supervision will be important to ensure compliance with design assumptions. Design revisions may have to be implemented to accommodate variations in rock conditions (both intact and frost-shattered), seepage inflows or otherwise. During construction, sampling and laboratory testing of both in situ materials and borrow (fill) materials used in construction will be r required.

8.0 PERFORMANCE MONITORING

In accordance with the "Guidelines for Abandonment and Restoration of Mines in the NWT", a performance monitoring program has been developed to provide a means of measuring the effectiveness of the closure measures. The monitoring requirements during reclamation and closure periods are fully detailed in the monitoring requirements report satisfying Water License Item G.9 (Table 1).

In general, the monitoring program provides for environmental monitoring during the 2 year Reclamation Period and for a subsequent 5 year Closure Period. During the Reclamation Period, a manpower presence at the mine site is anticipated for construction monitoring and maintenance purposes. This presence will enable the proposed monitoring programs to be carried out by the on-site personnel under the direction of an Environmental Coordinator, in consultation with a professional geotechnical engineer. During the Closure Period, environmental monitoring will be conducted to determine the success of reclamation measures. Continuous man power presence at the mine site is not planned during the closure period and environmental monitoring programs will be carried out on site visits and possibly utilizing local field assistants.

8.1 Routine Surveillance

Visual observation of the spillway should be part of the routine surveillance program. In particular, routine inspections should be completed to identify and correct blockage due to sloughing of rock or overburden slopes and integrity of the erosion protection should be done. Spillway infill material should be removed from the spillway periodically to ensure adequate flow capacity in the spillway. Some settlement and deformation may be expected in the overburden and frost-shattered bedrock units following melting of ground ice. As a result, some maintenance and re-grading will be required for the initial years following construction.

It will also be critical to observe the flow conditions at the inlet portion of the spillway. Any ice damming or build-up will have to be removed to ensure safe conditions for the West Twin Dyke. As a minimum, at least 2 m of "dry" freeboard must be available to the dam crest of West Twin Dyke at all times.

8.2 Maintenance

Maintenance will be required to repair erosion observed, as well as rock fall or soil slope slumping. An access road has been provided to facilitate this.

Removal of ice that may accumulate in the inlet portion of the spillway (to about ch. 0+200, Figure 6, where the base gradient is low) is also required.

GOLDER ASSOCIATES LTD.

David G. Ritchie, P.Eng.
Geotechnical Engineer

Alex Gordine, P.Eng.
Senior Water Resources Engineer

Ken A. Bocking, P.Eng.
Principal

DGR/AG/KAB/dh

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Golder, 2002. Report on *Hydrological Study*. *Nanisivik Spillway Design.* Project 011-1838. Mississauga, Ontario, Canada. February 2002.

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Smith, 1995. C. D. Smith. H*ydraulic Structures*. University of Saskatchewan, 1995.

TABLE 1
COMPONENT FCRP REPORTS

Water License Reference	Report
G3	Final Closure and Reclamation Plan
G4	Reclamation Cover Designs
G5	West Twin Disposal Area Talik Investigation
G6	Borrow Areas Development and Closure Plan
G7	West Twin Disposal Area Surface Cell Spillway Design
G8	Waste Rock and Open Pit Closure Plan
G9	Reclamation and Closure Monitoring Plan
G12	Annual Review of Reports G3 to G9 and Submission, for Approval, of Proposed Modifications
G13	Report on Environmental Site Assessment (ESA) Program
G14	Human Health and Ecological Risk Assessment (HHERA)
G15	West Twin Disposal Area Closure Plan
G16	Underground Mine Solid Waste Disposal Plan
G17	Landfill Closure Plan
G20	Annual Review of Reports G15 to G17 and Submission, for Approval, of Proposed Modifications
G21	Annual Reclamation Liability Cost Update
G22	2007 Terms of Reference for Comprehensive Assessment of Mine Site Remediation

