BGC ENGINEERING INC.
AN APPLIED EARTH SCIENCES COMPANY

BREAKWATER RESOURCES LTD.

HYDRAULIC STRUCTURES
AS-BUILT REPORT

NANISIVIK MINE, NU

FINAL REPORT

PROJECT NO.: 0255-012-05
DATE: MAY 30, 2008

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Project No. 0255-012-05
May 30, 2008

Mr. Bob Carreau
Vice President, CSR and Sustainability
Breakwater Resources Ltd.
Suite 950, 95 Wellington Street West
Toronto, ON
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RE: NANISIVIK MINE HYDRAULIC STRUCTURES, AS-BUILT REPORT

Dear Mr. Bob Carreau:

Please find attached our above referenced report dated May 30, 2008. This report presents the as-built information collected by BGC Engineering Inc. during the construction of various hydraulic structures at the Nanisivik mine site.

Should you have any questions or comments, please contact me at the number listed above.

Regards,
BGC Engineering Inc.
per:

Geoff Claypool, P.Eng.
Geological Engineer

encl: Final Report, Figures, Appendices

GKC/sf

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LIMITATIONS OF REPORT

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1.0 INTRODUCTION

1.1 Background

Nanisivik Mine is wholly owned by CanZinco Limited ("CanZinco"), which is a division of Breakwater Resources Limited ("Breakwater"). The Nanisivik Mine is located on the Borden Peninsula on northern Baffin Island in the Canadian Arctic at approximately 73º north latitude (Figure 1). The mine site is located on the south shore of Strathcona Sound, approximately 30 kilometres from Admiralty Inlet.

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. Mining operations were permanently ceased at that time.

The Final Closure and Reclamation Plan (FCRP) for the Nanisivik Mine was submitted to the Nunavut Water Board (NWB) by CanZinco in March 2004. Included in the appendix of the FCRP were the following reports;

- Detailed Design of the West Twin Dyke Spillway, prepared by Golder Associates Ltd., March 2004.
- West Twin Disposal Area Closure Plan, prepared by BGC Engineering Inc., Gartner Lee Ltd., and Golder Associates Ltd., March 2004.

These reports provided the following information:

- An engineering design for the construction of the West Twin Dike Spillway and the West Twin Outlet Channel,
- Reclamation plan for the East Twin Creek Diversion Channel,
- Recommended construction and engineering supervision practices, and
- A performance monitoring plan.

The regulatory review process included a technical meeting in Yellowknife in May 2004 and a public hearing in Arctic Bay in June 2004. The NWB conveyed its approval of the FCRP for Nanisivik Mine in a letter to Breakwater dated July 6, 2004.

1.2 BGC Scope of Work

BGC was retained by Breakwater to conduct on-site field monitoring during construction of hydraulic structures. During construction, a BGC Field Representative was on-site to document the following aspects of the reclamation work:

- construction schedule;
- material quantities;
- QA/QC test results;
- technical decisions and field modifications made to the original design and the associated rational;
- construction photos; and
- to provide direction to surveyors such that sufficient survey information is collected to prepare accurate as-built drawings of each reclamation measure.

The information recorded by the Field Representative forms the basis of this current report which provides a summary of as-built information collected during construction of the hydraulic structures at Nanisivik Mine.

2.0 DESIGN CONCEPTS AND OBJECTIVES

The general design concepts and objectives for each element were documented within BGC et al. (2004). The location of each reclamation measure is illustrated on Figure 2.

West Twin Dike Spillway

The general design objective of the West Twin Dike Spillway was to passively transfer water from the upper Surface Cell to the lower Reservoir. This was to prevent excessive ponding from occurring on the Surface Cell cover, which could have detrimental geothermal effects on the cover. The West Twin Dike spillway was to be an open channel excavated through soil and rock located at the south end of the Surface Cell. Additional details regarding the design are provided in Section 4.1.

West Twin Lake Outlet Channel

The general design objective of the West Twin Outlet Channel was to provide elevation control for the water level in the Reservoir while maintaining a low maintenance, flow-through system at the West Twin Disposal Area. This was to be implemented by removal of the existing control structure and construction of an open channel and overflow weir structure. Additional details regarding the design are provided in Section 4.2.

East Twin Diversion Channel Upgrade

The East Twin Diversion Channel is an existing structure which was constructed during mining operations to divert Twin Lakes Creek from its natural drainage course which originally drained directly into West Twin Lake. Prior to 2005, localised erosion of the face of the dike was observed. As such, a reclamation plan was developed to stabilize the areas where erosion had been observed and upgrade the dike in these areas to limit the potential for erosion in the future. Additional details regarding the reclamation plan for the East Twin Diversion Channel are provided in Section 4.3.

3.0 CONSTRUCTION PRACTICES

Construction of the reclamation measures at the Nanisivik Mine was challenging due to the remoteness of the site, short construction season, extreme climatic conditions and various complexities associated with the construction materials and ground conditions encountered throughout the construction process. Thus, a comprehensive, but adaptive, approach to reclamation was required to ensure the reclamation measures were constructed to the original design intent. The following sections summarize the personnel, equipment, and construction practices successfully utilized to implement the FCRP at Nanisivik Mine.

3.1 Construction/Administration Personnel

The Owner, Breakwater, was represented on site during the reclamation period by the following personnel:

- Site Manager – Mr. Murray Markle.
- Contract Administrator – Mr. Mike Weirmeir.
- Geotechnical Consultant – BGC Engineering Inc. (Project Engineer – Mr. Geoff Claypool, P.Eng.)

The primary Remediation Contractor was ATCON Construction Inc. (ATCON), who were on-site between August 2004 and October 2005. They were represented on site by the following personnel:

- Project Manager – Mr. Bruce Surek.
- Site Foremen – Mr. Leighton Taylor, Mr. Peter Sims and Mr. Ron Leblanc.

Additional personnel and subcontractors were utilized for small scale projects at various stages of the reclamation process.

3.2 Construction Equipment Summary

The following construction equipment was transported to site by ATCON, via the sea lift, in August 2004:

- 4, CAT 775D haul trucks;
- 1, CAT D8R dozer;
- 1, CAT 385ME excavator;
- 1, CAT CS-583C vibrating drum roller compactor; and,
- 2, SCH 5000 blasting drills (subcontractor Consbec).

The following mine fleet equipment was also utilized by the Contractor during construction:

- 1, CAT D8R dozer;
- 1, CAT D8N dozer
- 2, 980 series loaders;
- 1, John Deere 892LC excavator; and
- 1, CAT 140H grader.

3.3 Construction Monitoring

A BGC Field Representative was on-site during construction of the West Twin Dike Spillway, the West Twin Outlet Channel and the reclamation of the East Twin Creek Diversion Channel. The Field Representative was responsible for implementing any required QA/QC measures, documenting construction of each element and providing technical direction to the Contractor, as required. During construction of each element, daily reports were developed by the Field Representative documenting activities undertaken on a daily basis including estimated volumes of materials, problems encountered during construction activities and any technical decisions made. Detailed weekly reports were also developed by the Field Representative and submitted to the Contract Administrator. These weekly reports contained the following:

- A summary of the weekly activities including an estimate of material volumes;
- Problems encountered during construction activities;
- Technical decisions made and the rationale behind them;
- A summary of visual observations made during construction; and
- A summary of any other reclamation activities undertaken at site by the mine staff.

These daily and weekly reports have been compiled but are not included within this current report. The information contained within these daily and weekly reports has been used as a basis for developing the as-built report.

4.0 AS-BUILT RECORD

This section describes the specific activities that were undertaken as part of the construction of the various hydraulic structures at Nanisivik Mine undertaken between 2004 and 2006. For each reclamation measure, the following information is provided:

- General description of the pre-reclamation conditions;
- Detailed description of construction activities undertaken including schedule, equipment useage, material volumes, material placement techniques and difficulties encountered during construction;
- Documentation of design modifications and associated rational; and,
- As-built information including survey information and results of QA/QC testing.

The following sections provide the information outlined above as they apply to each specific reclamation element.

4.1 West Twin Dike Spillway

4.1.1 Pre-Reclamation Conditions

Prior to construction of the West Twin Dike Spillway, surface water was transferred from the Surface Cell to the Reservoir using siphons. No natural drainage occurred from the Surface Cell to the Reservoir due to the confinement provided by the surrounding topography.

The spillway was constructed within a natural drainage path of seasonal run-off from an area adjacent to, and upslope of, the Surface Cell. The surface topography was sloping towards the Reservoir at grades ranging from 5 to 10%. The drainage path was approximately 40 m wide and was confined along each side by outcrop of dolostone bedrock. Surface soils consisted of a thin layer of organic material (generally less than 5 cm) underlain by a mixture of sand, gravel and cobbles. Based on the results of the geotechnical drilling investigations undertaken in the spillway area, the overburden was generally observed to be ice-poor, except near the Reservoir shoreline where significant ground ice was observed.

4.1.2 Design Criteria

The design criteria for the West Twin Dike Spillway were documented in Golder (2004). The spillway was to consist of a 6 m wide, open channel approximately 574 m long. The spillway was designed to convey a flood resulting from the Probable Maximum Precipitation (PMP) storm event.

The spillway was to consist of three segments:

- The **inlet portion** of the spillway (approximately 120 m long) was to be flat (slope of 0%) with a nominal invert elevation of 384.0 m. The spillway inlet was to be constructed to match the Surface Cell cover contours.
- The **chute portion** of the spillway was to contain two sections. The upper chute (approximately 130 m long) was to be sloped at approximately 1% grade. The lower chute (approximately 218 m long) was to be sloped at 5% grade.

- The **outlet portion** of the spillway (approximately 106 m long) was to consist of a plunge pool and an associated outflow channel. The plunge pool was to have a floor elevation of 370.6 m. The grade of the outflow channel was to be constructed at approximately 1%.

The following hydraulic design criteria were also provided in Golder (2004):
- The estimated peak flow in the spillway was 5.2 m^3/s.
- The estimated peak water depth in the spillway, as designed, was 0.52 m in the upper chute and 0.31 m in the lower chute;
- The estimated peak flow velocity in the spillway, as designed, was 1.7 m/s through upper chute and 2.8 m/s through the lower chute.

The following additional design characteristics were also provided in the design report:
- Excavated slopes in intact bedrock were to be sloped at approximately 0.1(H):1(V).
- In frost affected bedrock, a slope of 1(H):1(V) was recommended.
- Overburden slopes were to be sloped at 4(H):1(V) and armoured within the design flow depth.
- The outlet area was to be armoured with riprap to a maximum elevation of approximately 370.8 m.
- The size of riprap was to vary, depending on the underlying materials and slope gradient of the section in which they were being applied, as per Table 1.

Table 1 Erosion Protect 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 Riprap – Boulders, D_{50} = 300 mm
   - Type 2 Riprap – Erosion Protection – Cobbles and Boulders, D_{50} = 100 mm
   - Type 3 – Filter – Sandy Gravel

4.1.3 Spillway Construction

Select photos of the construction process including pre-reclamation conditions, spillway excavation and armouring are illustrated on the photos included in Appendix I. The as-built drawings showing the construction details are provided on Figures 3 through 5. The following sections provide details on the construction schedule, construction materials, construction methodologies and the as-built survey information.

<u>Construction Details</u>

General

Construction of the West Twin Dike Spillway was undertaken by the primary Contractor ATCON, with some assistance from the mine site staff. Construction of the spillway was completed between May 12 and 27, 2005.

Prior to excavation, the materials were drilled and blasted typically using a 5.5 by 5.5 m dice grid pattern (see Photo 1) using ANFO as a blasting agent. A 0.6 m sub-drill was typically implemented to ensure the blasted material could be excavated to the design grade.

The materials were subsequently excavated using the 385ME excavator and transported to various stockpiles, depending on the material type. Overburden was mostly transported to the Mt. Fuji shale quarry and stockpiled in the base of the quarry (see Photo 3). This material was used to reclaim the quarry by grading the floor to prevent ponding. The frost-affected rock, denoted by its smaller size and red stained surface colour (see Photo 5), was transported to a stockpile adjacent to the spillway outlet area for later use as Type 2 riprap. The competent rock material, denoted by its larger particle size and lack of soil staining, was transported to a separate stockpile adjacent to the spillway outlet for later use as Type 1 riprap (see Photo 6). Grain size distribution analyses undertaken on both materials confirmed these materials were appropriate for use as riprap. The results of these analyses are provided on Figure 6.

It should be noted that throughout this section, standard hydrological convention is used when describing the sides of the excavation. All references to right hand side or left hand side are described as if looking in a downstream direction.

Spillway Inlet

The excavation of the inlet portion of the spillway is photographically documented on Photos 7 through 12. The inlet portion of the spillway was primarily excavated in competent dolostone bedrock. The maximum depth of excavation ranged from approximately 4 m along the left side of the channel to 8 m along the right side of the channel. Due to the significant depth of excavation, a bench was constructed along the right side of the channel, separating the slope approximately into two halves. The crest of the excavation along both sides of the channel were scaled using the excavator to remove loose rock.

The spillway inlet is approximately 100 m long and is essentially flat. The base width of the spillway through this section is approximately 6 m. The side slopes are essentially vertical in competent bedrock, but are sloped near the crest of the excavation where frost-affected bedrock was encountered.

Access Ramp

An access ramp was constructed on both sides of the spillway excavation, approximately 200 m from the inlet (see Photo 13). The ramp was excavated primarily in dolostone bedrock on the left side of the spillway and a mixture of overburden, frost-affected bedrock and competent bedrock on the right side of the spillway. The ramp was sloped at an angle of approximately 4(H):1(V) and a 0.3 m thick layer of armour rock was applied to the surface of the access ramp, on both sides of the channel, to improve trafficability.

Spillway Chute

The excavation of the inlet portion of the spillway is photographically documented on Photos 15 through 26. Excavation of the spillway chute started at the plunge pool and proceeded towards the access ramp. As the excavation proceeded, it was observed that a diabase dike was encountered through the majority of the sidewalls of the excavation (see Photos 15 and 16). The quality of the diabase material was observed to range from slightly weathered, to being locally altered to ice-rich clay. As the excavation reached approximately Station 290 (290 m from the inlet at the Surface Cell), it was apparent that the quality of the rock encountered in the side walls of the excavation was sufficiently poor that the stability of the excavation, as originally designed, could not practically be maintained in the long term (see Photos 17 and 18). As such, the design grades were altered to limit the exposure of diabase in the side walls and provide a stable slope configuration. The revised grades were achieved by backfilling the excavation with rockfill (Type 1 and Type 2 riprap) such that the remaining exposure of diabase rock in the side walls was limited in height to a maximum of approximately 1 m. During backfilling, approximately 1 m of Type 2 riprap was placed on the bottom of the excavation. Type 1 riprap was then placed in the excavation to the revised grade elevation. The average thickness of backfill through the chute section was approximately 2 m, with a maximum fill thickness of 3.5 m. The remnant exposed side walls were treated as frost affected bedrock and were armoured with Type 1 riprap.

As a result of the backfilling, the design grades within the spillway chute were altered. The grade along the base of the spillway through the chute section ranges from approximately 1% in the upper chute, to an average of 6% in the lower chute.

Along the right hand side of the spillway excavation, a bench was created at the interface between the diabase dike and the overburden (see Photos 23 and 24). The construction of the bench allowed the frozen overburden materials to be appropriately sloped, once the materials had thawed. Once thawed, the overburden materials were sloped to an average slope of 4(H):1(V), as per the original design. Additionally, armour rock which had been used to construct the adjacent access road was also placed along the face of the overburden slope, to further enhance stability.

The left hand side of the spillway chute abutted to a large bedrock outcrop. During excavation, these materials were also observed to be locally unstable at the proposed design slopes. As such, a portion of the dolostone outcrop was excavated by drilling and blasting to obtain a stable slope configuration (see Photos 25 and 26).

Plunge Pool

The plunge pool was constructed at the transition between the chute and outlet sections of the spillway. The base of the plunge pool was founded in frost-affected dolostone bedrock. Additional riprap was applied to the base and side slopes of the plunge pool area to provide additional erosion protection during high flow events.

The plunge pool is approximately 25 m long and 7 m wide. The grade of the upstream and downstream slopes of the plunge pool are approximately 4(H):1(V) (see Photo 29).

Spillway Outlet Channel

Construction of the spillway outlet channel was completed primarily through frozen overburden (see Photos 27 and 28). Based on the results of previous geotechnical drilling investigations in this area, the overburden was known to locally contain excess ground-ice. As such, the spillway outlet was over-excavated, in both depth and width, and backfilled with armour rock to prevent erosion and maintain stability.

The outlet channel is approximately 95 m long and varies in width from approximately 7 m at the plunge pool to 12 m at the Reservoir shoreline. The grade of the outlet channel is approximately 2%. The side slopes are approximately 4(H):1(V), as per the original design.

Water Deflection Berms

A water deflection berm was constructed upslope from the access ramp (see Photo 33). The berm is intended to direct surface water from the upslope areas towards the spillway at a controlled location. The berm is approximately 1.5 m high, 80 m long and is constructed of rockfill derived from the spillway excavation. The berm directs surface water to the edge of the spillway where additional armour rock has been applied to the crest of the spillway excavation.

Access Road

An access road was constructed along the right hand side of the spillway excavation. The road was constructed of blast rock obtained from the spillway excavation. The road is approximately 0.3 m thick and 10 m wide. A small ditch (approximately 0.3 m wide and 1.0 m wide) was excavated along the upslope edge of the access road to provide drainage pathway for surface water (see Photo 34).

4.1.4 Verification of Design Intent

While the spillway is considered to have been constructed to its design intent, a number of design modifications were made during construction due to the ground conditions that were encountered:

- The poor ground conditions encountered within the chute section of the spillway required the slope gradients to be modified in order to permit a stable configuration to be constructed. Although the gradients were slightly increased, the rockfill placed within the excavation should prevent any excessive erosion of the base of side slopes of the spillway from occurring.
- The gradient of the spillway outlet channel was constructed to 2%, which is slightly steeper that the 1% in the original design.
- Native overburden materials were considered to meet specifications for Type 3 bedding material. This was confirmed with grain size distribution analyses.
- In overburden slopes at the spillway outlet, Type 1 riprap was placed directly on the ground surface. This was due to the lack of available Type 2 riprap, which had already been placed in the base of the spillway. This was off set by increasing the thickness of the riprap layer to as much as 1 m thick.

During the initial flow event into the spillway observed in June 2005, it was noted that the surface flow turned into subsurface flow when it reached the portion of the chute that had been backfilled with rockfill. This may be expected to occur during the first few years of operation, as the voids within the underlying rockfill become ice-saturated and partially filled with soil particles entrained within the flow. This is not expected to have any detrimental effect on the overall performance of the spillway.

4.2 West Twin Outlet Channel

4.2.1 Pre-Reclamation Conditions

Prior to reclamation, water from the Reservoir was released to Twin Lakes Creek through a concrete control structure (see Photos 1 and 2, Appendix II). The existing structure contained a metal plate with a number of valves which permitted regulated release of water from the polishing pond, when required. The existing structure was founded on bedrock and generally maintained the water level in the Reservoir between an elevation of 370.5 and 371.5 m.

4.2.2 Design Criteria

The general reclamation plan was originally developed by Golder Associates and documented in BGC et. al (2004). The reclamation plan involved removal of the existing control structure at the outlet of the WTDA and construction of an open-flow channel, termed the West Twin Outlet Channel. The channel was to incorporate an overflow weir to provide passive elevation control of the water level in the Reservoir. The weir was to incorporate two elements; a foundation trench and a cut-off wall. Specific design criteria for the outlet channel, as outlined in BGC et al. (2004), include the following:

- A steel reinforced, concrete foundation trench was to be constructed to limit seepage through the foundation of the overflow weir.
- A 0.3 m wide, steel reinforced, concrete cut-off wall was to be constructed to provide a fixed, non-erodible invert and prevent seepage through the riprap and bedding zones at the sides of the channel. The invert elevation of the central portion of the wall was to be 370.2 m.
- The bedrock foundation beneath the trench was to be grouted to further reduce seepage under the wall.
- The wall was to extend 5 m laterally into the adjacent embankment fill and into the natural ground abutment to increase the length of the seepage path around the wall.
- Upstream of the cut-off wall, riprap erosion protection was to be provided on the side slopes of the channel up to an elevation of 370.8 m.
- Downstream of the wall, riprap erosion protection was to be applied 0.3 m above the channel floor.
- A 3 m long stilling basin was to be constructed downstream of the wall to prevent scour of the natural creek bed.
- Riprap was to be placed against the wall to smooth the flow lines and protect the concrete surface.

4.2.3 Construction Specifications

Construction specifications were provided by Golder (2005b). The specifications are not reviewed in detail within this current report, but the specification document is included in Appendix III. Within that document, technical specifications were provided for the following elements:

- Demolition of the previous structure;
- Stripping of foundation soils;
- Foundation preparation;
- Placement and compaction of fill materials;
- Fill materials;
- Development and operation of borrow sites;
- Surveying;
- Care of water and siltation control; and
- Concrete materials and placement.

Additionally, a backwater analysis of Twin Lakes Creek was undertaken by Golder (2005a). This study was completed to assess the potential for backwater effects into the Reservoir from Twin Lakes Creek under extreme flood conditions. The results of the study are provided in this current report in Appendix IV. The results of the study indicated that Twin Lakes Creek has sufficient hydraulic capacity to convey flood events without reversing the flow direction from the creek into the Reservoir.

4.2.4 West Twin Outlet Channel Construction

Select photos of the reclamation process including pre-reclamation conditions and weir construction are illustrated on the photos included in Appendix II. The as-built details of the West Twin Outlet Channel are provided on Figure 7. The following sections provide details regarding the construction schedule, construction materials, construction methodologies and the as-built survey information.

<u>Construction Schedule</u>

In 2005, the West Twin Outlet Channel was constructed by demolishing the existing concrete outlet works and constructing the new channel and associated control structure. The reclamation schedule is summarized by the following points:

- Late August 2005 – Demolition of the existing concrete outlet structure.
- Mid to Late September 2005 – Construction of the foundation trench and cut-off wall.
- Early October 2005 – Complete armouring of the outlet channel.

<u>Construction Materials</u>

Materials required for construction of the West Twin Outlet Channel were either sourced locally or transported to site via the sea lift in August 2005. The required construction materials are summarized in Table 2.

Table 2 Summary of Construction Materials – West Twin Outlet Channel

Element	Type	Source
Cement	Type 50 (High Sulphate Resistant)	Transported to Site
Admixture	Air Entrainment	
Rebar	19M (CSA Standard G30-12-M1977)	
Grout	Quikrete (Non-shrink Precious Grout #1585-00)	
Water-stop	CPD PVS Type 4	
Fine Aggregate		Screened sand and gravel from airport
Coarse Aggregate		DMS rejects
Riprap		WTDA Spillway Excavation
Water		East Twin Lake

Concrete Mix Design

Although no concrete mix design was provided in the construction specifications (Golder 2005b), the following design criteria were provided:

- Maximum aggregate size = 19 mm (0.75 inch).
- Slump at point of delivery = 75 to 100 mm (3 to 6 inches).
- Air content 5 to 8 per cent.
- Maximum water to cement ratio not to exceed 0.5:1 by weight.
- Cement shall be CSA Type 50 Portland Cement, CSA Standard A5-M83.
- Minimum cement content 295 kg/m^3 of concrete.
- Minimum compressive strength = 35 MPa at 28 days.

A design had to be developed at site, utilizing available construction materials, that would meet these design specifications. As such, a number of small scale test batches were developed and tested for both air content and slump. Due to time constraints, no compressive strength tests were completed on the test batches.

Table 3 summarizes the test batch mix designs and the results of QA/QC tests completed on samples obtained from each test batch.

Table 3 Summary of QA/QC Testing on Concrete Test Batches

Trial#	Materials				Material Ratio	QA/QC Results	
	Cement (kg)	Aggregate (kg)	Water (kg)	Admixture (ml)	W:C	Air (%)	Slump (mm)
1	5	30	2	10	0.4	3	30
2	5	30	2	25	0.4	4	10
3	5	30	2.25	35	0.45	5	200
Design Specifications					**0.5**	**5-8**	**75-100**

As can be seen, the consistency of the concrete, as measured by the slump, was sensitive to water content. The measured slump increased an order of magnitude as the water to cement ratio increased from 0.4:1 to 0.45:1. As such, the water to cement ratio was not to exceed 0.45:1 in the final mix design.

The admixture to air entrainment relationship, as suggested by the results of the test batches, was not considered to reflect expected field performance. This was due to the inability to properly mix the admixture in the test batch samples using manual methods. Thus, the admixture to water ratio included in the final mix design was taken from a standard relationship and modified based on actual field measurements during construction.

Based on the results of the test batching the following mix design criteria were developed:

- Water to cement ratio = 0.45:1.
- Aggregate to concrete ratio = 6:1.
- Fine aggregate to coarse aggregate ratio = 3:1.
- Admixture to water = 0.8 ml admixture to 1 kg water.

Measured grain size distribution curves for each aggregate type are provided on Figure 8. As can be seen, the combined grain size distribution meets typical concrete aggregate grain size specifications, as documented in CPCA (1995). It should be noted that the DMS reject material was used as coarse aggregate due to its appropriate grain size distribution and availability. No mineralogical information was available on this material. However, it was noted by mine staff that the DMS reject material was typically composed of dolostone.

Construction Details

The West Twin Outlet Channel was constructed in September and October, 2005. The construction of the West Twin Outlet Channel was a combined effort of both ATCON and the Nanisivik Mine staff. The following is a summary of all the construction phases of the outlet structure:

1. Demolition of existing structure;
2. Footing excavation and slush grouting;
3. Installation of foundation dowels;
4. Construction of rebar frame for the footing;
5. Concrete pour for the footing;
6. Construction of rebar frame for the weir;
7. Concrete pour for the weir;
8. Construction of weir abutments; and
9. Channel armouring.