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TABLE 2
COMPARISON OF SPILLWAY ALTERNATIVES

Location	Option	Inlet Elevation (m)	Over excavation of tailings required?	Estimated Amount of Cover Material Required
1	1a	388	No	Most ≈ 1,00,000 m^3
	1b	386	Yes	≈ 750,000 m^3
2	2a	388	No	≈ 900,000 m^3
	2b	386	Yes	≈ 750,000 m^3
3	3a	387	Yes	≈ 900,000 m^3
	3b	385	Yes	≈ 600,000 m^3
4	4	384.5	No	Least ≈ 550,000 m^3

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TABLE 3
KEY WATER LEVELS – SURFACE CELL

Surface Cell

Normal Water Level:	Elev. 384 m
Water Level during 100-year storm conditions:	Elev. 394.2 m
Peak Water Level (during design storm (PMP):	Elev. 384.6 m

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TABLE 4
KEY WATER LEVELS – RESERVOIR

Normal Water Level:	Elev. 370.2 m
Water Level during 100-year storm conditions:	Elev. 370.4 m
Peak Water Level (during design storm [PMP]):	Elev. 370.8 m

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TABLE 5
SUMMARY OF PEAK FLOWS
VELOCITIES AND DEPTHS

Structure:	Surface Cell Spillway			
Design Conditions:	100-yr		PMP	
Peak flow, m^3/s	1.35		5.2	
Channel Slope, %	1%	5%	1%	5%
Width, m	6	6	6	6
Peak Flow Depth, m	0.23	0.14	0.52	0.31
Peak Flow Velocity, m/s	1.0	1.6	1.7	2.8

TABLE 6
EROSION PROTECTION REQUIREMENTS

Spillway Gradient	Material Exposed	Erosion Protection Layers (Thickness)
0 - 1% Slope	Intact Rock	None
	Frost Shattered Rock	Type 2 (0.45 m)
	Overburden	Type 2 (0.45 m) ON Type 3 (0.3 m)
1% - 5% Slope	Intact Rock	none
	Frost Shattered Rock	Type 1 (0.6 m) ON Type 2 (0.45 m)
	Overburden	Type 1 (0.6 m) ON Type 2 (0.45 m) ON Type 3 (0.3 m)

Notes:
1. Thicknesses are measured perpendicular to slope
2. Material Descriptions
   - Type 1 – Rip Rap – Boulders, D_{50} = 300 mm
   - Type 2 – Erosion Protection – Cobbles and Boulders, D_{50} = 100 mm
   - Type 3 – Filter – Sandy Gravel (i.e. Twin Lakes Gravel)
3. D_{50} – mean particle size

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TABLE 7
PRELIMINARY MATERIAL SPECIFICATIONS

Sieve Size	Material Type and Percent Passing		
	Type 1 - Rip Rap	Type 2 - Bedding /Erosion Protection	Type 3 - Filter (Twin Lakes Gravel
600	100	-	-
450	100-0	-	-
300	20-0	100	100
150	0	100-30	100-85
75		28-0	100-75
50		15-0	100-65
25.4		0	70-40
19			67-40
12.5			55-25
9.5			45-12
4.75			35-0
2.38			30-0
1.16			23-0
.6			20-0
.3			13-0
.15			8-0
.075			5-0

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TABLE 8
ESTIMATED CONSTRUCTION QUANTITIES

Location	Construction Items					
	Excavation Volumes (m^3)			Fill Volumes (m^3)		
	Overburden Excavation	Frost-Shattered Bedrock	Intact Bedrock	Rip rap (Type 1)	Erosion Protection (Type 2)	Bedding (Type 3)
Inlet / Chute	21,800	9,800	3,400	225	250	75
Plunge Pool	1,500	300	-	200	150	75
Outlet Channel	10,050	1,950	-	1,275	1,150	250
Sub-totals	**33,350**	**12,050**	**3,400**	**1,700**	**1,500**	**400**
Grand total	**48,800 m^3**			**3,550 m^3**		

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APPENDIX A

SUMMARY OF GEOTECHNICAL INVESTIGATIONS