It should be noted that the main contractor, ATCON, undertook all earthworks operations, including foundation excavation and armour placement. Mine staff was in charge of all concrete operations including re-bar installation, form construction and concrete batching, pouring and finishing.

The following sections provide additional detail regarding each of the construction phases mentioned above.

1. Demolition of Existing Structure

The breach of the existing outlet structure took place on August 27, 2005. The existing concrete structure was demolished using the 385ME excavator. The demolition debris, including concrete and re-bar, was transported to the underground mine workings using the mine fleet haul trucks.

2. Footing Excavation and Slush Grouting

Once the existing structure was removed, the foundation trench was excavated. The excavation proceeded until refusal on competent rock was encountered. The foundation excavation was further excavated another 0.6 to 0.8 m into the competent rock utilizing a hydraulic hammer system installed on the John Deere 892 excavator. The foundation excavation was approximately 1.5 m wide, 1 m deep and 17 m long.

Once the foundation excavation was complete, the flow of seepage water from the near surface materials filled the excavation (see Photo 3, Appendix II). As such, the foundation trench was dewatered using a combination of upstream and downstream coffer dams and submersible pumps placed in the excavation (see Photo 4, Appendix II). The seepage rate was significantly reduced as the ambient air temperatures continued to decrease below the freezing mark by mid-September.

After the seepage flow was controlled, the top of rock surface was swept clean, by hand. Then the top of rock surface was slush grouted, using a water and cement (Type 50) mix, to fill the cracks and to generally smooth the rock surface.

3. Installation of Foundation Dowels

The dowel placement took place on September 13, 2005. In total, seventeen dowels were installed into the foundation of the weir. The dowel holes were drilled from the excavated surface to a depth of 1.2 m (bgs) using a handheld rock drill. Prior to installing the dowels, the holes were filled with grout. The dowels were then placed into the holes. Once placed, the dowels extended approximately 1.2 m above the bedrock surface. The dowels were installed with 1 m spacing, starting at 0.5 m from the projected end of the weir. The dowels were off-set by 0.05 m of centre line, in an alternating pattern (see Photo 5, Appendix II).

4. Construction of Rebar Frame for Foundation Trench

The rebar frame for the foundation trench was constructed between September 13 and 17, 2005. Photographic documentation of the rebar construction is provided on Photos 6 through 8 in Appendix II. The longitudinal bars were placed in two levels, with six bars per row. The first and second levels were approximately 0.2 and 0.5 m above the bottom of the excavation, respectively. The longitudinal bars in the lower level had a typical spacing of 0.2 m; except for the two bars in the centre which had 0.1 m spacing. The longitudinal bars in the second level had a typical spacing of 0.3 m; except for the two in the centre which had 0.1 m spacing. The perpendicular bars to the longitudinal bars had a typical spacing of 0.25 m and ran the entire length of the footing. The bent angle bars were placed at a typical spacing of 0.30 m in an alternating fashion for the entire length of the trench and typically extended to the top of the cut-off wall.

5. Concrete Pour for Foundation

The construction of the concrete foundation trench of the overflow weir was completed on September 20, 2005. The air temperature was approximately -6°C and no precipitation was recorded on that day. Photographic documentation of the rebar construction is provided on Photos 9 through 14 in Appendix II.

Concrete was batched on-site using a 6 m^3 concrete mixer. During batching, the following procedure was followed:

- Fine and coarse aggregate was placed into the mixer at approximately the prescribed ratio.
- The cement was placed into the mixer.
- The aggregate and cement were allowed to mix for approximately 5 minutes.
- Water was added to the mixer.
- Air entrainment admixture was added to the mixer.
- The concrete batch was allowed to mix for approximately 30 minutes and then the entire batch would be emptied into concrete placement bucket.
- Samples were collected from the bucket prior to placement and the QA/QC testing was undertaken.
- The concrete was then placed into the foundation trench or cut-off wall forms.
- The placed concrete was manually agitated by rodding and manually worked to ensure proper placement around the rebar.

In total, approximately 39 m^3 of concrete was placed in the foundation trench. Seven batches of concrete were made; six of the batches were approximately 6 m^3 in volume, and the final batch was approximately 3 m^3. At the completion of the pour, the water stop was placed along the centreline of the footing (as shown on Photo 15 in Appendix II). The water stop was embedded approximately 0.1 m into the foundation trench concrete, leaving approximately 0.1 cm exposed to be later incorporated within the cut-off wall. After the pour, the concrete foundation trench was covered with tarps and portable heaters were installed to allow the concrete to cure without freezing (see Photo 16, Appendix II). The tarps and heaters were removed seven days following the pour. It should be noted that the ambient air temperature during the week following the pour ranged between approximately -6° and -10°C.

Samples from each batch were collected for QA/QC testing. The air entrainment and slump tests were completed at site during each pour. Samples for compressive strength testing were collected at the time of the pour, stored in an insulated box under the heated tarp and sent off-site for testing approximately 1 week after being sampled. The results of the QA/QC testing are provided in Table 4. The results are summarized by the following points:

- The measured air content was observed to range from 4 to 9%, with an average of 6%.
- The measured slump was observed to range from 140 to 200 mm, with an average of 178 mm.

- The 28 day strength for the concrete footing ranged from the 27 to 33 MPa with an average of 31 MPa.

6. Construction of Rebar Frame and Concrete Forms for Cut-off Wall

The rebar frame for the cut-off wall was constructed between September 23 and 25, 2005. The construction process is illustrated on Photos 17 through 20, included in Appendix II. The rebar frame for the cut-off wall was built using a double rebar frame and u-joint at the top to hold the rebar in place and provide stability.

Following construction of the rebar frame, wooden forms were constructed for placement of the concrete for the cut-off wall.

7. Concrete Weir Construction

The construction of the concrete cut-off wall portion of the overflow weir was completed on September 29, 2005. The construction process is illustrated on Photos 21 through 25, included in Appendix II. The air temperature was approximately -10^{o}C with a wind chill of approximately -17^{o}C. Some light snow fell during the placement of the second batch of concrete.

In total, approximately 12 m^3 of concrete was used during construction of the cut-off wall. There were two batches of concrete made; both batches were approximately 6 m^3 in volume. It should be noted that, due to a low supply of Type 50 cement, approximately 360 kg of Type 10 cement and 360 kg of Type 30 cement were used in the final batch. Thus, approximately 40% of the concrete used in the final batch was non-Type 50 cement. It should be noted that Type 10 and Type 30 cement were used since they were the only materials that could be sourced in a short period of time. After the pour, the concrete wall was covered with tarps and heaters were installed to allow the concrete to cure without freezing. The forms were removed three days following the pour. The tarps and heaters were removed seven days following the pour. It should be noted that the ambient air temperature during the week following the pour was ranged between approximately -10° and -15°C.

Samples from each batch were collected for QA/QC testing. The air entrainment and slump tests were completed at site during each pour. Samples for compressive strength testing were collected at the time of the pour, stored in an insulated box under the heated tarp and sent off-site for testing approximately 1 week after being sampled. The results of the QA/QC testing are provided in Table 4. The results are summarized by the following points:

- The measured air content was observed to range from 4 to 5%, with an average of 4.5%.
- The measured slump was observed to range from 80 to 130 mm, with an average of 105 mm.
- The 28 day strength for the concrete footing ranged from the 25 to 26 MPa with an average of 25.5 MPa.

Table 4 Summary of Concrete QA/QC Testing for West Twin Outlet Structure

		Concrete Mix Details				Results		
Mix	**Date**	**Cement (kg)**	**Aggregate (kg)**	**Water (kg)**	**Admixture (ml)**	**Air (%)**	**Slump (mm)**	**Compressive Strength (MPa)**
Foundation 1	20-Sep-05	1800	10500	810	700	4	200	N/A
Foundation 2	20-Sep-05	1800	10500	810	800	6	200	33
Foundation 3	20-Sep-05	1800	10500	810	800	9	180	N/A
Foundation 4	20-Sep-05	1800	10500	810	750	6	200	32
Foundation 5	20-Sep-05	1800	10500	810	750	6	140	N/A
Foundation 6	20-Sep-05	1800	10500	810	750	7	150	27
Foundation 7	20-Sep-05	900	5250	405	400	N/A	N/A	N/A
Wall 1	28-Sep-05	1800	10500	810	750	5	80	26
Wall 2*	28-Sep-05	1800	10500	810	750	4	130	25
					Average	**6**	**160**	**29**
					Design Specifications	**5-8**	**75 to 100**	**>35**

Note: * Cement for Wall Batch #2 was a combination of Type 50, Type 30 and Type 10 cement.

8. Abutment Construction

To minimize seepage rates at the abutment locations, a bentonite seal was constructed at the north and south abutments of the weir. The powdered bentonite was placed between the side of the excavation and the concrete wall (Photo 26, Appendix II). The bentonite was placed to approximately 370.2 m elevation, the design normal water level. A 0.3 m thick layer of screened sand and gravel was placed on top of the bentonite and compacted using the excavator bucket (Photo 27, Appendix II). Riprap was placed on top of the sand and gravel to an elevation of approximately 370.8 m.

9. Channel Armouring

Armouring operations are illustrated on Photos 28 through 35 in Appendix II. Prior to armouring the channel, the remaining excavation between the top of the concrete foundation trench and the top of the adjacent ground surface was backfilled with screened sand and gravel from the Twin Lakes Delta deposit. Additionally, the area immediately downstream of the weir was cleaned down to frost-affected bedrock and the stilling pool was excavated into the frost-affected bedrock. Riprap was then applied to the bottom and sides of the channel. The riprap was sourced from the dolostone outcrop at the south end of the Surface Cell. The average size of the riprap was approximately 300 mm. Upstream and downstream of the weir, riprap was applied to sides of the channel to an elevation of approximately 370.8 m. In the bottom of the channel, the riprap was placed directly on frost-affected bedrock. Along the sides of the channel the riprap was placed directly on native ground which varied from frost-affected bedrock to overburden.

In the years following construction, it was observed that during periods of low inflows into the polishing pond the water levels upstream of the wall would drop below the invert elevation. This was inferred to suggest that some minor seepage losses through the foundation of the wall may be occurring. In response to these observations, a Geosynthetic Clay Liner (GCL) was installed upstream of the wall in 2007 to further reduce seepage losses through the foundation of the wall. The GCL was extended approximately 10 m upstream of the wall and was double layered to limit damage from overlying rockfill. The GCL was also embedded in the abutments of the wall to limit the potential for seepage losses at these locations.

It should be noted that at the time of the observations, water flow from the Reservoir to the polishing pond remained constrained by the culverts in the access road. As such, it is likely that when the road is breached and the Reservoir and polishing pond become an unconstrained flow through system, minor seepage losses at the outlet channel, should they occur, will not have a significant effect on the water level in the Reservoir. It should also be noted that, based on the hydrological assessment documented in BGC et al. (2004), the Reservoir water level is expected to range between elevation 370.0 and 370.8 m in response to the expected range of hydrologic conditions. Thus, the water level in the Reservoir is not always expected to be maintained at or above the invert elevation of the concrete weir.

<u>As-Built Drawings</u>

The as-built drawings of the West Twin Outlet Channel are provided on Figures 6 and 7. The main aspects of the as-built configuration are summarized by the following:

- The central portion of the weir is 7 m wide and 0.3 m thick;
- The invert elevation of the central portion of the weir is 370.2 m;
- The wall contains sloping side walls (4H:1V) and the elevation of the top of the side walls is approximately 370.8 m;
- The channel is armoured to approximately 370.8 m elevation;
- A 0.5 m deep plunge pool is located approximately 10 m downstream of the weir.

4.2.5 Verification of Design Intent

The West Twin Outlet Channel was generally constructed to the specifications provided in the design drawings with the exception of the following deviations:

- The final concrete batch used to construct the cut-off wall included a minor amount of non-Type 50 cement. This included approximately 360 kg of Type 10 and 360 kg of Type 30 cement. The effect of including this alternative cement type is unknown, but only half of the batch was required.
- The initial design specifications for the overflow weir required epoxy coated rebar. However, only non-epoxy coated rebar was supplied.
- The foundation beneath the cut-off trench was not grouted, as proposed in the original design. This was due to the frozen nature of the bedrock in the foundation. However, the installation of the GCL upstream of the concrete weir should provide additional seepage control for the foundation of the overflow weir.

It should be noted that the measured concrete strength parameters did not meet the design specifications provided by Golder (2005b). Since the cut-off wall is not considered a structural element, the reduced strength is likely not a significant concern. However, the long term durability of the structure may be affected.

4.3 East Twin Diversion Channel Upgrade

4.3.1 Pre-Reclamation Conditions

The East Twin Diversion Channel is an existing earth structure which was constructed during mining operations to divert Twin Lakes Creek from its natural drainage course which originally drained directly into West Twin Lake (the Reservoir). The water is diverted into the channel by the East Twin Diversion Dike. The dike is approximately 2 m high and is constructed primarily of granular materials derived from the Twin Lakes Delta deposit. Prior to 2005, localised erosion of the face of the dike was observed. As such, a reclamation plan was developed to stabilize the areas of the dike where erosion had been observed and to upgrade the dike in these areas to limit the potential for erosion in the future.

4.3.2 Reclamation Plan

In BGC et al. (2004), it was recommended that the face of the dike that had experienced erosion be regraded and armoured. The slope was to be regraded to 3(H):1(V) and a 0.45 m thick layer of riprap (D_{50} = 300 mm) was to be placed along the face of the slope.

4.3.3 Construction Details

In October 2005, the face of the dike where erosion was observed was re-graded to an approximate slope of 3(H):1(V) using the 385ME excavator. The existing materials were observed to consist of sand, gravel and cobbles derived from the Twin Lakes Delta deposit. Riprap derived from the West Twin Dike spillway excavation, was transported to the East Twin Diversion dike and placed along the face of the slope with the 385ME excavator.

4.3.4 Verification of Design Intent

The reclamation of the East Twin Diversion Channel was completed to the original design intent. The only design modification was that no bedding material was applied to the re-graded face of the dike prior to placement of the Type 1 riprap. No bedding material was considered necessary due to the coarseness of the original dike materials, which were comprised of sand gravel and cobbles.

5.0 SUMMARY

In general, the hydraulic structures have been constructed to the original design intent and have achieved the design objectives by demonstrating the following characteristics:

- The West Twin Dike Spillway effectively transfers water from the Surface Cell to the Reservoir and limits ponding of surface water on the Surface Cell to a small area at the spillway inlet.
- The West Twin Outlet Channel provides passive elevation control for the water level in the Reservoir while maintaining a stable, controlled outlet for the WTDA watershed.
- The Twin Lakes Creek Diversion Dike has been armoured to maintain its stability in the long term stability while deflecting water from the East Twin water shed into the diversion channel and out of the Reservoir.

Some minor modifications have been made to the original design of each element. The rational associated with each modification has been provided.

6.0 POST CONSTRUCTION MONITORING AND MAINTENANCE

Monitoring of the performance of the hydraulic structures should be included as part of the Annual Geotechnical Inspection of the reclamation measures undertaken annually at Nanisivik Mine. The inspections and monitoring should be conducted in July or August of each year, until 2010, to observe the closure measures that were undertaken at the Nanisivik Mine are meeting their designed intent.

Visual observations of the spillway will be undertaken annually to identify and correct blockage due to sloughing of rock or overburden slopes and the integrity of the erosion protection. Any material observed to be sloughing into the bottom of the spillway should be removed to ensure adequate capacity.

As per the recommendations in BGC et al. (2004), the West Twin Outlet Structure will be inspected for the presence of upstream or downstream blockages, as well as the condition of the concrete wall and riprap. Additionally, any indications of seepage losses are to be noted.

As per the recommendations in BGC et al. (2004), the East Twin Diversion Channel will be inspected for additional signs of erosion, both in the repair area and adjacent areas.

7.0 CLOSURE

This report summarizes the construction of the hydraulic structures undertaken during the reclamation of the Nanisivik Mine site.

We trust that this report meets your needs at this current time. Should you have any questions or comments concerning the information provided within this report, please contact the undersigned.

Respectively submitted:
BGC Engineering Inc.
Per:

Geoff Claypool, P.Eng.
Geological Engineer

James W. Cassie, M.Sc., P.Eng.
Specialist Geotechnical Engineer

REFERENCES

BGC Engineering Inc., Gartner Lee Ltd. and Golder Associates Ltd. 2004. West Twin Disposal Area Closure Plan. Submitted to CanZinco Ltd., March 4, 2004.

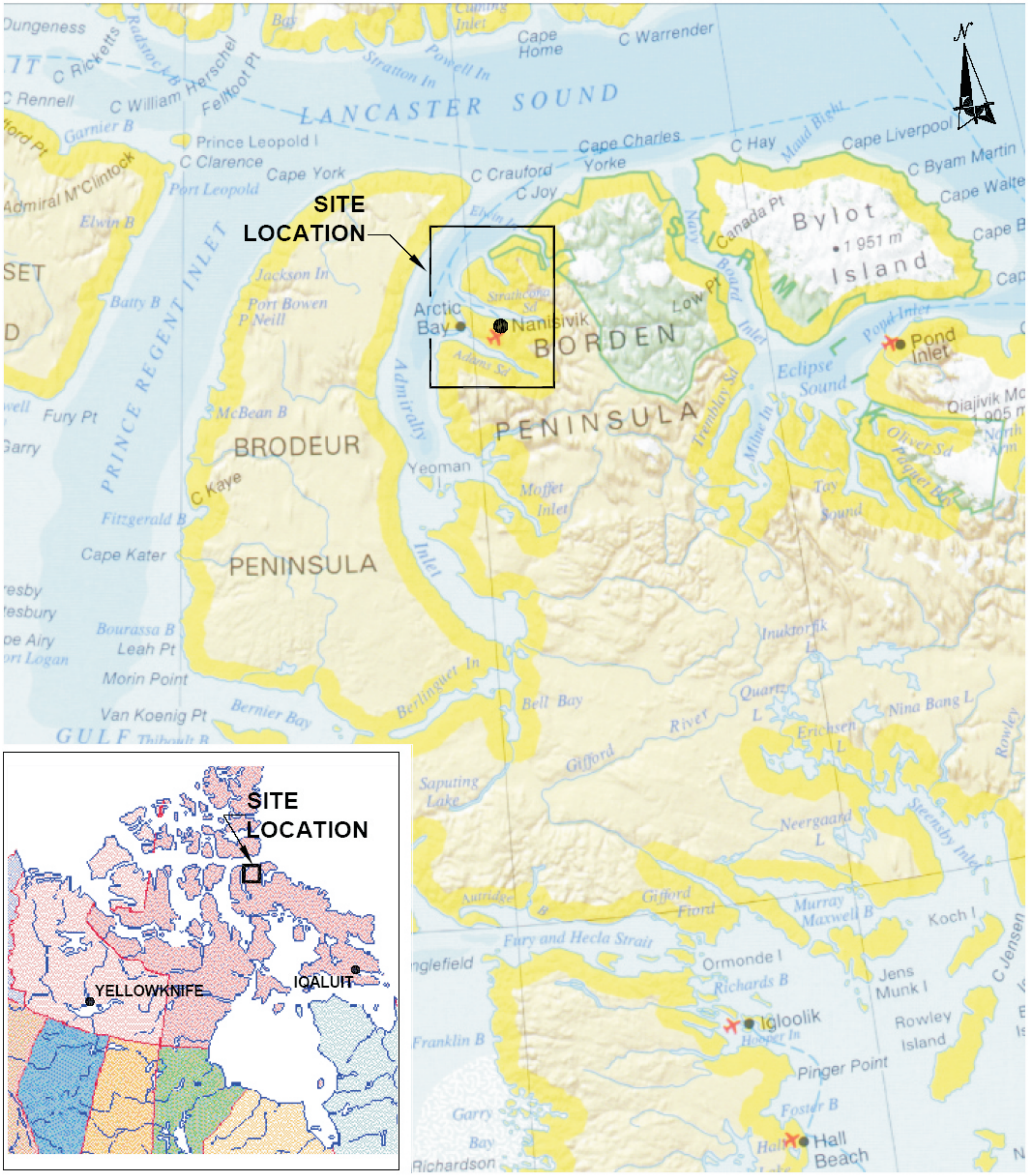
Canadian Portland Cement Association 1995. Design and Control of Concrete Mixtures Sixth Canadian Edition.

Golder Associates Ltd. 2004. Detailed Design of the West Twin Dyke Spillway. Submitted to Breakwater Resources Ltd., March 2004.

Golder Associates Ltd. 2005a. Backwater Assessment of the West Twin Disposal Area, Nanisivik Mine. Submitted to BGC Engineering Inc. July 22, 2005.

Golder Associates Ltd. 2005b. West Twin Outlet Channel Technical Specifications. Project No 051-118011. Submitted to Breakwater Resources September 2005.

FIGURES



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DATE:	MAY 2008	CHECKED:	GKC
DRAWN:	SLF	APPROVED:	JWC

CLIENT: BREAKWATER RESOURCES LTD

BGC Engineering Inc.
AN APPLIED EARTH SCIENCES COMPANY
Calgary, Alberta. Phone: (403) 250-5185

PROJECT
NANISIVIK MINE RECLAMATION
HYDRAULIC STRUCTURES AS-BUILT REPORT

TITLE
SITE LOCATION MAP

PROJECT No.	FIGURE No.	REV.
0255-012-05	1	0

Approixmate Scale
1:12,500

Note:

1. Photo derived from Google Earth January 10, 2008

2. Approximate date of photo is July 2005.

DATE: MAY 2008

DRAWN: SLF

REFERENCED DRAWING DESCRIPTION

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AN APPLIED EARTH SCIENCES COMPANY

Calgary, Alberta

Phone: (403) 250-5185

CLIENT: BREAKWATER RESOURCES LTD

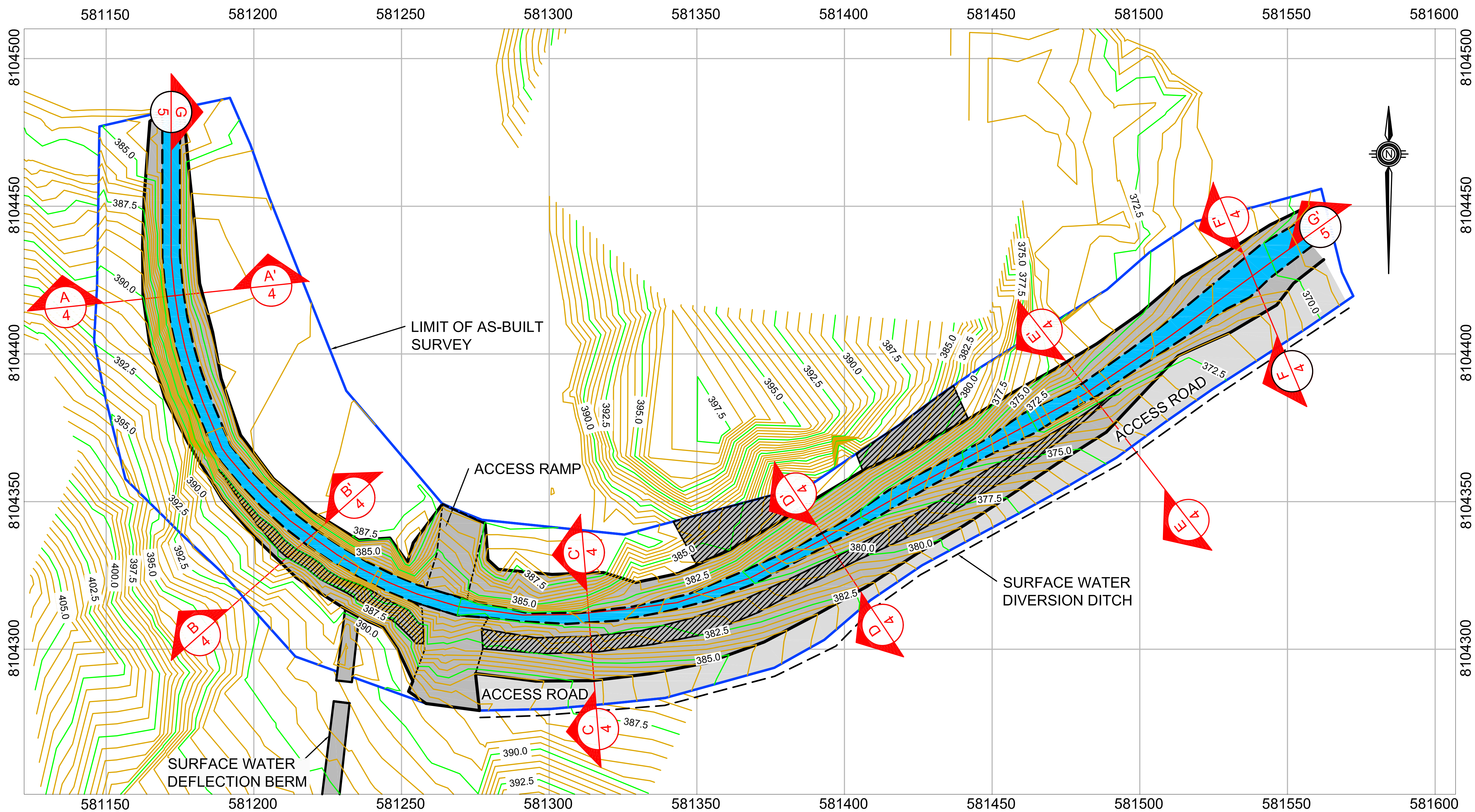
PROJECT: NANISIVIK MINE RECLAMATION HYDRAULIC STRUCTURES AS-BUILT REPORT

TITLE: WEST TWIN DISPOSAL AREA SITE PLAN

PROJECT No.: 0255-012-05

FIGURE No.: 2

REV.: 0



Scale 1:1500

30 0 30 60 90

metres

NOTES:

1. GRID IS BASED ON UTM, NAD 83, ZONE 16 COORDINATE SYSTEM.
2. ELEVATIONS ARE IN METRES.
3. CONTOUR INTERVAL IS = 0.5 m.
4. AS-BUILT SURVEY DATA PROVIDED BY ATCON CONSTRUCTION INC. DRAWING FILE ABSE21D1.
5. SEE FIGURES 4 AND 5 FOR SECTION VIEW DRAWINGS.

LEGEND

- CREST OF EXCAVATION.
- BASE OF SPILLWAY.
- BENCH.

REV.	DATE	REVISION NOTES	DRAWN	CHECK	APPR.

SCALE:	AS SHOWN
DATE:	MAY 2008
DRAWN:	REM
DESIGNED:	GKC
CHECKED:	GKC
APPROVED:	GKC

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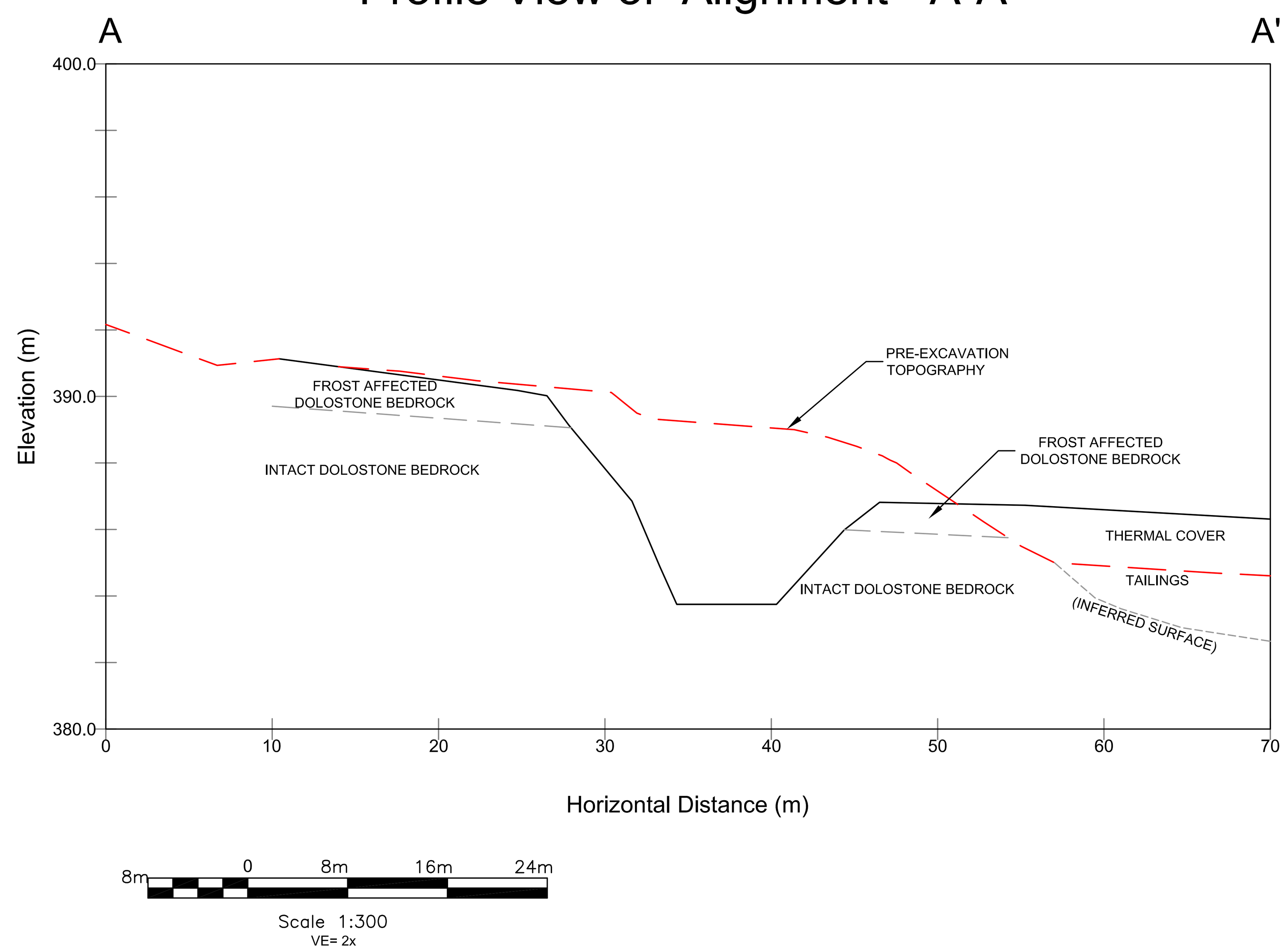
PROJECT: NANISIVIK MINE RECLAMATION HYDRAULIC STRUCTURES AS-BUILT REPORT

TITLE: WEST TWIN DIKE SPILLWAY PLAN VIEW

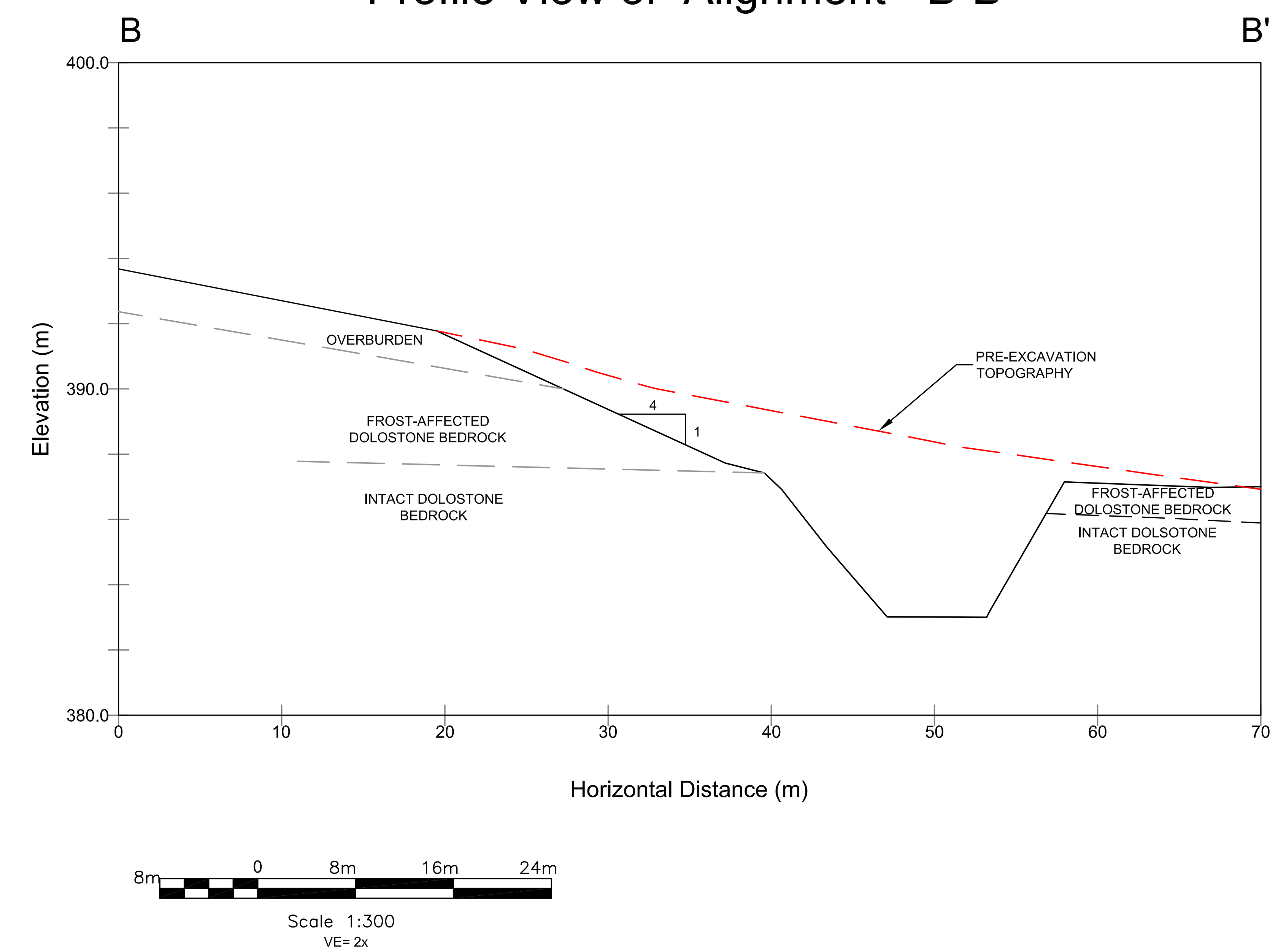
PROJECT No.:	FIGURE No.	REV.:
0255-012-05	3	0

K:\Staff Folders, Edmonton\HE, Willa\Calgary Office\0255-012-05 Spillway-May 2008.dwg Layout: Figure 3-11x17 Plot Date Jun 5 08 Time: 4:54 PM

Profile View of Alignment - A-A'



Profile View of Alignment - B-B'



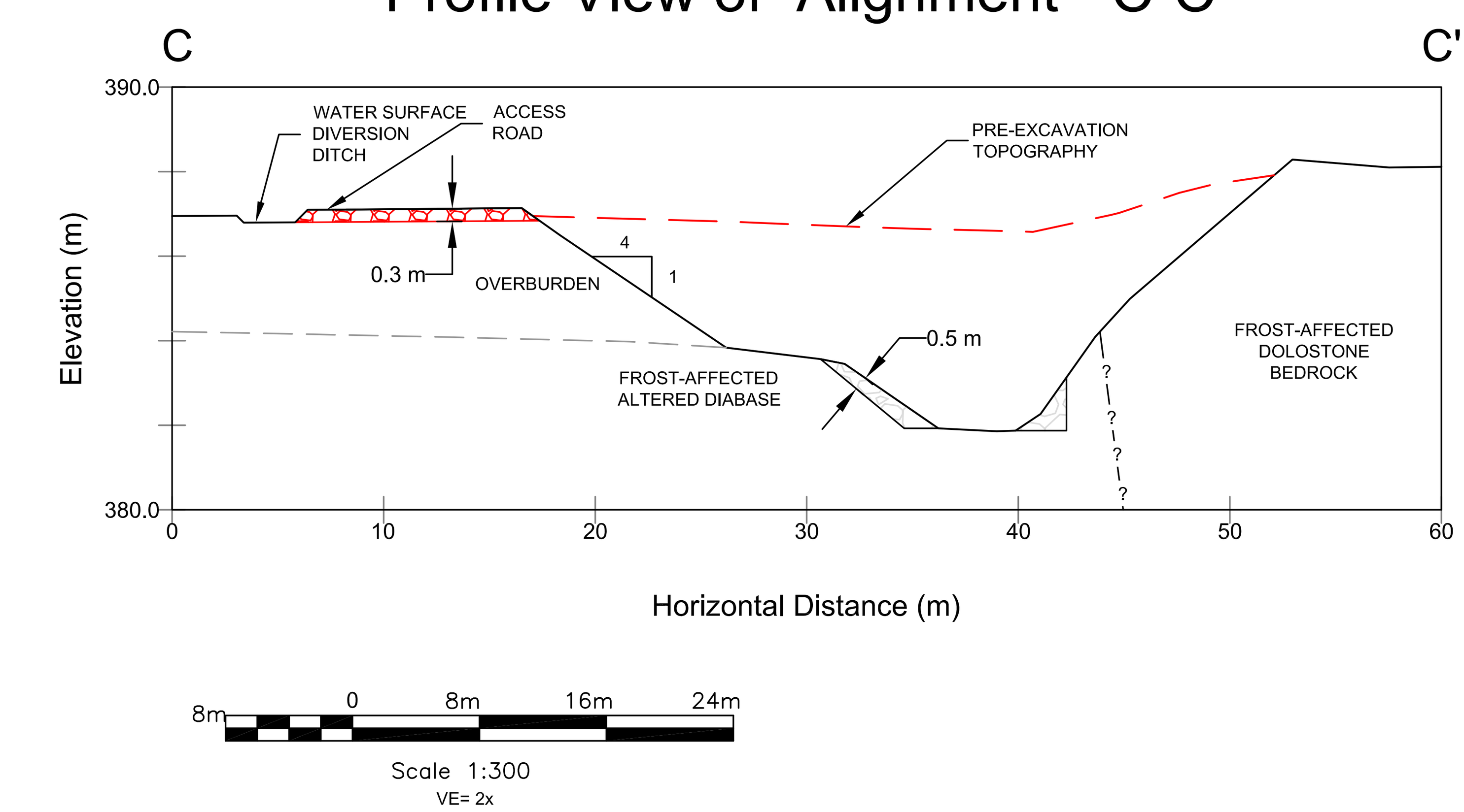
LEGEND:

- PRE-EXCAVATION TOPOGRAPHY ALONG CENTERLINE OF SPILLWAY EXCAVATION.
- TYPE 1 RIP RAP D_{50} = 300mm
- TYPE 2 RIP RAP D_{50} = 100mm

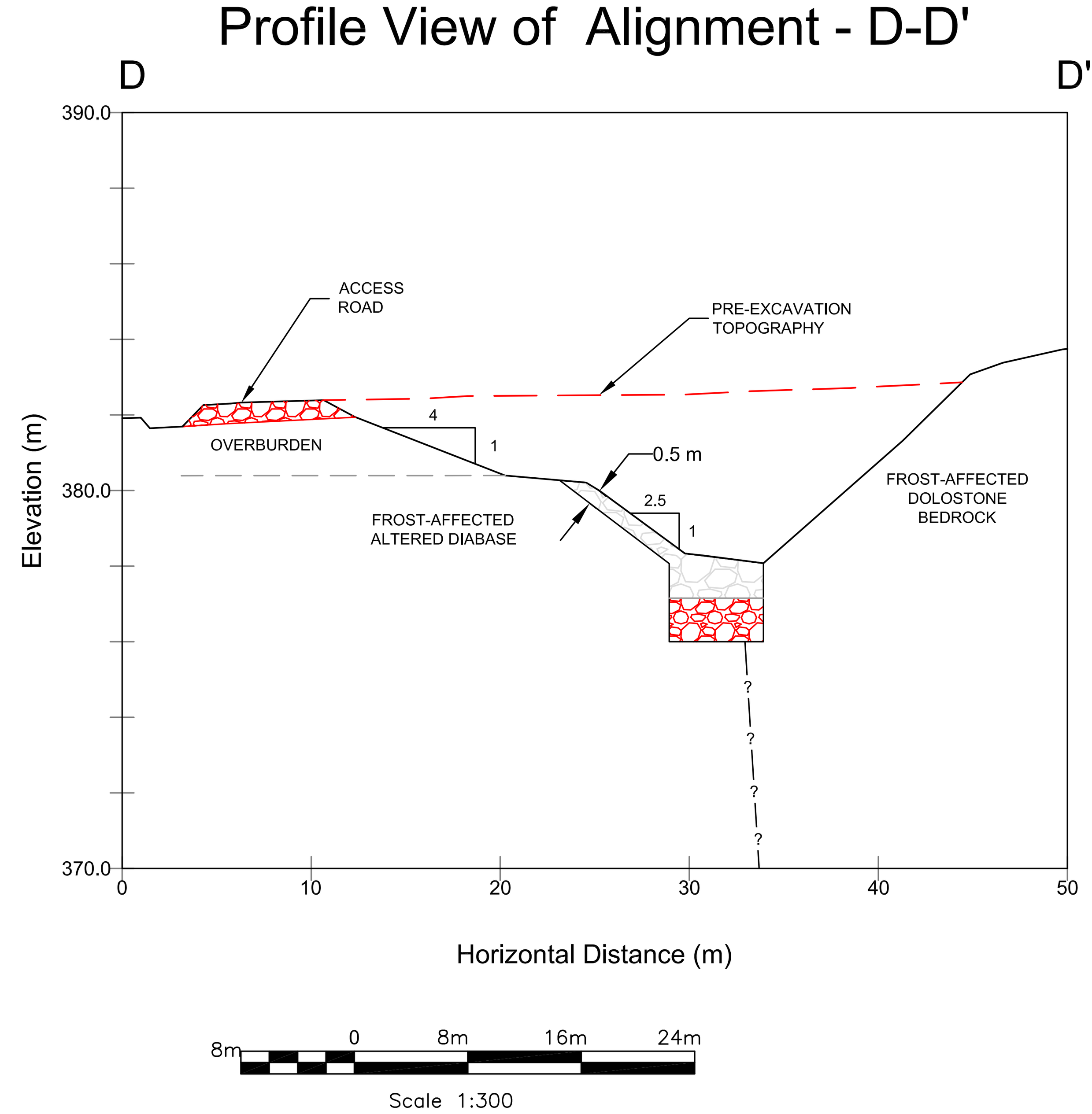
NOTES:

1. ELEVATIONS ARE IN METRES.
2. SCALE IS APPROXIMATE.
3. AS-BUILT SURVEY DATA PROVIDED BY ATCON CONSTRUCTION INC. DRAWING FILE ABSE21D1.
4. PRE-EXCAVATION TOPOGRAPHY AND TOPOGRAPHY OUTSIDE AS-BUILT SURVEY AREA DERIVED FROM ORIGINAL GROUND SURVEY CONDUCTED BY NORTHTECH CONSULTANTS AUGUST, 2004.
5. LITHOLOGY PROVIDED ON SECTION VIEWS BASED ON VISUAL OBSERVATIONS MADE DURING CONSTRUCTION.
6. SEE FIGURE 3 FOR LOCATION OF SECTIONS.

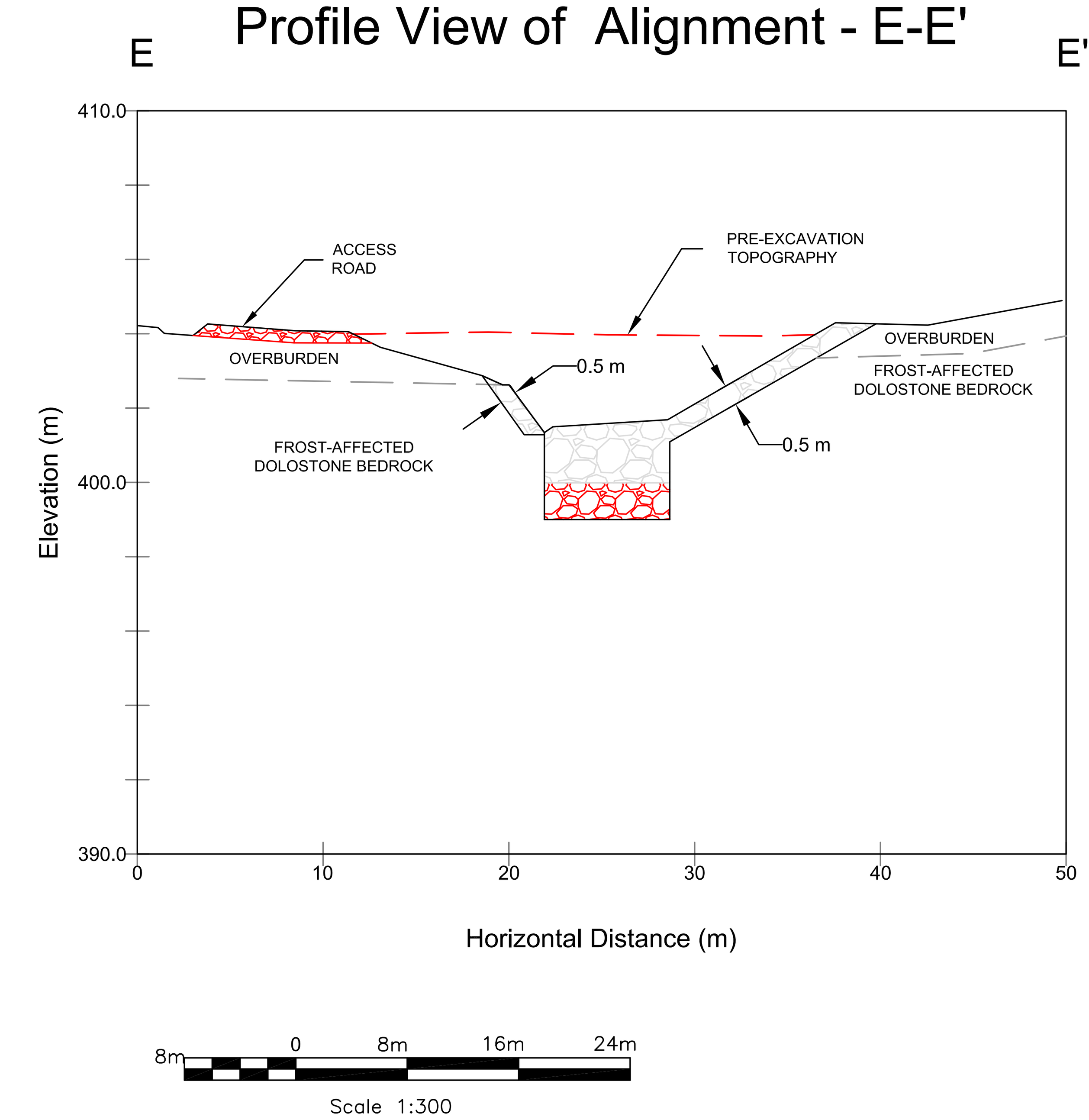
Profile View of Alignment - C-C'



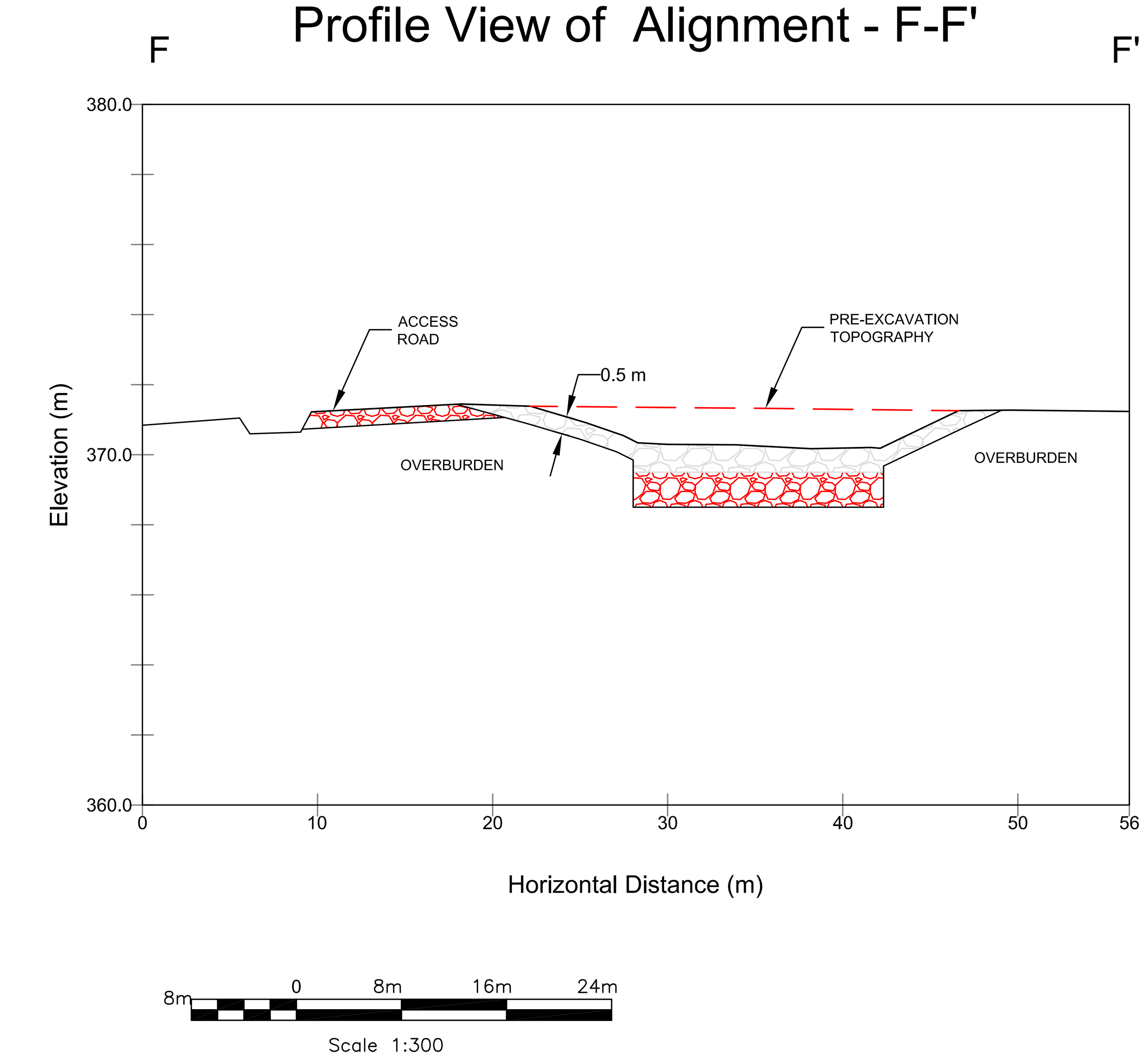
Profile View of Alignment - D-D'



Profile View of Alignment - E-E'



Profile View of Alignment - F-F'



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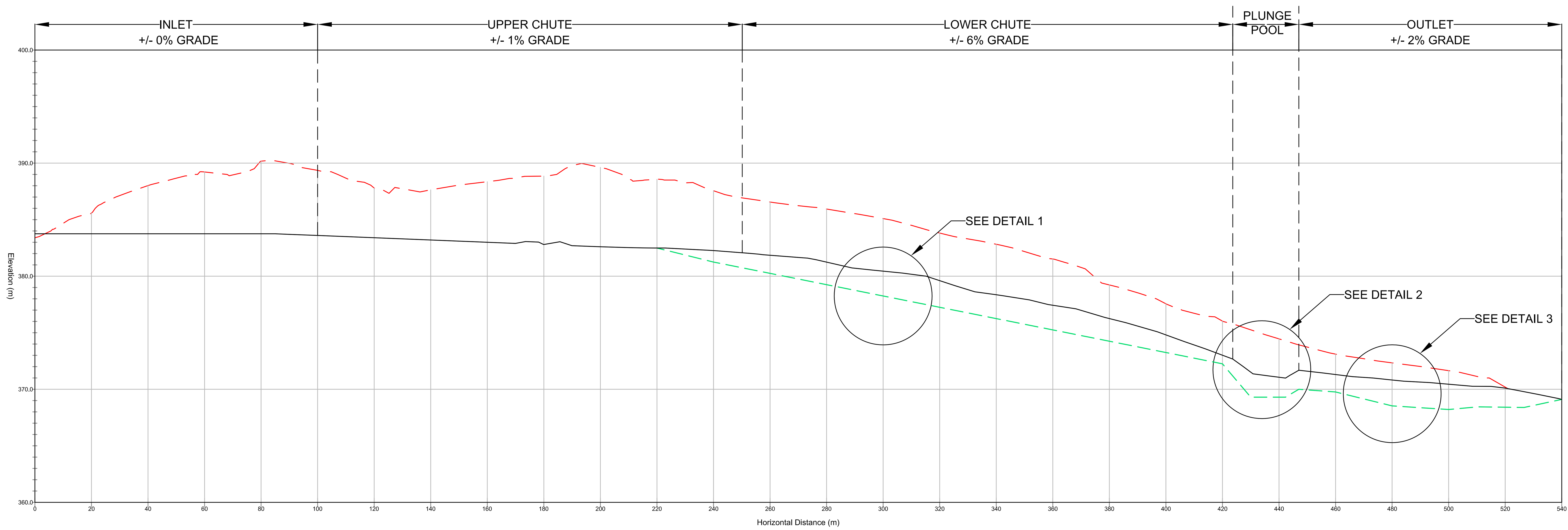
PROJECT: NANISIVIK MINE RECLAMATION HYDRAULIC STRUCTURES AS-BUILT REPORT

TITLE: WEST TWIN DIKE SPILLWAY SECTION VIEWS 1

PROJECT No.:	FIGURE No.	REV.:
0255-012-05	4	0

Profile View of Alignment - G-G'



20m 0 20m 40m 60m

Scale 1:1000
VE= 4x

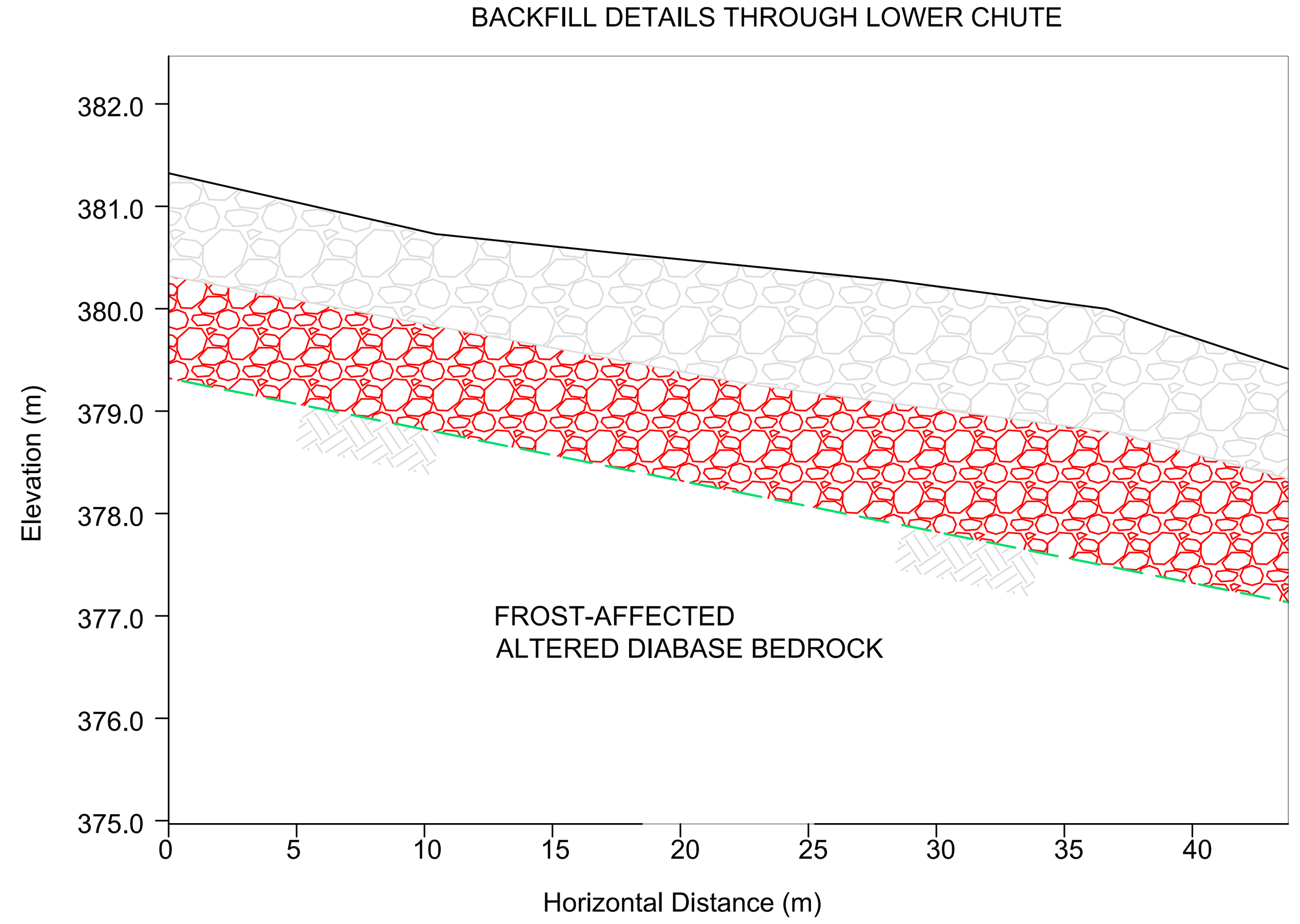
NOTES:

1. ELEVATIONS ARE IN METRES.
2. SCALE IS APPROXIMATE.
3. AS-BUILT SURVEY DATA PROVIDED BY ATCON CONSTRUCTION INC. DRAWING FILE ABSE21D1.
4. PRE-EXCAVATION TOPOGRAPHY AND TOPOGRAPHY OUTSIDE AS-BUILT SURVEY AREA DERIVED FROM ORIGINAL GROUND SURVEY CONDUCTED BY NORTHTECH CONSULTANTS AUGUST, 2004.
5. LITHOLOGY PROVIDED ON SECTION VIEWS BASED ON VISUAL OBSERVATIONS MADE DURING CONSTRUCTION.
6. SEE FIGURE 3 FOR LOCATION OF SECTION.

LEGEND:

- PRE-EXCAVATION TOPOGRAPHY ALONG CENTERLINE OF SPILLWAY EXCAVATION.
- BASE OF SUB-EXCAVATION.
- BASE OF SPILLWAY.
- TYPE 1 RIP RAP D_{50} = 300mm
- TYPE 2 RIP RAP D_{50} = 100mm

DETAIL 1
BACKFILL DETAILS THROUGH LOWER CHUTE



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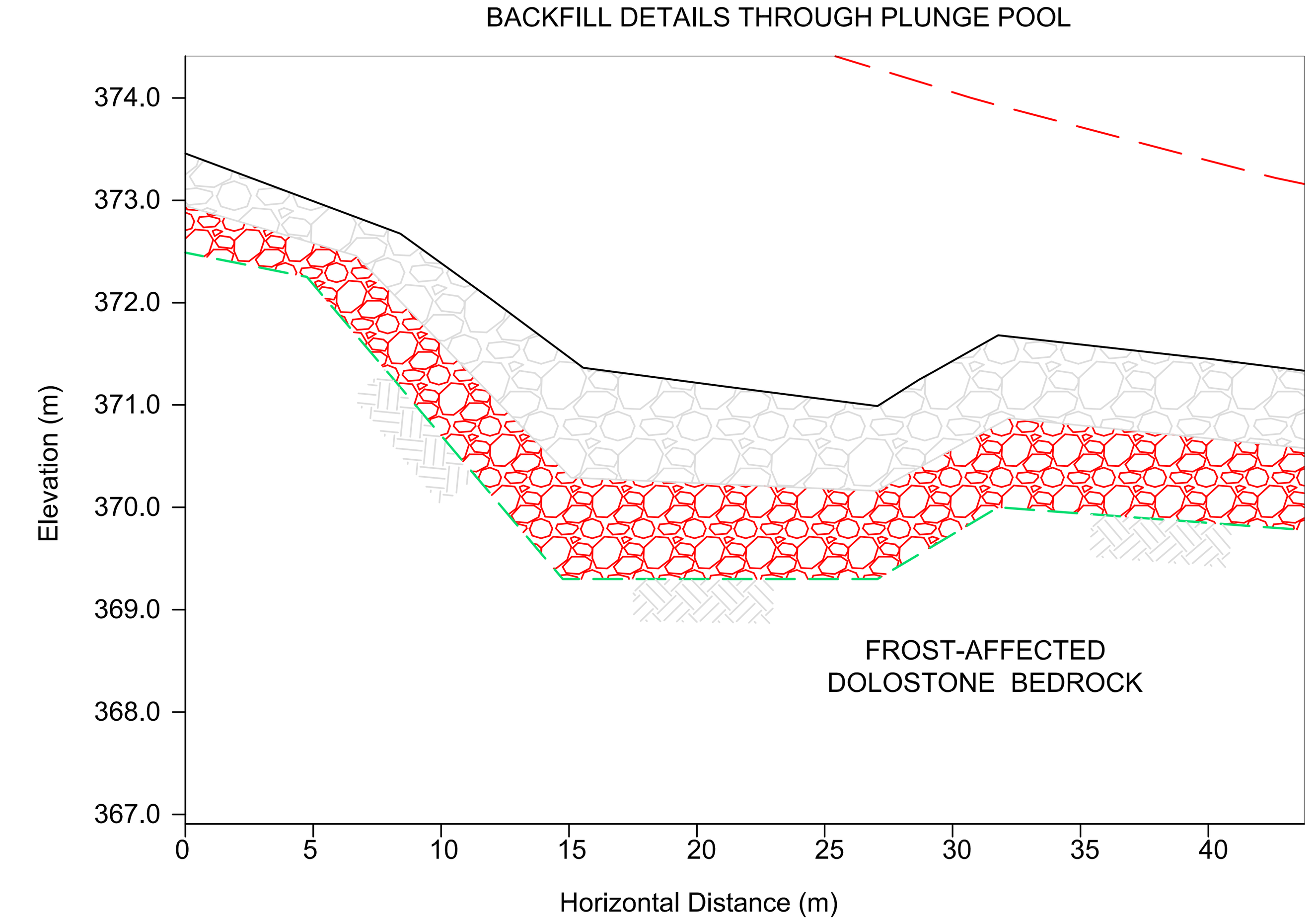
Horizontal Scale 1:250
VE= 4x

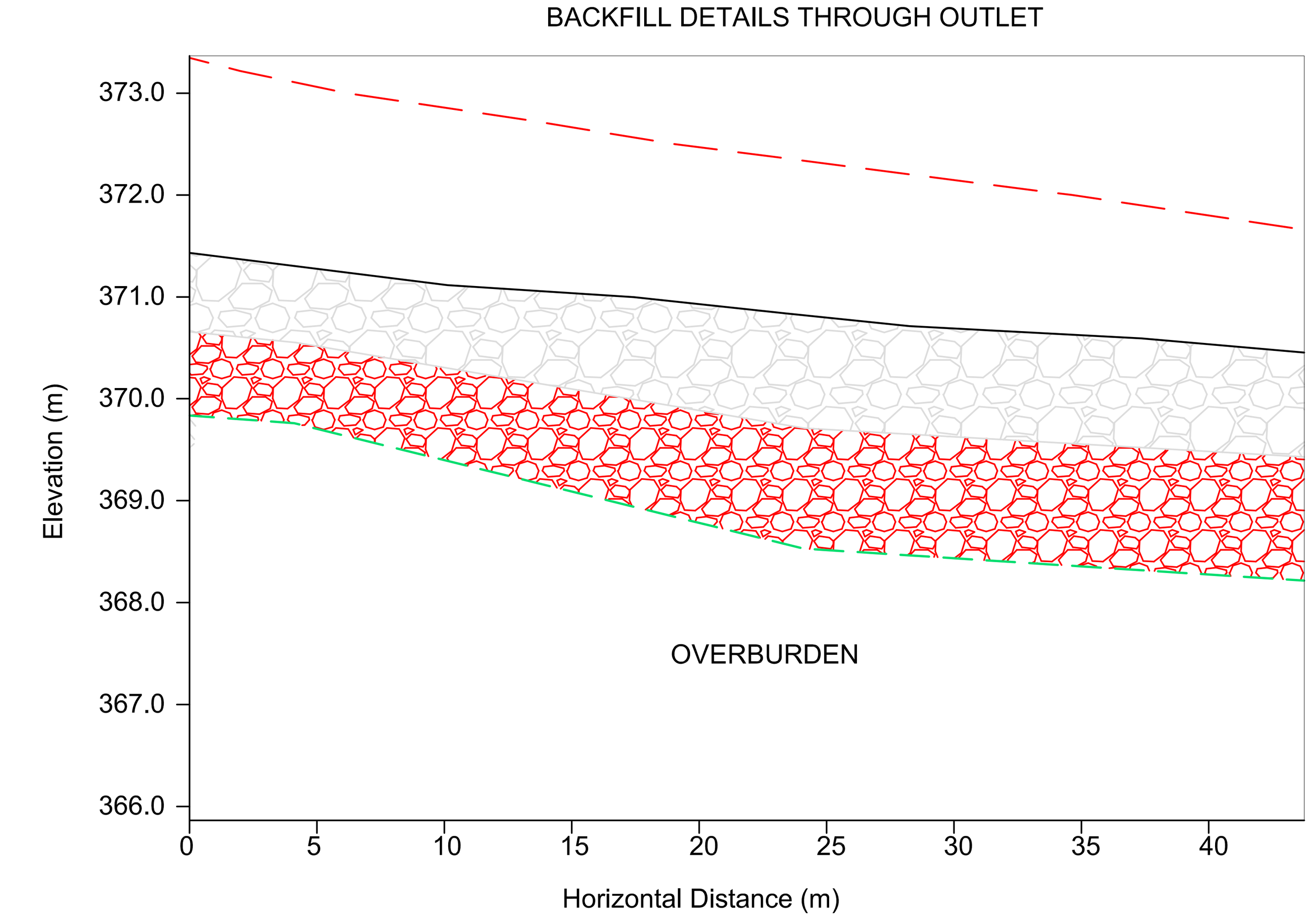
DETAIL 2
BACKFILL DETAILS THROUGH PLUNGE POOL



5m 0 5m 10m 15m

Horizontal Scale 1:250
VE= 4x

DETAIL 3
BACKFILL DETAILS THROUGH OUTLET



5m 0 5m 10m 15m

Horizontal Scale 1:250
VE= 4x

REV.	DATE	REVISION NOTES	DRAWN	CHECK	APPR.

SCALE:	AS SHOWN
DATE:	MAY 2008
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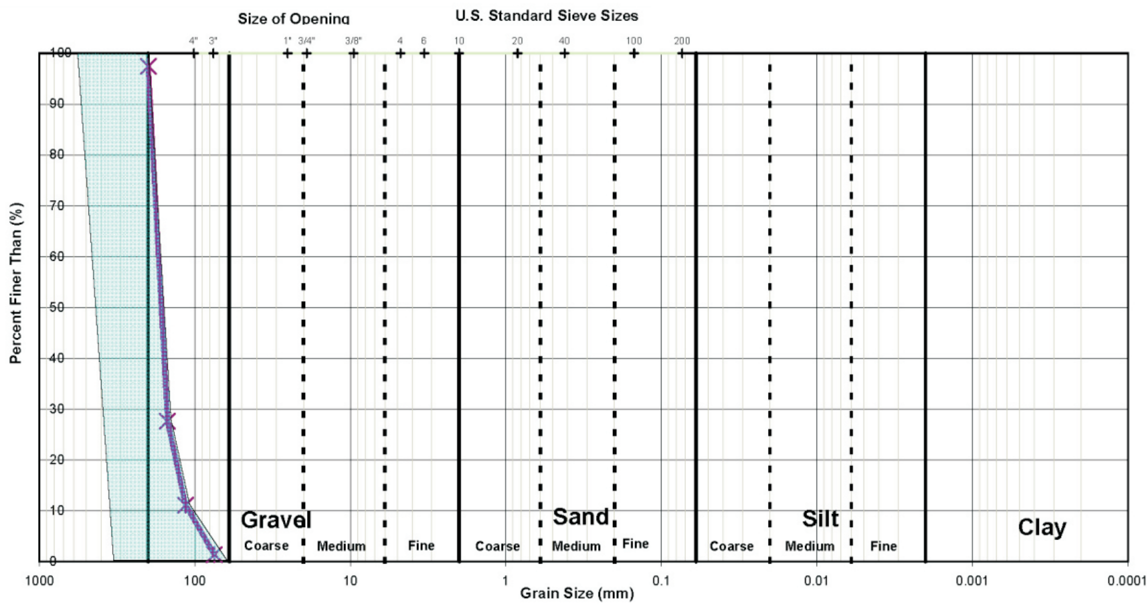
HYDRAULIC STRUCTURES AS-BUILT REPORT

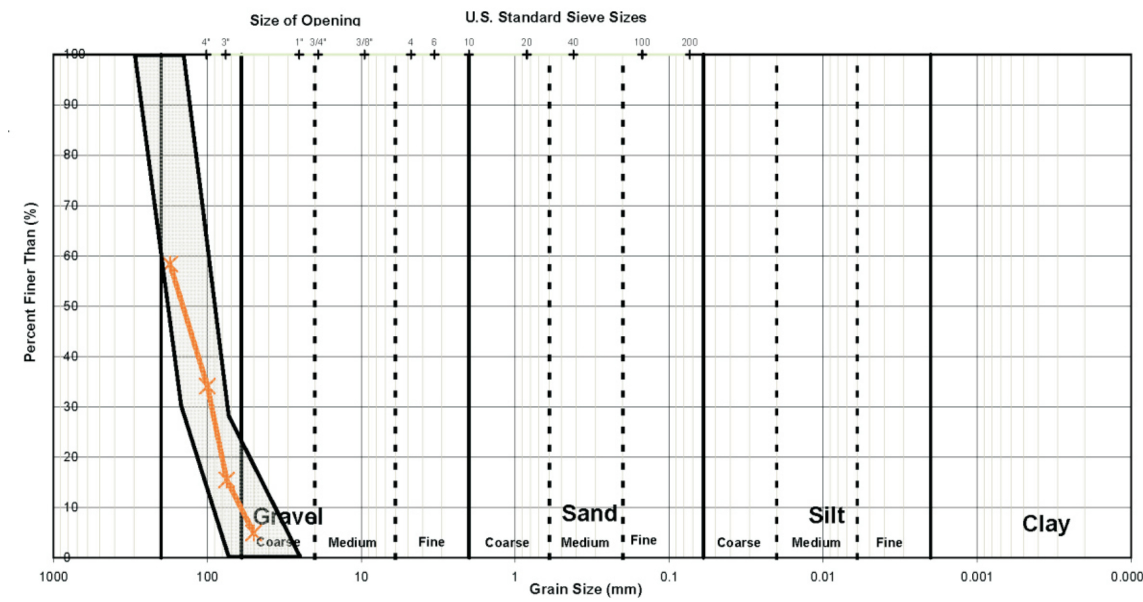
TITLE: WEST TWIN DIKE SPILLWAY
SECTION VIEWS 2

PROJECT No.:	FIGURE No.	REV.:
0255-012-05	5	0

Type 1 - Riprap



Type 2 - Riprap



Type 3 - Bedding Material

Note: Shaded areas denote design specifications (Golder 2005b).

CLIENT:

BREAKWATER RESOURCES LTD

REV.	DATE	REVISION NOTES	DRAWN	CHECKED	APPROVED
	-	- -	-	-	-
-	-	- -	-	-	-
-	-	- -	-	-	-

SCALE:	N/A
DATE:	MAY 2008
DRAWN:	SLF
DESIGNED:	GKC
CHECKED:	GKC
APPROVED:	GKC

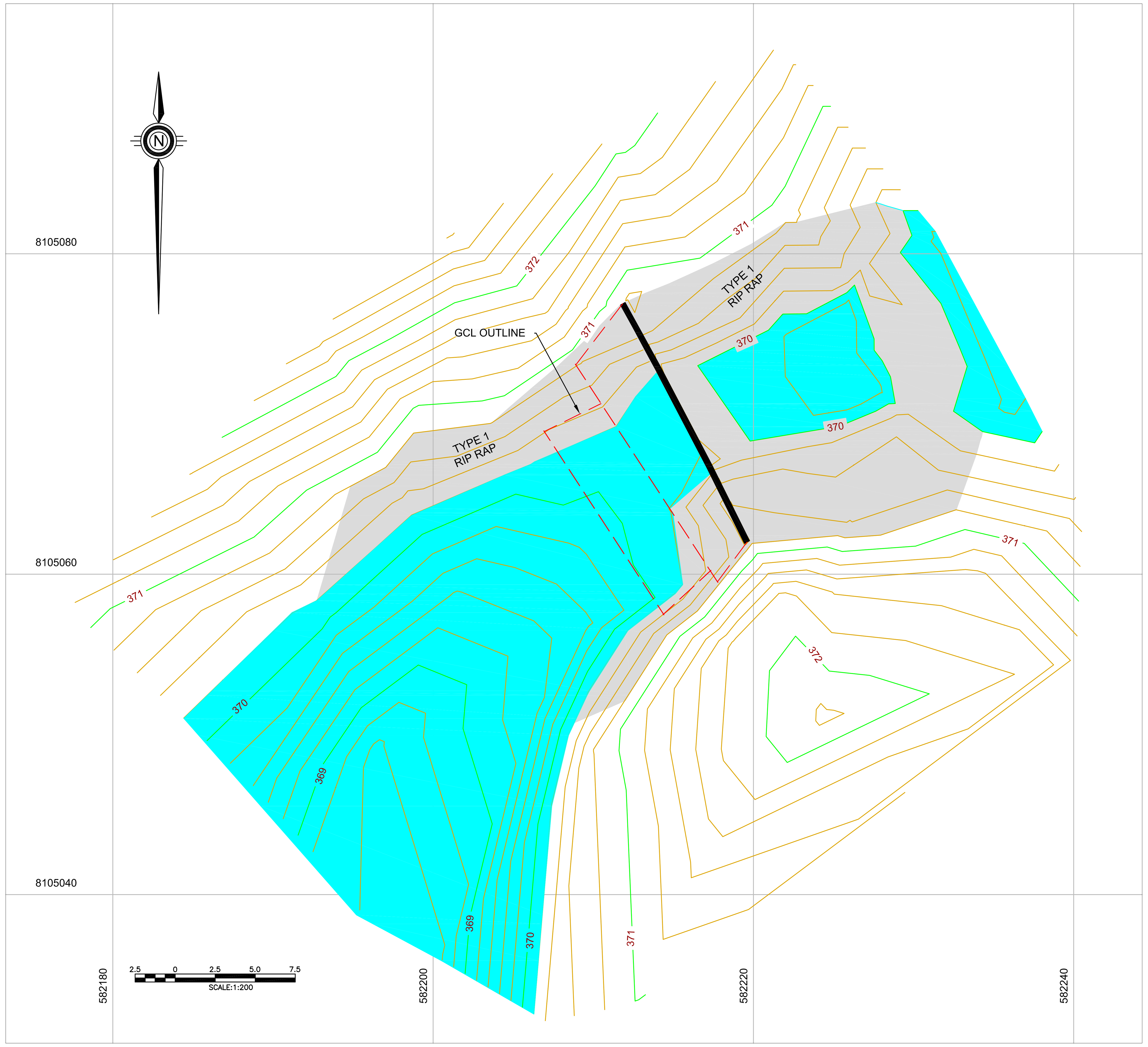
PROJECT: NANISIVIK MINE RECLAMATION HYDRAULIC STRUCTURES AS-BUILT REPORT

TITLE: ARMOUR MATERIALS GRAIN SIZE DISTRIBUTION ANALYSES

PROJECT No.	FIGURE No.	REV.
0255-012-05	6	0

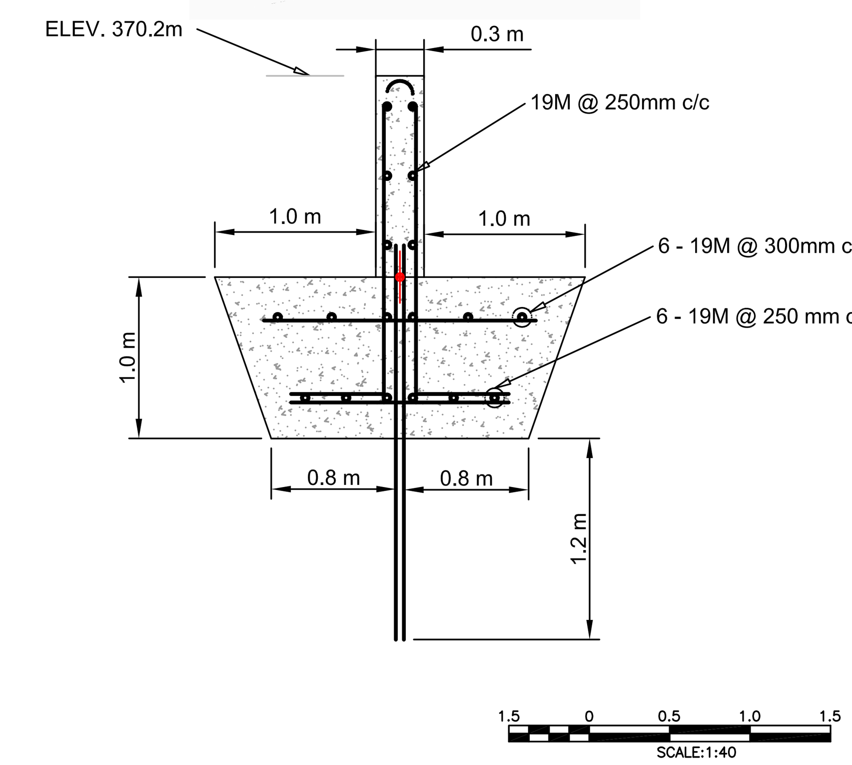
BGC Engineering Inc.
AN APPLIED EARTH SCIENCES COMPANY
Calgary Alberta
Phone: (403) 250-5185

PLAN VIEW - WEST TWIN OUTLET CHANNEL



DETAIL 1

TYPICAL REBAR, DOWEL AND WATERSTOP DETAILS

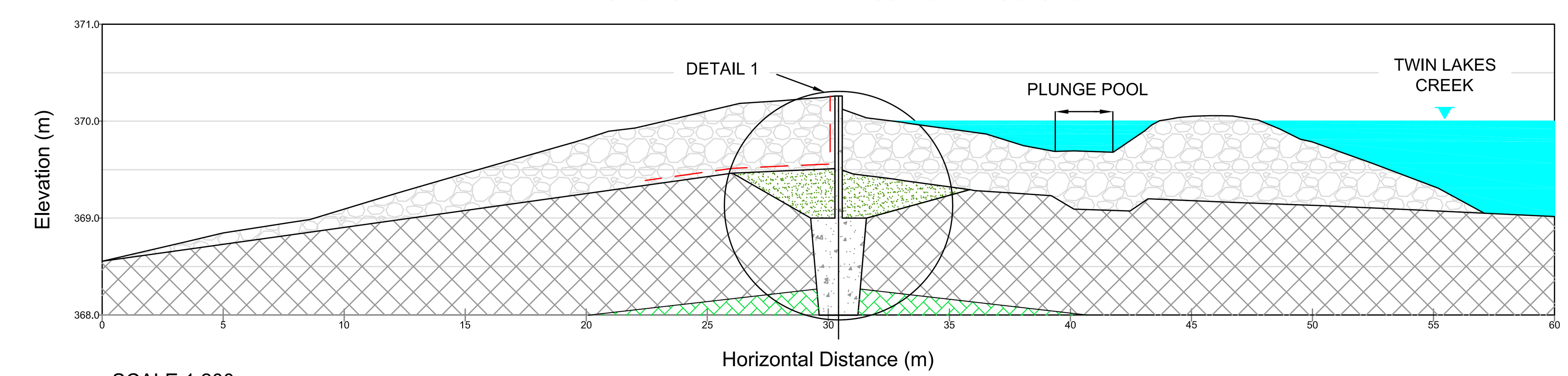


LEGEND:

- CONCRETE
- 19M REBAR
- WATER STOP
- TYPE 1 RIP RAP - D_{50} SIZE = 300mm MIN,
- BEDDING - D_{50} SIZE = 100 mm MIN.
- FROST-AFFECTED BEDROCK
- INTACT BEDROCK
- BENTONITE
- SCREENED SAND AND GRAVEL
- GEOSYNTHETIC CLAY LINER

SECTION A-A'

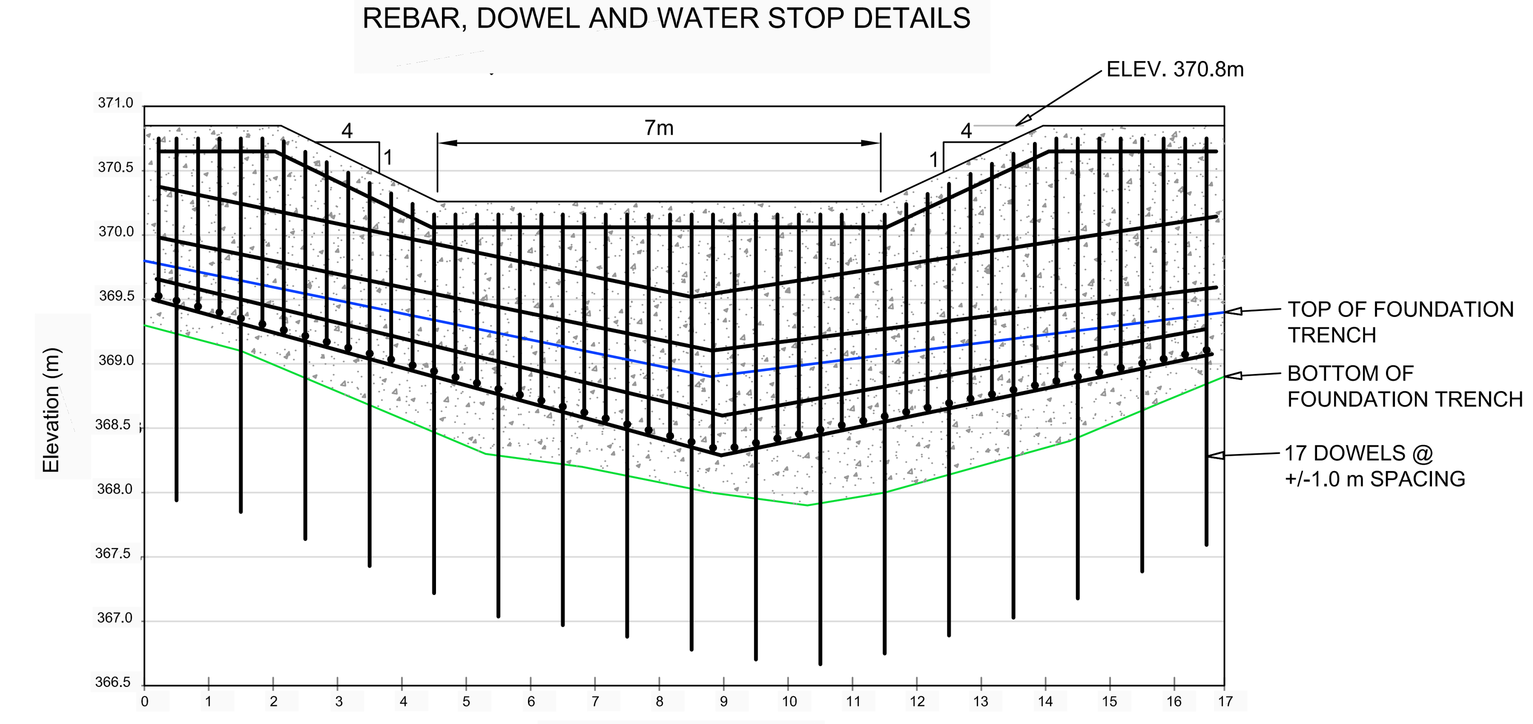
PROFILE OF THE WEST TWIN OUTLET STRUCTURE



SCALE 1:200
VE: 4x

SECTION B-B'

REBAR, DOWEL AND WATER STOP DETAILS

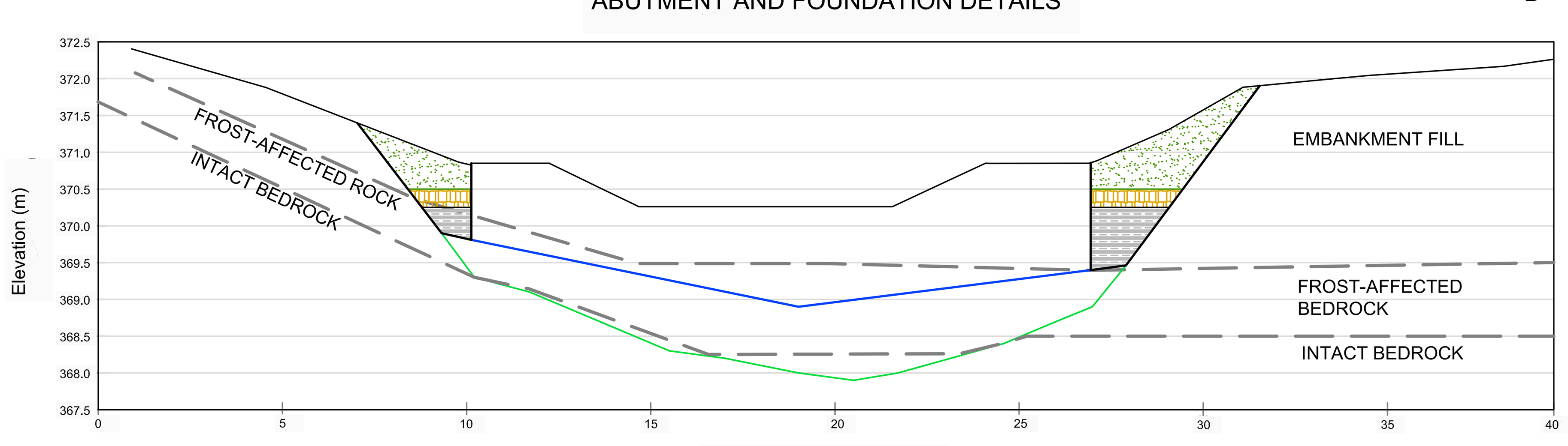


SCALE 1:100
VE: 2x

NOTES:
1. GRID SYSTEM PROVIDED ON PLAN VIEW IS BASED ON UTM, NAD 83, ZONE 16 COORDINATE SYSTEM.
2. ALL ELEVATIONS ARE PROVIDED IN METRES.
3. CONTOUR INTERVAL ON PLAN VIEW IS 0.25 m.
4. AS BUILT SURVEY DATA PROVIDED BY ATCON CONSTRUCTION INC. DRAWING FILE ABOC08D1.
5. LOCATION OF GCL LINER AS SHOWN IS BASED ON SITE OBSERVATIONS AND MEASUREMENTS TAKEN DURING INSTALLATION AND SHOULD BE CONSIDERED AS APPROXIMATE.

SECTION B-B'

ABUTMENT AND FOUNDATION DETAILS



SCALE 1:150
VE: 2x

REV.	DATE	REVISION NOTES	DRAWN	CHECK	APPR.

SCALE:	AS SHOWN
DATE:	MAY 2008
DRAWN:	REM
DESIGNED:	GKC
CHECKED:	GKC
APPROVED:	GKC

PROFESSIONAL SEAL:

BGC ENGINEERING INC.
AN APPLIED EARTH SCIENCES COMPANY

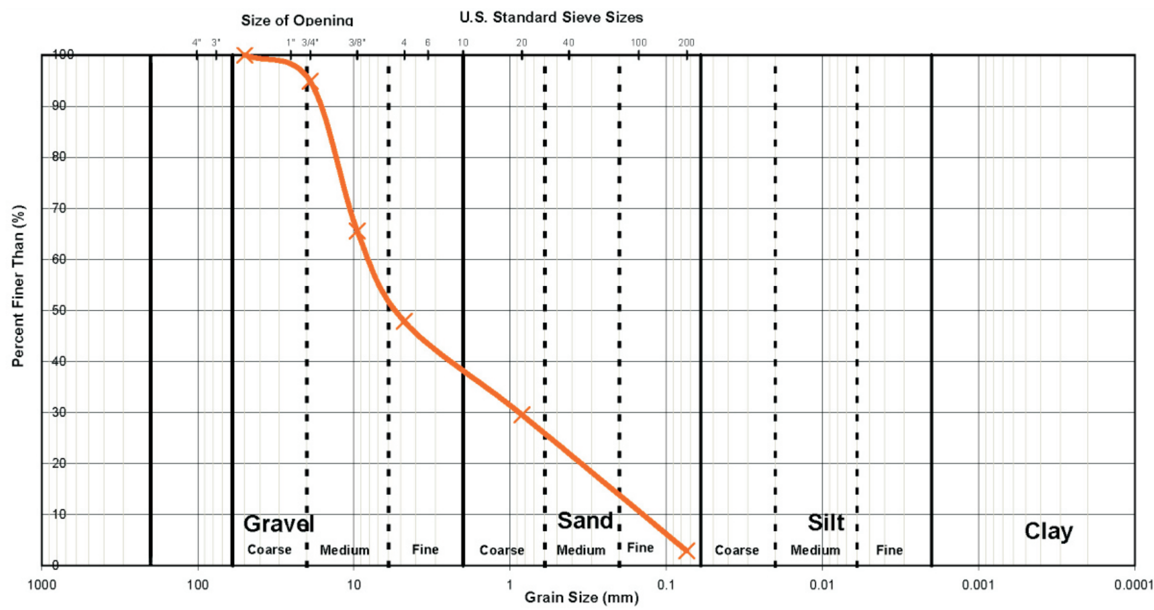
CLIENT: BREAKWATER RESOURCES LTD

PROJECT: NANISIVIK MINE RECLAMATION HYDRAULIC STRUCTURES AS-BUILT REPORT

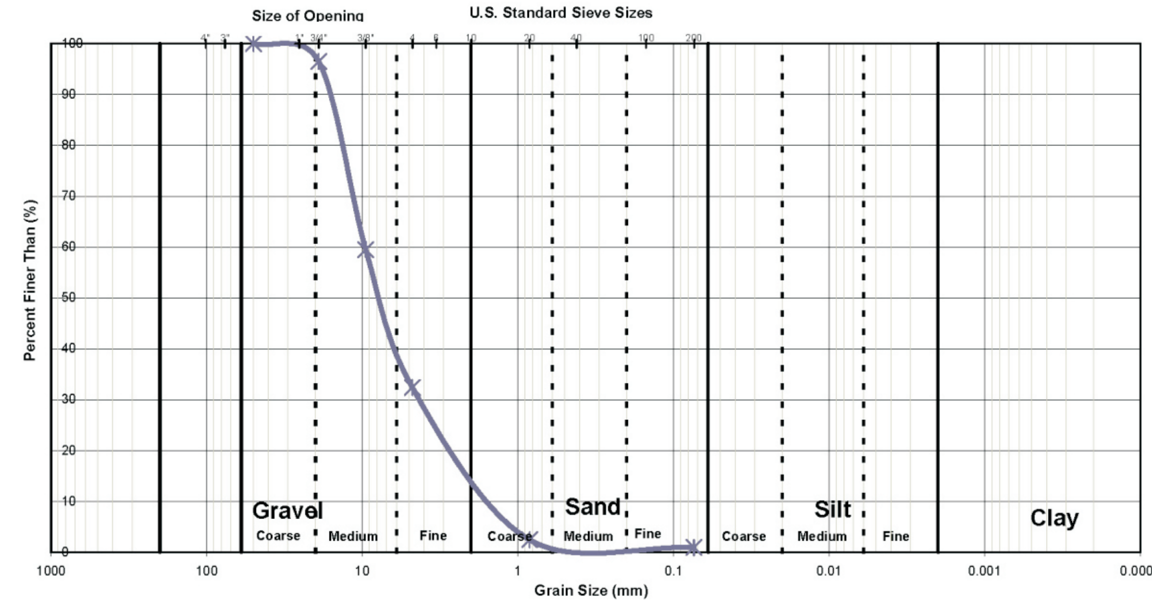
TITLE: WEST TWIN OUTLET CHANNEL AS-BUILT SECTIONS AND DETAILS

PROJECT No.:	FIGURE No.	REV.:
0255-012-05	7	0

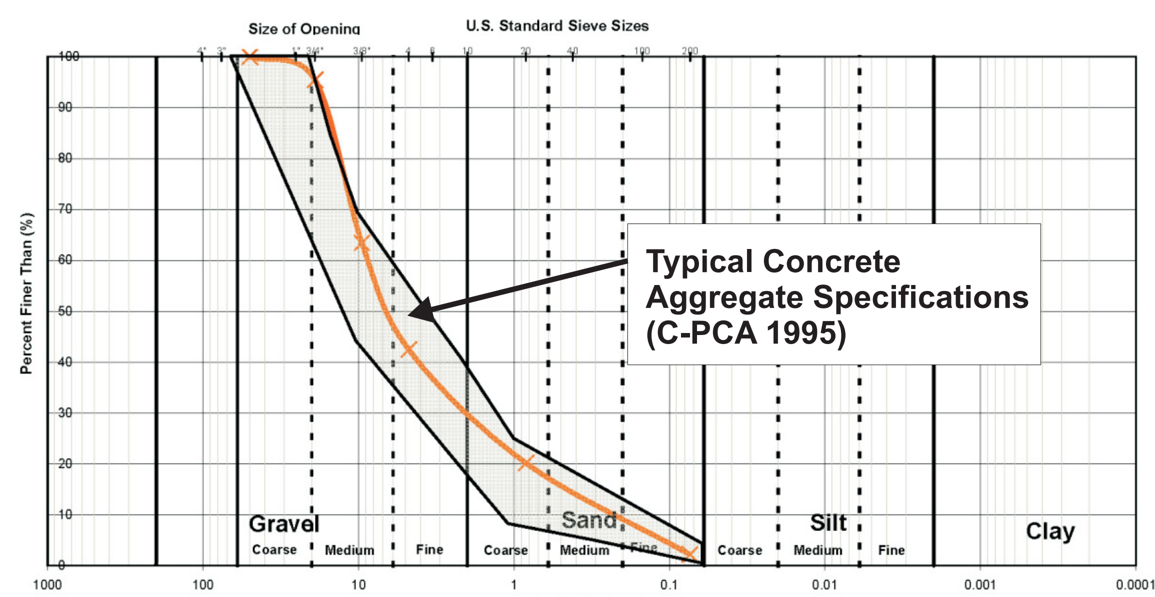
Fine-Grained Aggregate



Coarse-Grained Aggregate



Combined Aggregate



CLIENT:

BREAKWATER RESOURCES LTD

REV.	DATE	REVISION NOTES	DRAWN	CHECKED	APPROVED
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-	-	- -	-	-	-
-	-	- -	-	-	-

SCALE:	N/A
DATE:	MAY 2008
DRAWN:	SLF
DESIGNED:	GKC
CHECKED:	GKC
APPROVED:	GKC

PROJECT: NANISIVIK MINE RECLAMATION HYDRAULIC STRUCTURES AS-BUILT REPORT

TITLE: CONCRETE AGGREGATE GRAIN SIZE DISTRIBUTION ANALYSES

PROJECT No.	FIGURE No.	REV.
0255-012-05	8	0

BGC Engineering Inc.
AN APPLIED EARTH SCIENCES COMPANY
Calgary Alberta
Phone: (403) 250-5185

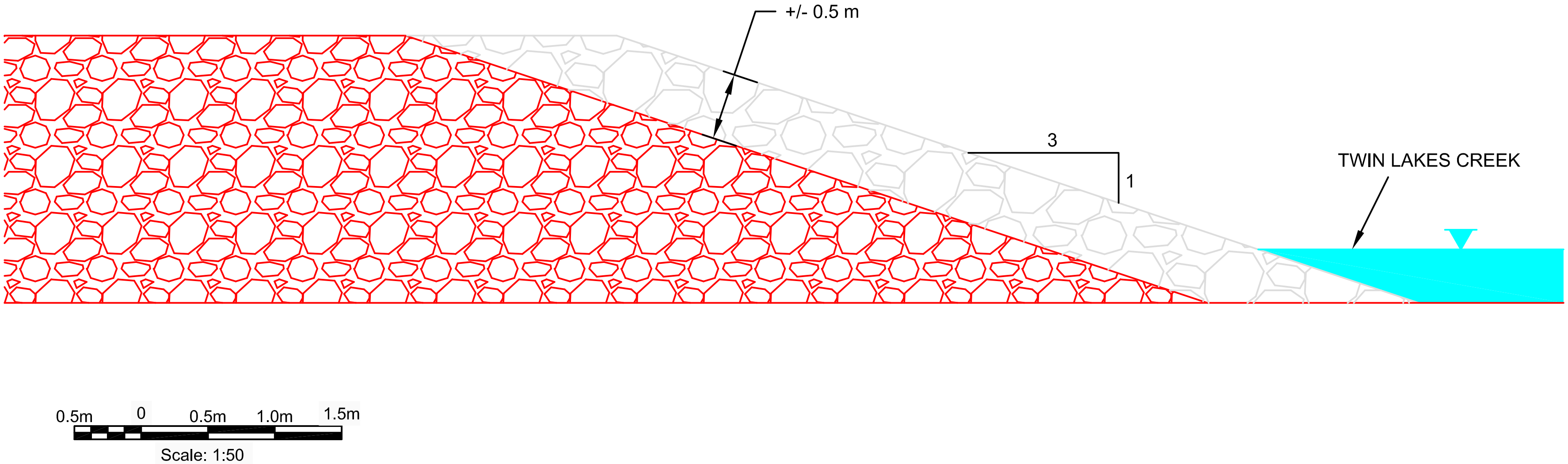
NOTES:

1. DRAWING IS A SCHEMATIC TO CONVEY AS-BUILT INFORMATION. NO AS-BUILT SURVEY WAS COMPLETED.
2. SCALE IS APPROXIMATE.
3. SEE APPENDIX V FOR AS-BUILT PHOTOS.

LEGEND:.

TYPE 1 RIP RAP D_{50} = 300mm

TWIN LAKES SAND AND GRAVEL



SCALE: AS SHOWN	DATE: MAY 2008	DRAWN: REM	DESIGNED: GKC	CHECKED: GKC	APPROVED: GKC

BGC ENGINEERING INC.
AN APPLIED EARTH SCIENCES COMPANY
Calgary, AB Phone: (403) 250 5185

CLIENT: BREAKWATER RESOURCES LTD

PROJECT
NANISIVIK MINE RECLAMTION
HYDRAULIC STRUCTURES AS-BUILT REPORT

TITLE
EAST TWIN CREEK DIVERSION CHANNEL
DIVERSION DIKE RECLAMATION MEASURES

PROJECT No. 0255-012-05

FIGURE: 9

REV.

0255-012-05 Diversion Dike.dwg

APPENDIX I
CONSTRUCTION PHOTOS
WEST TWIN DIKE SPILLWAY

Photo 1 May 15, 2005
Drilling for subsequent blast at spillway inlet.
Note cones identifying drill holes.

Photo 2 May 14, 2005
Blast at spillway.

Photo 3 May 11, 2005
Stockpiling spillway excavation materials at floor of Mt. Fuji Quarry.

Photo 4 May 21, 2005
Stockpiling Type 1 riprap adjacent to spillway outlet.

Photo 5 May 20, 2005
Stockpile of Type 2 riprap.
Note field book for scale.
Also note red staining on surface of rock.

Photo 6 May 20, 2005
Stockpile of Type 1 riprap.
Note hard hat for scale.

Photo 7 May 17, 2005
Excavating material from inlet portion of spillway.

Photo 8 May 18, 2005
Excavated spillway channel near inlet
Note near vertical intact bedrock on right side of photo.
Also note bench constructed near mid-point of slope on right side of photo.

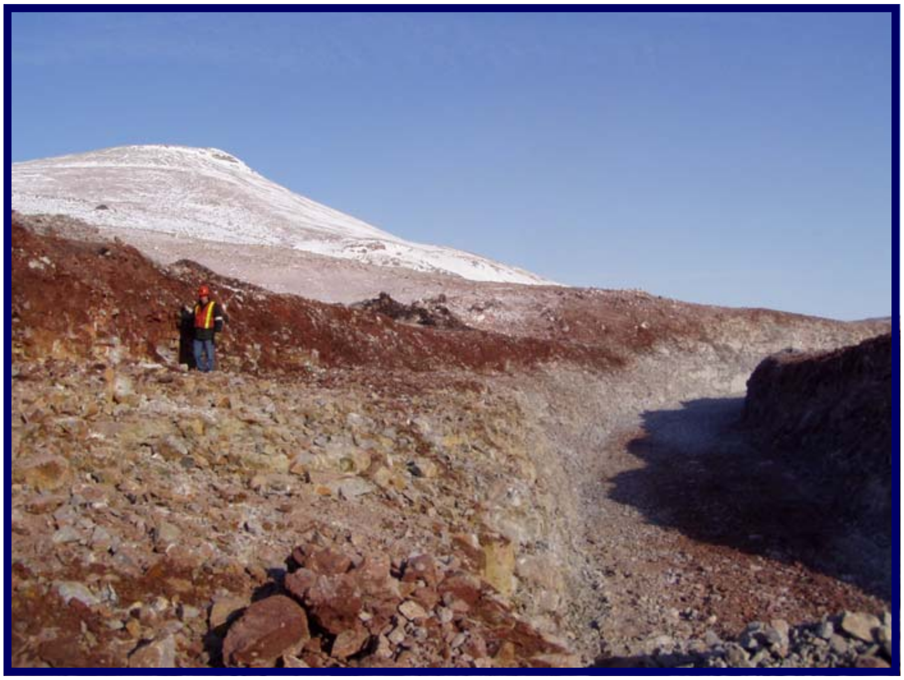


Photo 9 May 18, 2005
View looking upstream. Inlet portion of spillway excavation as seen from access riprap.
Note bench constructed along slope on left hand side of photo at interface between intact and frost bedrock.

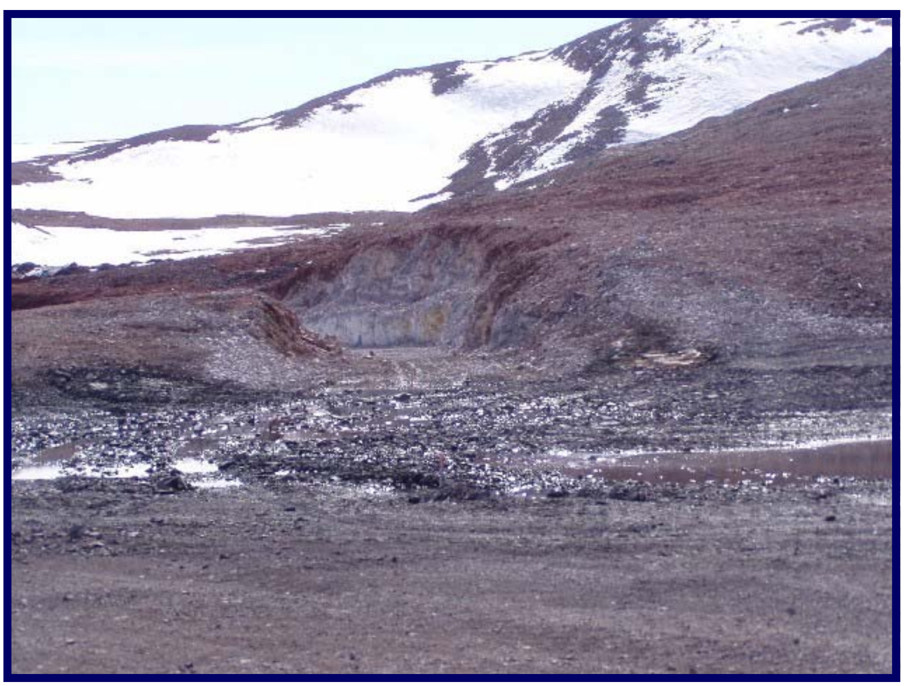


Photo 10 June 6, 2005
View of spillway inlet as seen from Surface Cell cover.
Note armour layer not yet applied to Surface Cell cover at the time.

Photo 11 May 25, 2005
View of inlet portion of spillway.
Note transition between red stained frost affected bedrock and grey intact bedrock.

Photo 12 May 14, 2005
View of sidewall of spillway exaction.
Note transition between red-stained, frost-affected bedrock and grey intact bedrock.

Photo 13 May 18, 2005
Access ramp between inlet and chute portions of spillway.

Photo 14 August 28, 2007
Access ramp into spillway.

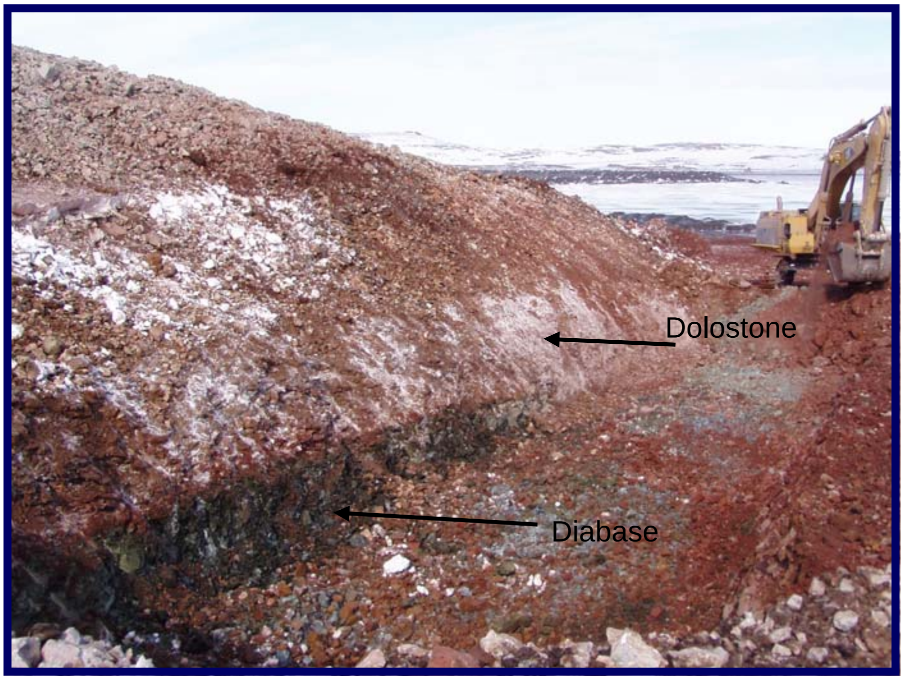


Photo 15 May 18, 2005
Initial excavation of lower portion of spillway chute.
Note exposed diabase (dark green colour) in sidewall.

Photo 16 May 19, 2005
Initial excavation of lower portion of spillway chute.
Note transition between dark green diabase and grey dolostone in sidewall and base of spillway.

Photo 17 May 21, 2005
Excavation of upper portion of spillway chute.
Note poor quality of rock in sidewall of excavation.

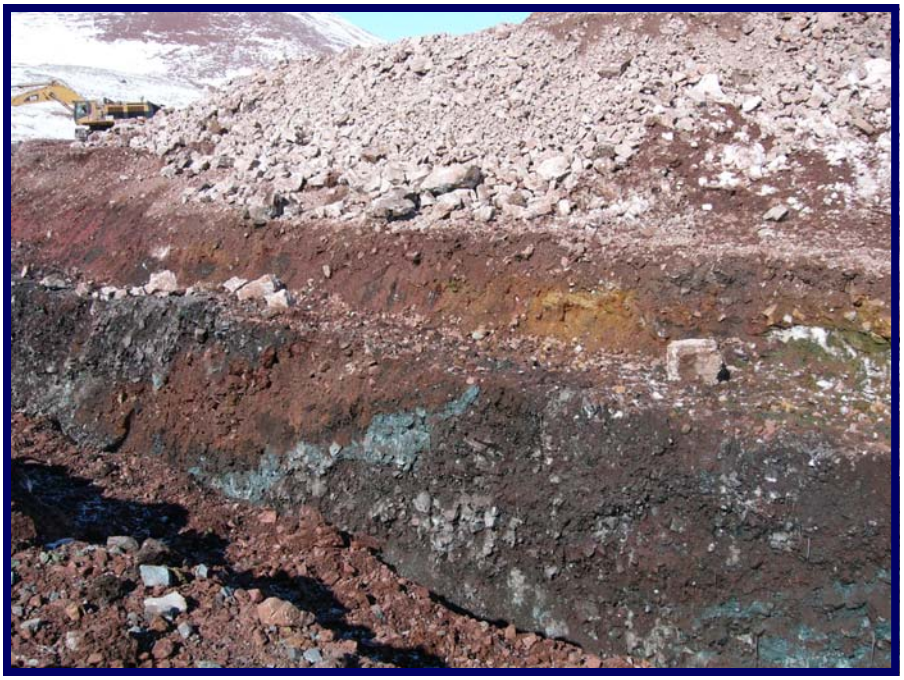


Photo 18 May 21, 2005
Left hand sidewall of spillway excavation in chute.
Note weathered and altered diabase material in sidewall of excavation.

Photo 19 May 26, 2005
Backfilling chute portion of spillway with Type 2 riprap.

Photo 20 May 26, 2005
Backfilled spillway excavation with Type 2 riprap.

Photo 21 Type 2 May 27, 2005
Backfilling spillway chute with Type 1 riprap.

Photo 22 May 27, 2005
Backfilling spillway chute with Type 1 riprap.

Photo 23 - August 2006
View of overburden bench on right hand side of spillway excavation.

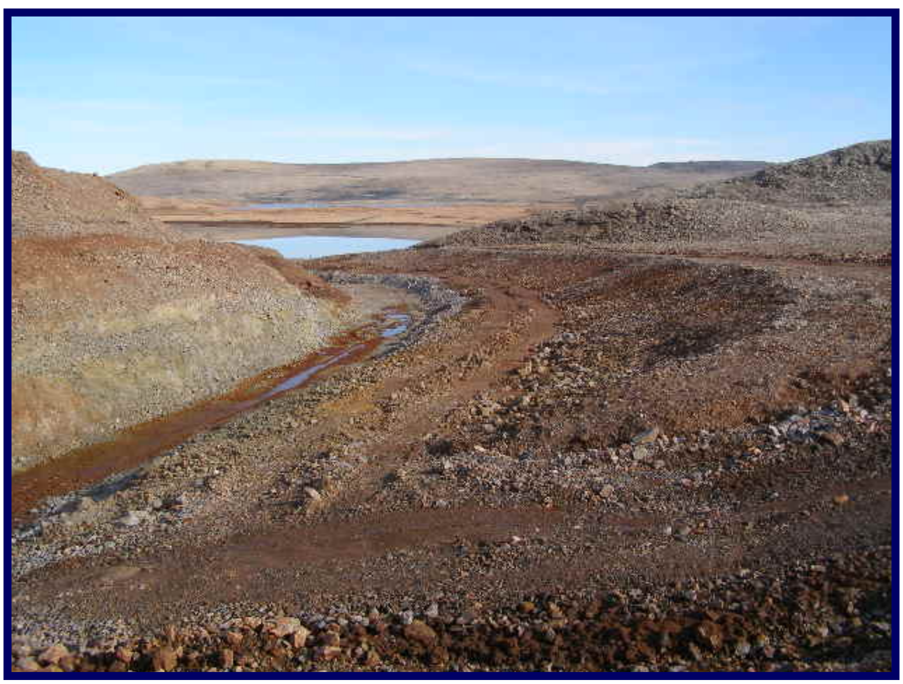


Photo 24 - August 2006
View of overburden bench on right hand side of spillway excavation.

Photo 25 May 17, 2005
Dolostone outcrop on left hand side of spillway excavation prior to benching.

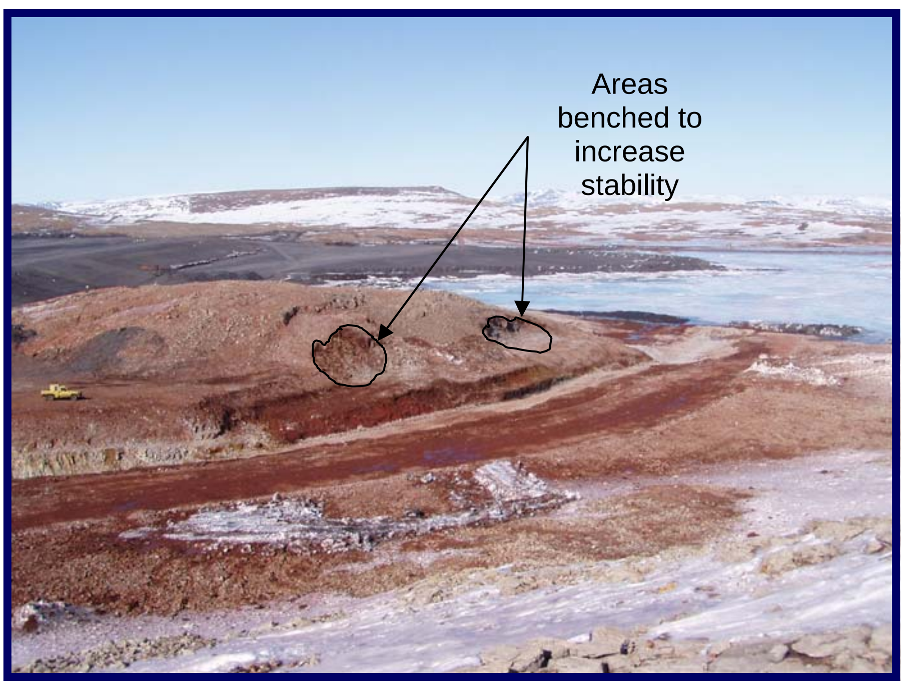


Photo 26 May 27, 2005
Dolostone outcrop after benching.

Photo 27 May 25, 2005
Excavation of spillway outlet channel.

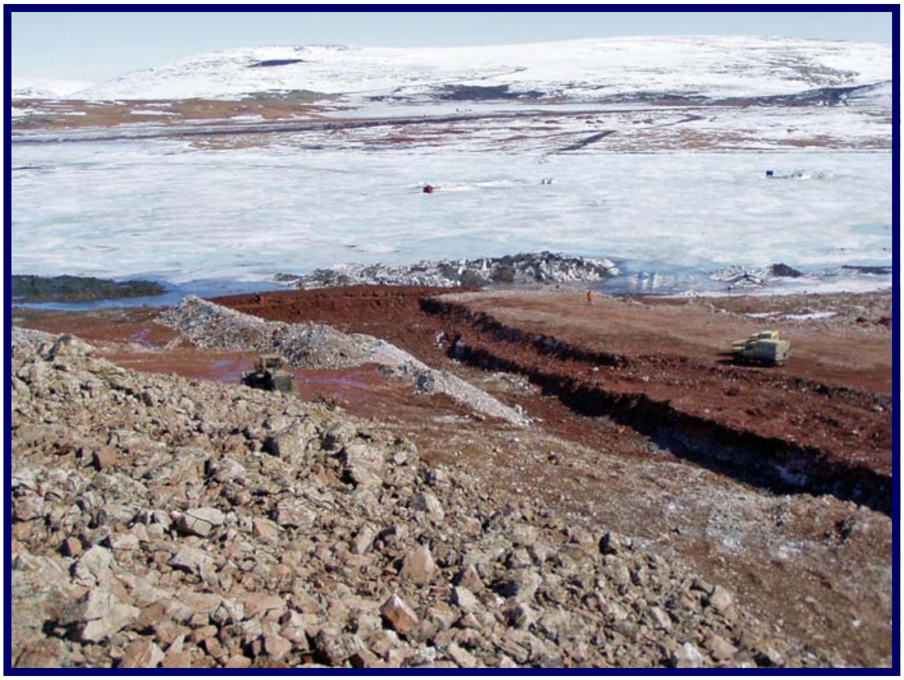


Photo 28 May 25, 2005
Excavated spillway outlet channel.
Note plug left at outlet to keep area from backflooding.

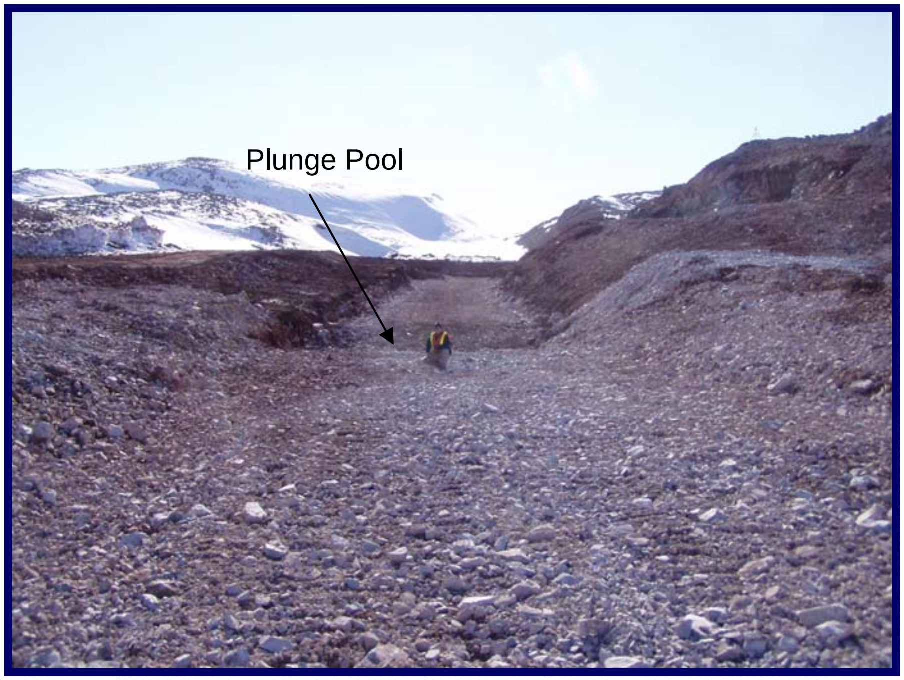


Photo 29 May 27, 2005
View looking upstream. Armoured outlet channel.
Note person in photo is standing just downstream of plunge pool.

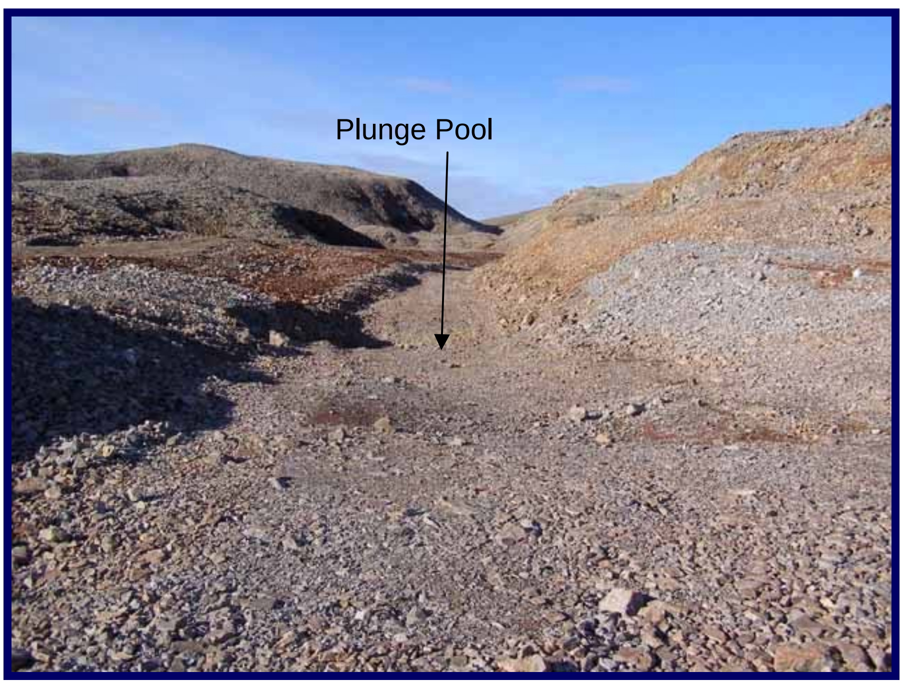


Photo 30 August 28, 2007
View of spillway looking up gradient after armouring side walls.

Photo 31
Applying Type 1 riprap to right hand side of spillway channel through chute section.

Photo 32 August 31, 2007
Bottom of Spillway looking downslope. Note riprap applied to left side of spillway.

Photo 33 May 27, 2005
Water deflection berm near spillway access ramp.
Note person on berm for scale.

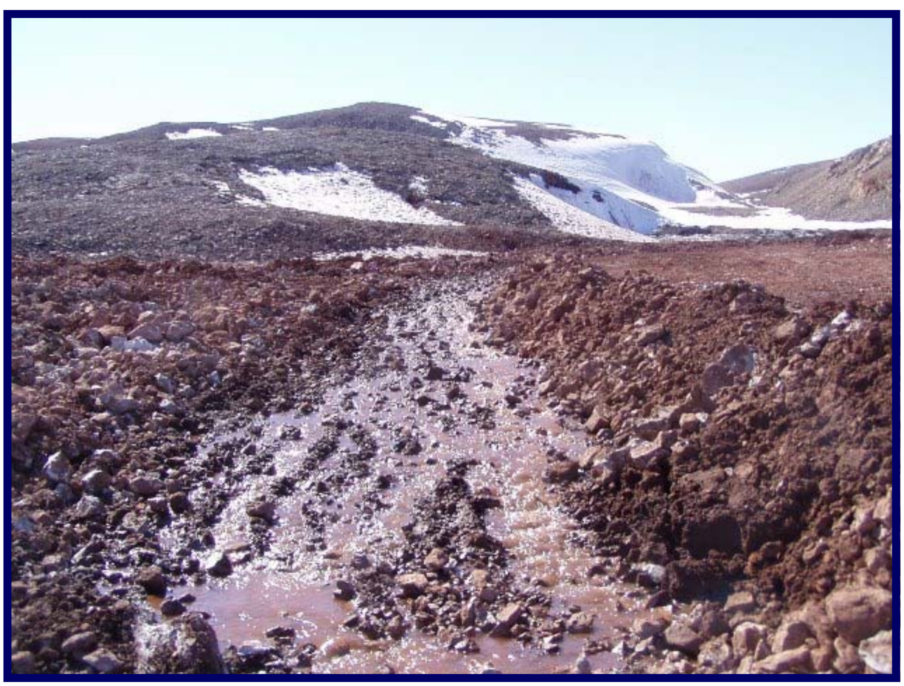


Photo 34 June 10, 2005
Water diversion ditch adjacent to access road.

Photo 35 August 28, 2007
Flow through entrance area of West Twin Dike Spillway.

Photo 36 August 28, 2007
Spillway outlet as seen from Reservoir control structure.

APPENDIX II
CONSTRUCTION PHOTOS
WEST TWIN OUTLET CHANNEL

Photo 1 - July 2004
Previous concrete outlet structure.

Photo 2 - July 2004
Previous concrete outlet structure.

Photo 3 - September 2005
Trench excavation for outlet trench.

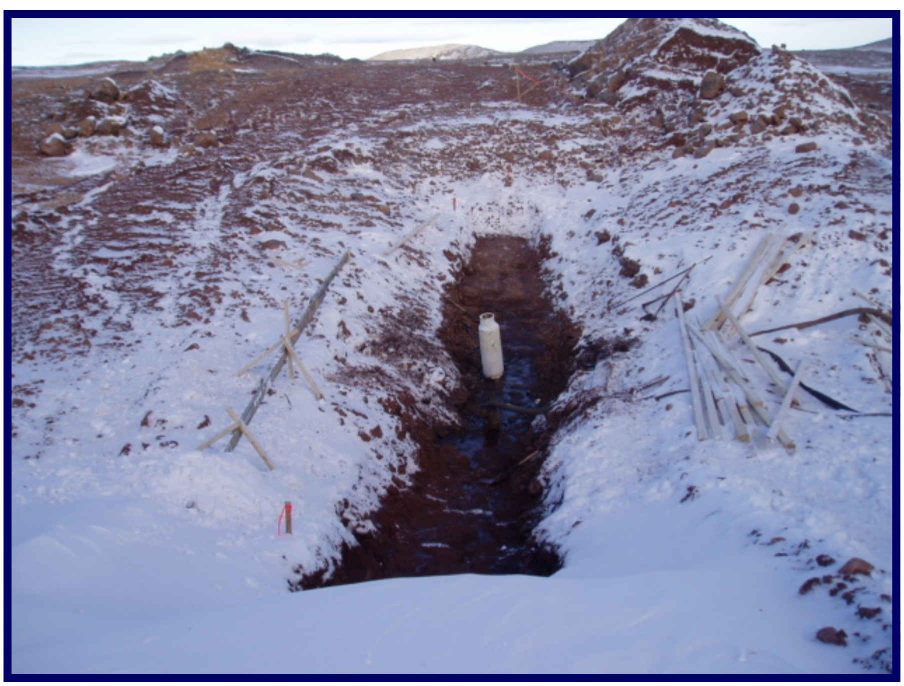


Photo 4 - September 2005
De-watered outlet trench.

Photo 5 - September 12, 2005
Grouting of the dowels into competent bedrock in foundation trench.

Photo 6 - September 14, 2005
Rebar mesh constructed for the foundation trench.

Photo 7 - September 17, 2005
Close up view of the rebar mesh constructed for the foundation trench.

Photo 8 - September 19, 2005
Sump area and submersible pump used to de-water the foundation trench excavation .

Photo 9 - September 20, 2005
Excavator adding aggregate to the concrete mixer.

Photo 10 - September 20, 2005
Concrete hopper used to place the concrete in the foundation trench excavation.

Photo 11 - September 20, 2005
Concrete being placed into the foundation trench excavation.

Photo 12 - September 20, 2005
Manually agitating concrete placed in foundation trench excavation.

Photo 13 - September 20, 2005
Manually finishing, concrete around rebar in foundation trench.

Photo 14 - September 20, 2005
Concrete placed in foundation trench.

Photo 15 - September 20, 2005
Completed concrete pour in foundation trench.
Note the water stop placed into the foundation.

Photo 16 - September 20, 2005
Tarps and heaters used to allow the concrete to cure after the concrete pour in foundation trench.

Photo 17 - September 23, 2005
Cured concrete in foundation trench.
Preparing to construct rebar mesh for cut-off wall.

Photo 18 - September 23, 2005
Horizontal rebar constructed for the cut-off wall.

Photo 19 - September 25, 2005
Concrete forms being constructed for the cut-off wall.

Photo 20 - September 29, 2005
The form constructed for the cut-off wall.

Photo 21 - September 29, 2005
Placing concrete into forms to construct the cut-off wall.

Photo 22 - September 29, 2005
Slump test from concrete used in the cut-off wall.

Photo 23 - September 29, 2005
Concrete placed in the cut-off forms.

Photo 24 - October 3, 2005
Cut-off wall following the removal of the forms.

Photo 25 - October 3, 2005
Cut-off wall following the removal of the forms.

Photo 26 - October 7, 2005
Bentonite powder placed at the south abutment to 370.2 m elevation.

Photo 27 - October 7, 2005
Screened sand and gravel placed at the south abutment on top of the bentonite powder.

Photo 28 - October 7, 2005
Excavator placing sand and gravel adjacent to the cut-off wall.

Photo 29 - October 7, 2005
Outlet channel prior to placement of riprap.

Photo 30 - October 7, 2005
Excavation of the stilling pool downstream of cut-off wall.

Photo 31 - October 7, 2005
D8 dozer further removing materials downstream of cut-off wall down to bedrock.

Photo 32 - October 7, 2005
Dozer placing the riprap at the Outlet Channel.

Photo 33 - October 8, 2005
Placement of riprap upstream of cut-off wall.

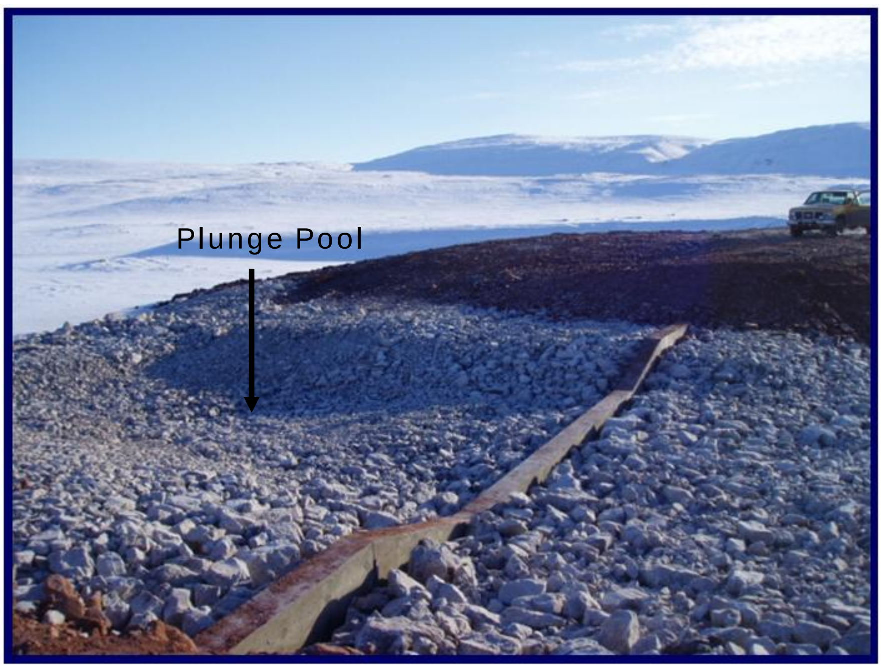


Photo 34 - October 8, 2005
Shows a view of the armoured West Twin Outlet Channel, including the downstream plunge pool.

Photo 35 - October 8, 2005
View of armoured channel leaking upstream.

Photo 36 - August 2006
Water flowing through channel in August 2006.

Photo 37 - August 2006
View of completed West Twin Outlet Channel.

Photo 39 - August 2006
Plunge pool at West Twin Outlet.

Photo 39 - August 30, 2007
The GCL liner placed upstream of the wall post construction.

Photo 40 - September 1, 2007
Water flowing in WT Outlet channel post GCL installation.

APPENDIX III
WEST TWIN OUTLET CHANNEL
CONSTRUCTION SPECIFICATIONS
(GOLDER 2005b)

BREAKWATER RESOURCES

NANISIVIK MINE

WEST TWIN OUTLET CHANNEL

TECHNICAL SPECIFICATIONS

Prepared by: **Golder Associates Ltd.**
Project No. **051-118011**
Date: **September, 2005**
Revision No: **A**

Golder Associates

TABLE OF CONTENTS

The following Construction Drawing forms the basis of this scope of work and constitutes part of the Contract Documents:

REPAIR OF THE TAILINGS DAM NO. 1 SPILLWAY

Drawing No.	Rev. No.	Title
24	A	West Twin Outlet Channel Plan, Section and Details

1.0 SCHEDULE

.1 Within one week after the award of the contract, the Contractor shall submit a detailed construction schedule for review by the Owner. The schedule shall include the date of completion of application of re-vegetation measures.

2.0 ENVIRONMENT, HEALTH AND SAFETY

.1 The Contractor shall carry out the Work in compliance with a Health and Safety Policy to be provided by Nanisivik Mine.

.2 Use of haul roads within the Nanisivik Mine property shall be subject to the approval of the Owner. The Contractor shall observe all existing rules regarding safety of operating vehicles.

.3 Environmental requirements regarding protection of surface water are described in Specification No. 1000.

1.0 GENERAL

.1 The Contractor shall provide all labour, equipment, materials and supervision necessary to complete the work to be done as outlined in section 2.0.

.2 All materials and procedures are subject to the approval of the Engineer.

.3 The Contractor shall be responsible for complying with all government regulations.

.4 The Contractor shall be responsible for care of water measures required to complete the construction, according to Section No. 1000.

2.0 WORK TO BE DONE

General

.1 Lower the water level in the Polishing Pond and/or construct suitable coffer dams to facilitate the construction.

.2 Make arrangements to obtain fill materials from on-site borrow areas.

.3 Arrange for Source Approval of all borrow sources as required by Specification 0500.

.4 During the period of construction, manage borrow areas in an efficient manner as outlined in Specification 0700.

.5 At the completion of construction, stabilize the waste rock slopes in the borrow areas as required by Specification 0700.

.6 Provide all surveying service necessary during construction, except those which are specifically the responsibility of the Owner. (Specification No. 0800).

.7 When the work is completed, remove all materials, equipment, construction debris and the like from the work sites. Dispose of the same in areas designated by the Owner.

Access Roads

.1 Revise the alignments of existing access roads where necessary to avoid the spillway construction, such as areas of backslope cuts, the tailings waste berm, etc.

.2 Reinforce the existing access roads as necessary to support construction traffic.

.3 Maintain the access roads throughout the period of construction.

Construction of the West Twin Outlet Channel

.1 Demolish and remove the existing discharge structure and dispose of the debris as directed by the Engineer

.2 Excavate test pits if and where directed by the Engineer.

.3 Excavate overburden from the cutoff wall foundation trench, exposing the bedrock surface.

.4 Excavate loose, friable or frost-shattered bedrock from under the cutoff wall foundation trench, exposing bedrock which is judged by the Engineer to be "competent".

.5 Clean the surface of the "competent" bedrock, which is exposed in the base of the cutoff wall foundation trench and apply slush grout.

.6 Drill and grout dowels into the "competent" bedrock.

.7 Place re-bars and concrete in the cutoff wall foundation trench.

.8 Install an approved waterstop beneath the cutoff wall.

.9 Form the cutoff wall, place re-bars and pour the concrete.

.10 Excavate soil from the outlet channel to achieve the lines and grades shown on the Construction Drawing.

.11 Obtain all fill materials from designated borrow areas, process the materials as necessary, transport them to the work sites, place and (where required) compact the fill materials to the required grades shown on the Construction Drawings.

3.0 MATERIALS AND WORK EXCLUDED FROM THE WORK

3.1 Materials to Be Supplied by the Owner

The following will be supplied to the Contractor by the Owner at no charge:

.1 The survey services to be supplied by the Owner as outlined in Specification 0800,

.2 Use (without royalty) of the site borrow areas to obtain and produce fill materials.

1.0 GENERAL

This Specification describes the requirements for the demolition and removal of the existing discharge structure, for the stripping of any organic or deleterious materials, and for the disposal of waste materials produced.

The following related work is excluded from the scope of this section: any stripping and overburden removal from borrow pit areas (in Section 0700).

2.0 DEMOLITION OF THE EXISTING DISCHARGE STRUCTURE

.1 The existing decant structure shall be demolished and all demolition debris shall be removed from the area.

.2 Tear down the steel components and salvage the steel and valves for re-use or as scrap, as directed by the Owner.

.3 Demolish the reinforced concrete components of the existing decant structure. Dispose of the rubble as directed by the Owner. Subject to the Owner's approval, suitably sized pieces of clean concrete rubble may be placed on the shoreline of the Polishing Pond adjacent to the outlet structure to serve as erosion protection. Otherwise, the rubble shall be disposed as directed by the Owner.

3.0 STRIPPING

.1 Strip organic soil, organic debris or other perishable materials from the footprint of the outlet channel.

.2 Stripped organic materials shall be stockpiled separately in a location approved by the Owner.

1.0 GENERAL

This Specification covers those work items which are required for foundation preparation prior to placement of fill for the construction of the outlet channel. Excluded from the scope of this specification are the demolition of the existing discharge structure and stripping, both of which are covered in Specification 0200.

2.0 EXCAVATION

.1 Excavation shall be carried out as required to achieve the lines, grades and dimensions shown on the Drawings and/or as required to expose suitable subgrade materials, which shall be determined by the Engineer.

.2 Excavation lines and grades shown on the Construction Drawing have been determined from the topographic map. The Engineer may modify the actual excavation lines and grades from those shown on the Construction Drawing to adjust for inaccuracies in the contour map.

.3 Excavation lines and grades shown on the Construction Drawing are based on assumed bedrock conditions. The Engineer may modify the actual excavation lines and grades from those shown on the Construction Drawing to adjust for actual bedrock conditions.

.4 Safe and stable side slopes shall be maintained as required in accordance with the requirements of applicable legislation. It may be found necessary or advantageous to vary slopes, grades or dimensions of excavations from those specified or shown. Such proposed variations shall be reviewed by the Engineer and must be approved prior to being put into effect.

.5 Where excavated material meets the fill specifications for a particular zone (Specification 0500), the material may be reused as fill, subject to the Engineer's approval.

.6 Material excavated from the spillway subgrade, which is not reused as fill shall be disposed of at a nearby location in a manner and location which is subject to the Engineer's approval.

3.0 BEDROCK SURFACE PREPARATION

.7 This section applies to the preparation of the cutoff wall foundation trench.

.8 General: The surface of bedrock is irregular; therefore bedrock may be encountered at locations other than those suggested by profiles shown on the drawings or suggested by existing geotechnical information.

.9 Excavation: The overburden shall be removed, exposing the existing bedrock surface. All loose, soft, disintegrated and ice-shattered rock fragments which are exposed in the cutoff wall foundation trench shall be removed using an excavator equipped with a non-toothed bucket (or a toothed bucket with a plate welded across the teeth). The excavation shall continue until the Engineer approves the exposed "competent" bedrock surfaces.

.10 Cleaning: The bedrock surface shall be cleaned so that concrete will bond to the rock. This generally requires jetting with air and water. As required by Sections 1000 and 1010, no sediment shall be allowed to migrate to any waterways. No fill shall be placed on cleaned bedrock surfaces until the cleaning has been inspected and approved by the Engineer.

.11 Following the initial cleaning, the Engineer will inspect and geologically map the exposed bedrock surfaces, and will specify which areas require slush grout surface treatment, placement of dental concrete or bedrock excavation by blasting or other mechanical means.

.12 Slush Grout Surface Treatment: Thin surface cracks and fractures in the bedrock subgrade shall be filled with slush grout as directed by the Engineer.

.13 Slush grout shall consist of 5 parts cement and 1 part sand (by volume) thoroughly mixed with sufficient water to produce a free flowing mixture which can be poured and broomed into cracks and fractures in the bedrock subgrade and which will completely fill open joints, crevices and minor imperfections in the bedrock subgrade.

.14 The actual water-cement ratio to be used shall be determined and approved by the Engineer on the basis of trial mixes produced with the mixing and pumping equipment proposed by the Contractor. In any event, the water to cement ratio shall not exceed 0.6:1 (by weight) unless otherwise directed by the Engineer.

.15 Cement shall be sulphate resistant Portland Cement meeting the requirements of ASTM C-150, Type V or CSA (CAN A23.1) Type 50 and contained in standard 40 kg (88 pound) water-repellent type bags. Use of bulk cement is not permitted. Cement shall be less than three months old at the time of use in the grout. Cement that has become partially hydrated or contains foreign matter shall not be used in grout.

.16 Sand shall be a clean medium to fine Mortar Sand of a gradation approved by the Engineer.

.17 Water and sand shall meet the requirements set out in CSA CAN3-A23.1. Grout shall be mixed in a mechanical mixer for at least 5 minutes and shall be applied to the rock surface and broomed into cracks and fractures within 40 minutes of mixing.

.18 Slush grout shall not be placed when the air or rock temperature is below 0°C (32°F). All bedrock surfaces receiving slush grout treatment shall be wetted prior to slush grouting.

.19 No traffic shall be allowed to travel over slush grouted areas until the grout has fully set.

4.0 INSTALLATION OF DOWELS

.1 Concrete structures bearing directly on bedrock shall be dowelled into competent bedrock as shown on the drawings or as directed by the Engineer. This includes the cutoff wall foundation trench.

.2 Cement grout used for grouting steel rock dowels to be a fast-setting, high strength non-shrink cement grout such as QUIKRETE Non-shrink Precision Grout # 1585-00 or approved equivalent. Grout shall be mixed and handled according to the manufacturers recommendations.

.3 Dowels shall comprise epoxy coated 19M deformed steel reinforcing bars complying with Specification No. 1400.

.4 Grout shall have a water/solid ratio of 0.16 - 0.19.

.5 Grout shall have a minimum 28 days compressive strength of 43 MPa when tested as mortar cubes according to ASTM C109-77, U.S. Corps CRD-C227.

.6 Steel dowels shall be installed in drill holes having a minimum diameter of 37.5 mm (1.5 inches) in accordance with the layout shown on the drawings.

.7 All dowels shall be installed centrally in the drill holes and grouted into place with approved cement grout at least 48 hours prior to commencement of formwork for the walls.

.8 Depth of drill shall be no more than 100 mm (4 inches) deeper than the length of the dowel to be installed.

.9 The drill holes shall be flushed of all drill cuttings, sludge and debris with compressed air prior to the installation of the steel dowels.

.10 The holes shall be drilled at an angle of no less than 5º below the horizontal.

1.0 GENERAL

.1 This section covers hauling, placing and compacting fill materials for construction of the works. Works shall be constructed as shown on the drawings.

.2 Tolerances: Dimensions shall not be less, nor the slopes steeper, than those specified. Dimensional tolerances of all zones shall be the following unless specifically otherwise noted:

- Level tolerance -0 to +0.03 m for the spillway rip rap surface
  Level tolerance -0 to +0.15 m for all other earthworks
  Level tolerance -0.01 to +0.01 m for top of the cutoff wall
- Horizontal tolerance -0 to +0.5 ft for all works

.3 The Contractor shall provide and have available at all times during his working hours, the necessary staff and equipment to ensure the proper and correct setting out of the Works. Should any errors in setting out of the Works occur, such errors shall be corrected and any necessary adjustments to previously placed fill materials made to the satisfaction of the Engineer prior to further placement of fill materials.

.4 Methods of carrying out the fill placement shall be subject to approval by the Engineer. Equipment suitability, method of working, rate of progress and quality of work shall be demonstrated during the initial stages of the work. In the event that the work performance is unsatisfactory, the Contractor shall immediately implement such changes as are required to ensure the proper completion of the work.

2.0 FILL PLACEMENT PROCEDURES

.1 Commencement: Placement and compaction of fill materials shall generally commence in the lowest areas of the work. No fill shall be placed until the Engineer has approved the foundation preparation.

.2 Drainage: Placement and compaction of the fill shall be done in near horizontal lifts, and the fill surface shall be sloped to provide drainage during construction. Water shall not be allowed to pond on the fill surface.

.3 Obstacles: Fill placed within 1.5 m of obstacles, including the cutoff wall, shall be kept one lift higher than the surrounding area. Such fill shall be compacted with equipment which is suitable for working in a confined space (and, if necessary, placed by hand using a lift thickness of one half the normal thickness). Particular care shall be taken to ensure that there is no particle segregation near the contact between fill and adjacent concrete or bedrock surfaces.

.4 Filter Zones: Fill in Zone 3 shall be protected from contamination from adjacent zones.

.5 Frozen material: Frozen fill shall not be placed, nor shall fill be placed on frozen surfaces. Frozen fill shall be removed prior to the placement of additional material.

.6 Segregation: Material that has segregated during transportation or placement shall be mixed prior to compaction.

.7 Gradation: The fill shall be free from lenses, pockets or layers of material which are significantly different in gradation from the surrounding material of the same zone. Material placed which does not meet the specified gradation requirements shall be removed or otherwise reworked to produce material which does.

.8 Zone Compatibility: Where zone requirements allow large gradational variations in fill materials within the zone, materials shall be placed so as to prevent the migration of particles from finer materials into voids in coarser materials. Where the gradation of two adjacent materials are incompatible and the danger of particle movement exists, a minimum 0.3 m. layer of transitional material, with an approved intermediate gradation, shall be placed between the fill materials in question.

.9 Oversize: Oversize particles shall be removed. Where reduced lift thicknesses are required, particles larger than 2/3 of the reduced lift thickness shall be removed prior to compaction. Oversize materials which are removed shall be disposed of in a manner approved by the Engineer.

.10 Suspension of Operations: Before the suspension of operations each day, or before inclement weather, the fill in place shall be compacted and the surface rolled smooth and crowned to facilitate runoff of precipitation.

.11 Construction traffic: Any damage to the spillway, or to the fill already placed, due to construction traffic, shall be repaired prior to the placement of the next lift. This may include, but shall not be limited to, the removal of ruts, contaminated fill materials and repairs to fill boundaries.

.12 Lift Thickness: The maximum loose lift thickness, before compaction, for subgrade materials or shall generally not exceed 0.3 m.; however thinner lifts may be required depending on the particular material, compaction equipment utilized and weather factors. The applicable lift thickness for each case will be defined by the Engineer in the field. Fill placed within 1.5 m. of any obstacles, including the cutoff and bedrock abutments shall be placed in lifts one half of the normal specified loose lift thickness.

.13 Water Content and Compacted Density: Zone 4 Subgrade Fill shall be compacted, where possible, at a water content within the range given below, and it shall be compacted to a dry density exceeding the minimum value given below:

Zone	Compaction Water Content Range (%)*	Minimum Dry Density (%)**

1.	Rip Rap	N.S.	N.S.
1A.	Fine Rip Rap	N.S.	N.S.
2.	Bedding	N.S.	N.S.
3.	Filter	N.S.	N.S.
4.	Subgrade Fill	-3.0% to +0.5%	93%
5.	General Fill (Waste Rock)	N.S.	N.S.

NOTES: * Water content range is relative to the standard Proctor optimum water content determined according to ASTM D698-78.

** The minimum dry density is stated as a percentage of the Standard Proctor corrected maximum dry density determined according to ASTM D698-78.

N.S. - no specification - no compaction required.

.14 The Contractor shall adjust and maintain the fill materials at the specified water content for compaction in a manner which shall ensure uniform moisture distribution throughout each lift thickness.

.15 Compaction Method: Material for Zone 4 Subgrade Fill shall be compacted with a heavy, self propelled, smooth drum, vibratory roller.

.16 Compaction shall be as uniform as practicable over the entire lift surface.

.17 The compaction equipment shall not travel at speeds exceeding 4 km/hr, and a minimum 1.0 ft. overlap between adjacent passes of the compactor shall be provided.

.18 Compacted fill which has suffered a reduction in density due to frost action, precipitation, or for any other reason, or which has cracked due to drying, shall be reworked or replaced to the full depth of damage prior to placing the next lift. If the surface of a compacted lift is too dry or too smooth to bond properly with the next lift of material to be placed, the fill surface shall be scarified to a minimum depth of 100 mm and water shall be added as necessary. The scarified surface shall then be recompacted to provide a satisfactory bonding surface before the next lift is placed.

.19 Fill Boundaries: The maximum difference in elevation between adjacent compacted surfaces within the fill shall be one lift thickness. Temporary slopes within the fill are undesirable and shall be avoided. Prior to the placement and compaction of fill materials against a temporary slope, the surface of the slope shall be cut back to expose dense material. New material shall be placed and compacted against temporary slopes in the same manner as specified for placing fill close to obstacles.

1.0 GENERAL

This section provides the gradational limits and other properties for the various fill materials which are to be incorporated in the work.

Identification of Material Types: Fill materials are identified on the Construction Drawings by a Zone or Material Type number 1B, 1A, 2, & 3A respectively.

2.0 GENERAL REQUIREMENTS

General Requirements: Fill materials shall be free of deleterious substances, organics, peat and other unsuitable matter, and they shall not be frozen at the time of placement.

Fill materials generally shall be well graded within the specified gradation limits. Fill materials shall be free of segregation.

Where excavated material meets the fill specifications for a particular zone (specification 0500), the material may be reused as fill, subject to the Engineer's approval.

Safe and stable side slopes shall be maintained as required in accordance with the requirements of applicable legislation and drawing.

Source Approval: Sources of materials (i.e. selection of borrow pits or waste rock sources, including individual source areas within borrow pits or waste rock sources), require the approval of the Engineer. Allow at least 4 working days for inspection, sampling, testing and approval of proposed material sources before commencing fill placement.

Rejection of Materials at the Construction Sites: The Engineer will examine fill materials hauled to the construction sites and may sample and test those materials to check that they meet these specifications and that they originate from an approved source. Fill materials hauled to the sites may be rejected if they do not meet these specifications, whether or not they have actually been placed. Acceptance of sources of materials shall not preclude such rejection of materials at the construction sites.

Wasting Rejected Materials: Fill materials which have been rejected shall be disposed of in a location and manner approved by the Engineer.

3.0 MATERIAL TYPE 1B – COARSE RIP RAP

1. Material Type 1B will generally be used on the surface of the outlet downstream of the cutoff wall to resist erosion.

2. Material Type 1B may consist of selected and/or screened boulders with an average particle size D50 of 350mm minimum.

3. Alternatively, Material Type 1B may consist of selected and/or screened or crushed coarse mine waste rock, providing the waste rock has been classified as non acid generating when tested according to Price (1997).

4. Material Type 1B shall consist of solid pieces which will not disintegrate when exposed to wet or moist environments.

5. Material Type 1B shall consist of boulders which are reasonably well graded within the following limits:

   (a) Particles shall not exceed a maximum diameter of 22 inches (570 mm).

   (b) Not more than 40% of particles by weight shall exceed a mean diameter of 18 inches (450 mm).

   (c) 50 to 70% of particles by weight shall exceed a mean diameter of 14 inches (350 mm).

   (d) 15 to 30% of particles by weight shall have a diameter of 9 inches (240 mm) or less.

4.0 MATERIAL TYPE 1A – FINE RIP RAP

1. Material Type 1A will generally be used on the surface of the outlet upstream of the cutoff wall to resist erosion.

2. Material Type 1A may consist of selected and/or screened cobbles and boulders, with an average particle size D50 of 200mm minimum.

3. Alternatively, Material Type 1A may consist of selected and/or screened or crushed coarse mine waste rock, providing the waste rock has been classified as non acid generating when tested according to Price (1997).

4. Material Type 1A shall consist of solid pieces which will not disintegrate when exposed to wet or moist environments.

5. Material Type 1A shall consist of rip rap which is reasonably well graded within the following limits:

   (a) Particles shall not exceed a maximum diameter of 18 inches (450mm).

   (b) Not more than 40% of particles by weight shall exceed a mean diameter of 12 inches (300 mm).

   (c) 50 to 70% of particles by weight shall exceed a mean diameter of 8 inches (200 mm).

   (d) 15 to 30% of particles by weight shall have a diameter of 6 inches (150 mm) or less.

5.0 MATERIAL TYPE 2 – BEDDING

1. Material Type 2 will generally used as bedding area as part of the erosion protection system between Material Types 1 or 2 (Rip Rap) and Material Type 3A (Filter) as a transition as designated on the Construction Drawings.

2. Material Type 2 may consist of selected and/or screened cobbles and boulders, with an average particle size D50 of 150 mm minimum.

3. Alternatively, Material Type 2 may consist of selected and/or screened or crushed mine waste rock, providing the waste rock has been classified as non acid generating when tested according to Price (1997).

4. Material Type 2 shall consist of solid pieces which will not disintegrate when exposed to wet or moist environments.

5. Material Type 2 shall consist of rip rap bedding which is reasonably well graded within the following limits:

   (a) Particles shall not exceed a maximum diameter of 12 inches (300 mm).

   (b) Not more than 40% of particles by weight shall exceed a mean diameter of 9 inches (230 mm).

   (c) 50 to 70% of particles by weight shall exceed a mean diameter of 6 inches (150 mm).

   (d) 15 to 30% of particles by weight shall have a diameter of 3 inches (75mm) or less.

6.0 MATERIAL TYPE 3A – FILTER

1. Material Type 3A will be used as a granular filter between sub-grade soil and Zone 2 bedding material, where required.

2. Material Type 3A shall be free of organics or other deleterious material.

3. Material Type 3A shall consist of sand and gravel which is reasonably well graded within the following gradation limits:

U.S. Standard Sieve Size	Per Cent Finer Than
150mm (6 ")	100
75mm (3 ")	100
50mm (2 ")	100
25.4mm (1 ")	82 – 100
19mm (3/4 ")	78 – 100
12.5mm (1/2")	63 – 85
No. 4 (4.75 mm)	50 – 75
No. 10 (2.00mm)	40 – 65
No. 20 (0.850mm)	28 – 56
No. 40 (0.425mm)	18 – 40
No. 100 (0.150mm)	4 – 18
No. 200 (0.075 mm)	0 – 8

1.0 ANTICIPATED SOURCES OF FILL MATERIALS

.1 The following table indicates the anticipated sources of earth fill for the various material types. Additional information on the anticipated sources will be provided by the Owner.

ANTICIPATED SOURCE OF EARTH FILL

Material No.	Proposed Use	Material Type	Anticipated Source of Material
1B	Coarse Rip Rap	Boulders D50 size= 350mm Min	Mine Site.
1A	Fine Rip Rap	Boulders and Cobbles D50 size= 200mm Min	Mine Site.
2	Bedding	Select Pit Run or Screened Cobbles mixed with Gravels D50 size= 150mm Min	Mine Site (To be screened from selected materials).
3A	Filter	Selected screened Sand and Gravel	Mine Site (To be screened from selected materials).

.2 Mine waste rock piles and borrow pits shall be made available for the use of the Contractor without royalty charges.

.3 The Contractor may decide to obtain borrow for the various material types from sources other than those indicated on the foregoing table. In the case of sources not listed above, the Contractor shall be responsible for any royalties payable. As indicated in Section 0500, all potential borrow sources require approval by the Engineer.

2.0 MANAGEMENT OF BORROW SOURCES

.1 The Contractor shall be responsible for the efficient management of all borrow sources.

.2 Borrow sources shall be used in such a way as to prevent suitable borrow being rendered unsuitable due to flooding, freezing, equipment traffic, or contamination with organic or other deleterious material.

.3 Once the Contractor has finished using the borrow sources, the base of the pits shall be levelled and graded so that they will not pond water and all slopes shall be flattened to a stable angle as directed by the Owner.

.4 No separate payment will be made for the development or management of borrow sources. The Contractor shall include the costs of obtaining; processing, hauling and placing borrow material as fill in the unit costs.

1.0 SURVEYS PROVIDED BY THE OWNER

.1 A topographic survey of the area of the West Twin Outlet area was completed by NorthTech Consultants on August 27, 2004. This survey shall be taken as representing pre-constructions conditions.

.2 The Owner will arrange for an additional pre-construction survey to be undertaken in the area of the spillway discharge chute. When completed, this survey shall be taken as representing the pre-construction conditions.

.3 The Owner will provide surveying to establish the following:

- Reference elevation bench marks near the West Twin Outlet area.
- Reference hubs indicating the horizontal location of the centreline of the West Twin Outlet.

.4 The Owner may, at his option, conduct surveys to check pay quantities, including:

- topographic survey of borrow pits,
- elevations of prepared spillway foundations, and
- cross-section surveys of completed work.

2.0 COOPERATION WITH THE OWNER’S SURVEYOR

.1 For the duration of the contract the Contractor will:

- Provide the Owner with reasonable assistance which may be required at any time in checking the work.
- Protect and preserve all bench marks, reference points and other survey marks established by the Owner.
- Inform the Owner immediately if any bench mark or reference point established by the Owner has been disturbed or damaged, and pay for the repair or replacement of it.
- If it becomes necessary to remove any survey points established by the Owner, the Owner shall be notified at least 3 days in advance. If such removal is deemed necessary, the Owner will arrange for its re-establishment at the Owner’s expense.

3.0 SURVEYING TO BE PROVIDED BY THE CONTRACTOR

.1 The Contractor shall be responsible for providing all surveys necessary, aside from those provided by the Owner, listed in Section 1. Surveys provided by the Contractor shall include, but not be limited to the following:

- slope stakes indicating the horizontal limits of the outlet excavation and fill zones, and
- alignment and grade stakes for the outlet channel.

These surveys will be conducted notwithstanding any check surveys of pay quantities carried out by the Owner.

1.0 GENERAL

.1 The Contractor shall provide all equipment, material, and labour necessary for draining and dewatering all work areas of both surface and subsurface water.

.2 Maintain suitable dry working conditions throughout construction by providing one or more of: effective surface drainage, temporary cofferdams, pumping or other construction dewatering.

.3 To complete the new construction at the entrance to the spillway as shown on the Construction Drawing, the Contractor shall construct a new temporary cofferdam. The Contractor shall submit to the Engineer for approval, a plan detailing the design of the new coffer dam to be provided.

.4 Construction dewatering of the work sites shall be conducted in a manner that limits total suspended solids released into Twin Lakes Creek to 30 ppm or lower. No water shall be discharged downstream of the existing beaver dam. Other measures to achieve this could include the use of settling ponds, silt bags, silt fences, hay bale barriers, etc.

1.0 GENERAL

This section describes the material standards and placement requirements for all concrete works shown on the Construction Drawing. This section also covers the supply and installation of cast-in-place concrete, including the following: formwork, reinforcing, joints, concrete placing, assistance to the Engineer, curing and protection, finishing, encasements and concrete fill, grouting of surface defects in concrete, casting-in of items supplied by others, design of formwork, rebar bending schedules and placing drawings, and clean-up.

Specifications regarding the supply and setting of dowels are provided in Specification 300.

2.0 QUALITY ASSURANCE

All work shall conform to the requirements of the latest edition of the following:

i) Canadian Standards Association (CSA) standards:

CAN/CSA-A5-88	Portland Cement
CAN3-A23.1-M90	Concrete Materials and Methods of Concrete Construction
CAN3-A23.2-M90	Method of Test for Concrete
CAN3-A266.2-M78	Air-Entraining Admixtures for Concrete
CAN3-A266.2-M78	Chemical Admixtures for Concrete
CAN3-A266.4-M78	Guidelines for the Use of Admixtures in Concrete
G30-12-M1977	Billet-Steel Bars for Concrete Reinforced Concrete Construction

ii) American Concrete Institute (ACI) standards:

315-80 (86)	Manual of Standard Practice of Detailing Reinforced Concrete
347-R88	Guide to Formwork for Concrete
302-1R-89	Guide for Concrete Floor and Slab Construction
305R-91	Recommended Practice for Hot Weather Concreting
306	Recommended Practice for Cold Weather
308	Recommended Practice for Curing Concrete

3.0 MATERIAL REQUIREMENTS

.1 All concrete, formwork and reinforcing steel be in accordance with CSA Standard CAN33-A23.1-M90, “Concrete Materials and Methods of Concrete Construction” and “Method of Test for Concrete”.

.2 The Contractor shall submit his concrete mix design to the Engineer prior to batching and the mix shall comply with the following:

Structural Concrete

i) Max. aggregate size = 19 mm (0.75 inch)
ii) Slump at point of delivery = 75 to 100 mm (3 to 6 inches)
iii) Air content = 5 to 8 per cent

iv) Maximum water:cement ratio = 0.5 by weight
v) Cement shall be CSA Type 50 Portland Cement, CSA Standard A5-M83
vi) Min. cement content 295 kg/cu.m. of concrete
vii) Min. compressive strength = 35 MPa at 28 days

.3 All reinforcing steel bars shall be deformed bars conforming to G30.12-M-1977 and shall have a minimum tensile strength of 400 MPa (58ksi).

.4 All reinforcing bars shall be epoxy coated.

.5 Admixtures shall comply with CAN3-A266.1 & A266.2. The use of admixtures shall be to the approval of the Owner's Representative and in accordance with CAN3-A266.4.

.6 Construction joints in the cutoff wall shall incorporate a CPD PVS Type 4 water stop (ribbed type with centre bulb, 150 mm width) or approved equivalent meeting the following specifications:

Water Stop Specifications[1]

Property	**Value**	**Test Method**
Tensile Strength	2000 psi min.	ASTM D-412
Ultimate Elongation	370% min.	ASTM D-412
Hardness Shore A	80 ± 3	ASTM D-2240
Stiffness in Flexure	700 psi min.	ASTM D-747
Water Absorption	0.5 max (48 hrs.)	ASTM D-570
Low Temp Brittleness	-50 Degrees F	ASTM D-746
Accelerated Aging		CRD C-572
Tensile Strength	1500 psi min	
Elongation	300% min	
Effect of Alkalies		CRD C-572
Weight change	-0.1 + 0.25	
Hardness change	+/-5	
Tensile change	15% max	

Note 1: Meeting or exceed the requirements and performance criteria of Corps of Engineers Specifications CRD-C572-74.

.7 Materials shall be stored in such a manner as to prevent deterioration or intrusion of foreign matter. Deteriorated or contaminated materials shall not be used for concrete.

.8 Reinforcing steel shall be supported on sleepers. It shall be grouped as to destination or size, and bars shall be identified with metal tags, in accordance with bar lists.

4.0 FABRICATION AND PLACEMENT

.1 Fabricate and bend reinforcing steel in accordance with CSA CAN A23.3-M-1977 to the dimensions shown on the construction drawing. Do not bend or straighten bars in any manner which will damage the steel or reduce the cross-section. Do not use bars with kinks or sharp bends.

.2 The minimum length of laps on bars shall be 24 times the diameter of the largest bar.

.3 The source and methods of mixing, delivery and placement of concrete shall be subject to approval. All reinforced concrete shall be vibrated with a proper concrete vibrator. Vibrators shall not be used to cause the concrete to flow horizontally or on slopes.

.4 Concrete shall be rejected if more than 60 minutes has elapsed from the time of initial introduction of mixing water to the batch.

.5 Placed concrete shall not be subjected to excessive vibrations between the time of initial set and 5 days thereafter, nor shall formwork be removed within that period.

.6 All concrete shall be properly cured in accordance with CAN3-A23.1-M90 and ACI 308. Immediately after placement, the concrete shall be protected from premature drying, excessively hot or cold temperatures, and mechanical injury. All concrete shall be maintained with minimum moisture loss at relatively constant temperature for a period of not less than 3 days. In hot weather, concrete shall be cured in accordance with one of the methods described in CAN3-A23.1-M90 and ACI 305.

.7 Concrete shall be prevented from freezing for a minimum of 5 days after pouring. Use of Calcium Chloride as an additive to prevent freezing is prohibited.

.8 The work area shall be maintained in a neat condition and all waste and extraneous materials shall be removed as they accumulate.

5.0 INSPECTION, TESTING AND REMEDIATION

.1 Inspection and Testing: Inspection services will be provided by the Engineer (i.e. the Owner's Representative). The use of such services does not relieve the Contractor of his responsibility to furnish work in compliance with the Contract Documents.

The Contractor shall advise the Engineer at least 24 hours in advance of concrete pouring operations to allow for the desired quality tests and for assignment of personnel. The Contractor shall leave one face of wall forms open until reinforcement has been inspected and approved by the Engineer. Any other Contractors incorporating materials in a pour shall also be notified at least 24 hours before placement so that they can inspect the Works.

.2 The appropriate strength requirement of the concrete shall be considered to be satisfied if for each set of three cylinders tested at 28 days, both the following conditions are met:

1) None of the strength of three cylinders is below 80% of the specified minimum strength.

2) The average strength of the cylinders is greater than the specified minimum strength.

.3 In the event of the test cylinders failing to conform to the specified standards, the portion of the works represented by the cylinders shall be liable for rejection or alternatively, the Contractor may, subject to the approval of the Engineer, propose other remedial measures to rectify the shortcoming and sub-standard works. All works ordered under this sub-clause shall be at the Contractor's own expense.

APPENDIX IV
TWIN LAKES CREEK
BACKWATER ASSESSMENT
(GOLDER 2005a)

TECHNICAL MEMORANDUM

Golder Associates Ltd.
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TO:	BGC Engineering Inc., Attention: Mr. Geoff Claypool	**DATE:**	July 22, 2005
FROM:	Alex Gordine, Luis Vasquez, Ken Bocking	**JOB NO:**	05-1118-011
EMAIL:	agordine@golder.com		
RE:	**BACKWATER ASSESSMENT OF WEST TWIN DISPOSAL AREA NANISIVIK MINE**		

This Technical Memorandum presents an assessment of a possible backwater condition at the outlet of the West Twin Disposal Area (WTDA) Polishing Pond at the Nanisivik Tailings Management Area.

Background

Water draining from the Reservoir through the outlet channel and over the weir will join with water flowing from East Twin Lake and flow into the Twin Lake Creek (Figure 1).

The WTDA Closure Plan (BGC *et al*, 2004) suggested that

- The stop log structure between the Reservoir and the Polishing Pond will be replaced with a 30 m wide open channel. Therefore, the Reservoir and the Polishing Pond will be combined into a single water body.
- The Polishing Pond outlet control structure will be replaced with an outlet channel and a 7 m wide overflow weir.
- The weir invert was set at an elevation of 370.2 m. This elevation was suggested based on the following considerations:
    - It corresponds roughly to the West Twin Lake water level under the natural, pre-development conditions.
    - It is approximately 0.2 m higher than the water level in Twin Lakes Creek, which receives the Reservoir effluent. (Using available maps, the normal water level in Twin Lakes Creek downstream of the proposed weir was estimated to be approximately 370.0 m.)

A concern was expressed that, during high floods, Twin Lake Creek could create backwater at the WTDA Polishing Pond outlet. The purpose of this memo is to address this issue.

Surveyed Area

A surveyed profile of Twin Lake Creek (Figure 1) was provided by BGC. The surveyed section starts approximately 220 m downstream of the outlet weir and extends approximately 400 m in the upstream direction (Figure 1). Figure 2 shows the Twin Lake Creek profile. The following notes were made from the survey data:

- Based on the survey, the creek bottom slope downstream of the outlet channel ranges from 1% to 2%.
- The creek bottom elevation is higher than that inferred previously from the maps. The Twin Lakes Creek bottom elevation at Cross-section P9 is 369.74 m (Figure 1). Therefore, the creek bottom is approximately 0.44 m below the proposed weir invert (370.2 m).
- Assuming a typical water depth in Twin Lakes Creek of 0.2 m, the water level differential between the Reservoir and the creek will be approximately 0.2 m to 0.25 m.
- No cross-sections of Twin Lakes Creek were surveyed. It was therefore necessary to assume the geometry of a flow constricting section of the creek for use in the hydraulic calculations. This geometry was estimated from the topographic mapping. The uncertainty associated with this approach is noted.

Hydraulic Analysis

Peak water levels in the Twin Lakes Creek were estimated for the same design precipitation events that were used in the previous Reservoir hydraulic assessment (Golder, 2004a): (i.e. a 100-yr event, a 500-yr event, and a PMP event).

The peak water levels in the Twin Lakes Creek were calculated and then compared to the corresponding peak water levels in the Reservoir, which were reported previously (Golder, 2004b).

A constricting cross-section in Twin Lakes Creek was selected approximately 90 m downstream of the Polishing Pond weir (Figure 1). The cross-section is approximately 20 m wide. Peak water level in the constricting section was calculated using two approaches:

- Using Manning's Equation (based on the assumed constricting cross-section geometry, on the surveyed longitudinal slope of 1%, and using a Manning's n value of 0.03; and
- Using the broad crested weir equation (i.e. assuming that the constricting section of West Twin Creek acts as a weir).

The following table summarize the design precipitation events, the flows after the confluence of the East Twin Lakes Creek flows and the WTDA effluent, the corresponding peak water levels on the Reservoir and the estimated water levels in the cross-section studied at Twin Lakes Creek.

Design Precipitation Events (1)	**Peak Water Level in Reservoir/Polishing Pond** (1)	**Design flood conditions in Twin Lakes Creek at Reservoir Outlet**		
		Design Flow[1]	**Peak Water Level (Manning's Equation)**	**Peak Water Level (Weir Equation)**
-	(m)	(m^3/s)	(m)	(m)
100 year return period	370.36	15	370.15	370.3
500 year return period	370.4	19	370.22	370.4
PMP	370.8	55	370.66	371.08

(1) Source: Golder, "*Addendum to the report on extended hydrological study, Nanisivik mine closure*". Technical Memorandum, March 2004.

The following observations could be made from the Table:

- Peak water levels calculated using the weir equation at the constricting cross-section are higher than those calculated using the Manning's equation. Therefore, using the weir equation in the analysis is a safer, more conservative approach.

- For 100-yr and 500-yr storm events, water levels in Twin Lake Creek are slightly lower than the corresponding peak water levels in the Reservoir. Therefore, there will be no reverse flow from the creek into the Reservoir.

- The calculations suggest that, under the PMP event, the peak water level in Twin Lakes Creek would be higher than that in the Reservoir. The calculated PMP water level, however, is higher than the topography of the ground adjacent to the creek. (The topographic mapping shows a highest elevation of 370.5 m on the left bank; there are no survey data on the right bank, Figure 1). Therefore, the assumption of Cross-section P9 constricting the channel and acting as a weir at such high water levels is invalid. Although the PMP flow conditions are hard to predict accurately based on the limited survey data, it is likely that the creek would overflow its banks, causing extensive flooding, but also that the creek water levels would not be sufficiently high to reverse the flow direction into the Reservoir.

CONCLUSION

Based on the available topographic information and hydraulic calculations, Twin Lakes Creek has sufficient hydraulic capacity to convey flood events without reversing the flow direction from the creek into the Reservoir. Some backwater at the outlet of the reservoir is likely to take place under extreme flood conditions.

AG/LV/KB/dh

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Figure 1
TWIN LAKES CREEK
Channel Bottom Profile

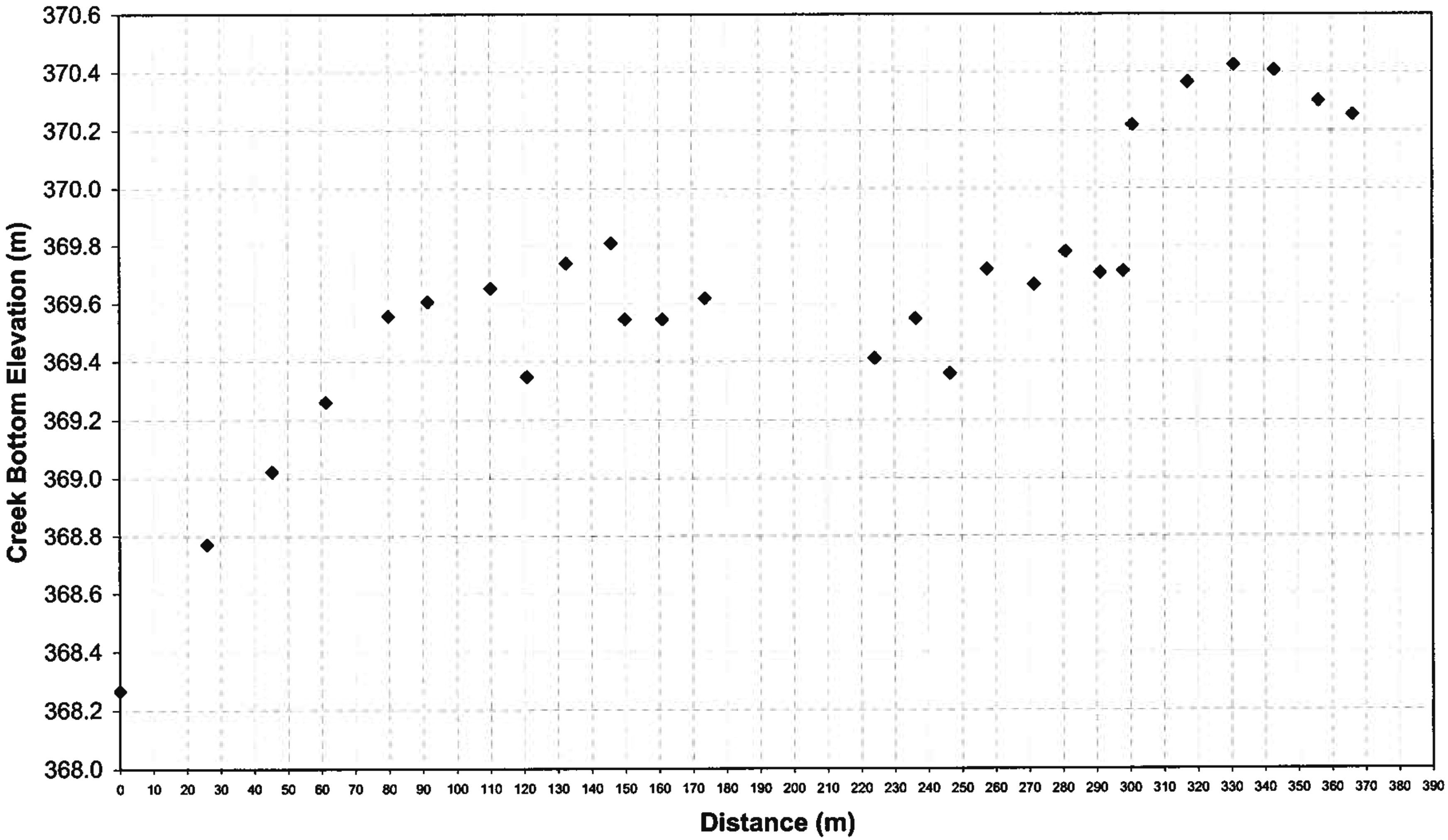
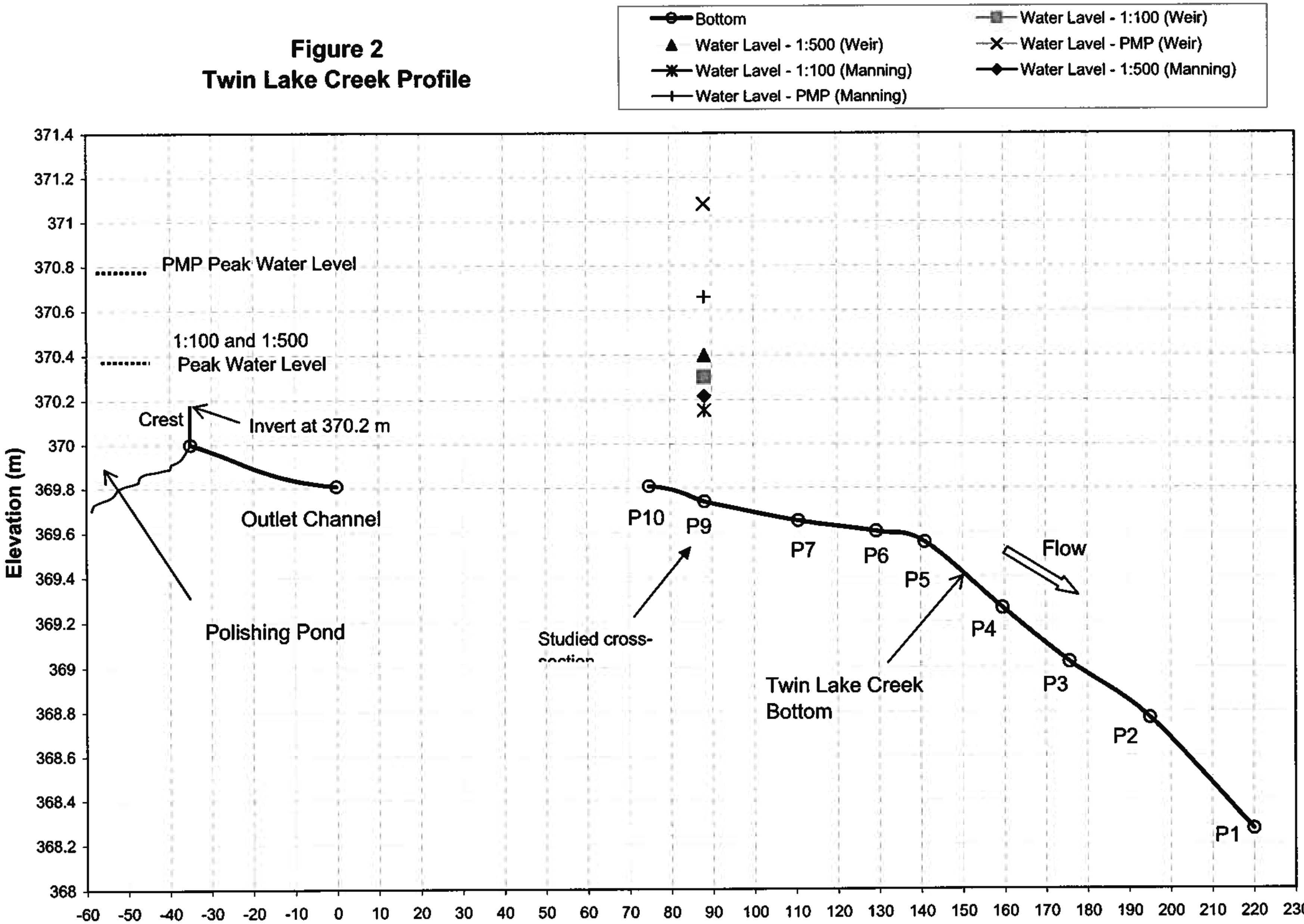


Figure 2
Twin Lake Creek Profile



Golder Associates

APPENDIX V
CONSTRUCTION PHOTOS
EAST TWIN CREEK DIVERSION CHANNEL

Photo 1 - July 2004
East Twin Diversion Channel prior to reclamation.
Note area of erosion on face of dike.

Photo 2 - July 2004
East Twin Diversion Channel prior to reclamation.
Close up view of area of erosion on face of dike.

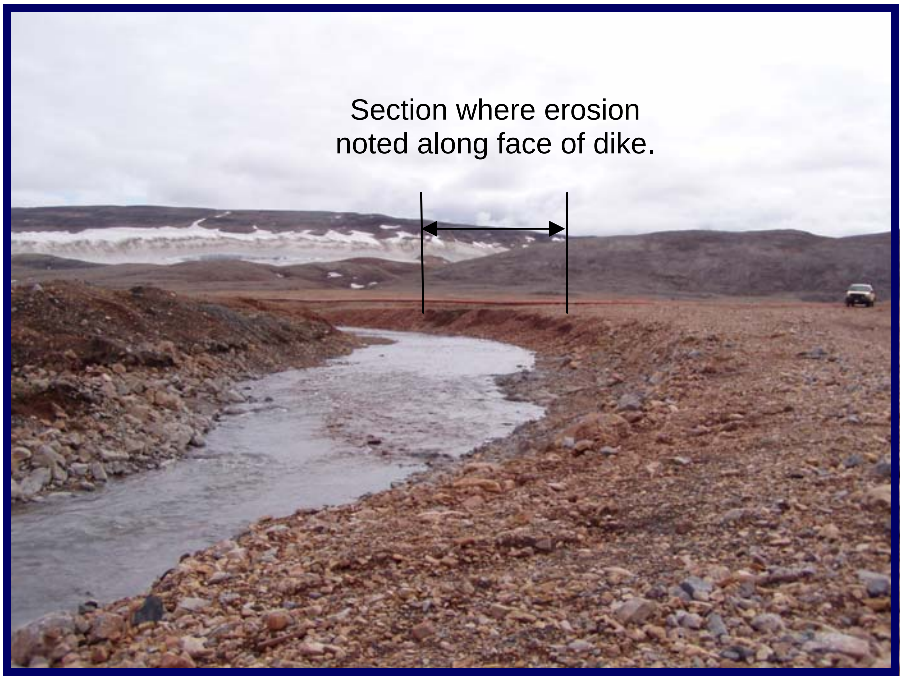


Photo 3 - July 2004
View looking upstream.
Note area of erosion on dike.

Photo 4 - September 2005
Regraded portion of East Twin Creek Diversion Channel Dike.

Photo 5 - August 2006
View looking upstream.
Armoured section of diversion dike.
Note grain size distribution of rockfill.

Photo 6 - August 2006
View looking downstream.
Armoured section of diversion dike.