APPENDIX X-4

NCRP Final Closure Design

29 March 2017

REPORT ON

Diavik Diamond Mine North Country Rock Pile Closure Design

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REPORT

Golder Reference Number: 1521339-1471-R-Rev3-7000

Diavik PO Number: D03279

Work Plan Number: WP 473 Rev.0

Distribution:

1 Electronic Copy - Diavik Diamond Mines (2012) Inc.
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Executive Summary

Golder Associates Ltd. (Golder) was retained by Diavik Diamond Mines (2012) Inc. (DDMI) to prepare a construction package for the closure of the North Country Rock Pile (NCRP) at the Diavik Diamond Mine in the Northwest Territories.

The NCRP provides permanent storage for run-of-mine waste rock generated from the development and operation of open pits and underground mines at the Diavik Diamond Mine. The NCRP is approximately 60 m high and covers an area of approximately 1,750,000 m^2. The site is in the zone of continuous permafrost. The closure design includes re-sloping and covering the areas of the NCRP that contain potentially acid generating materials. This report presents a design for construction of the closure of the NCRP including an assessment of the physical stability of the closure cover, material quantities, drawings and technical specifications.

Areas of the NCRP containing waste rock with the potential for acid generation are re-sloped from angle of repose slopes to 3 horizontal to 1 vertical slopes and covered with a minimum 1.5 m (metres) of till and 3 m of non-acid generating (Type I) waste rock. The cover layer thicknesses and materials were designed by others and provided by DDMI as an input to the design presented here. The till layer function is to reduce infiltration and promote permafrost within the underlying waste rock. The till layer is designed as a permanently frozen layer (or permafrost aggradation layer), rather than as a low permeability layer. The 3 m waste rock capping layer serves as erosion and thermal protection over the till. Studies of thermal effects, geochemistry, seepage, surface water management, and environmental components of the NCRP closure design are referenced where available, and considered to be outside the scope of this report.

Areas of the NCRP where the waste rock is considered to have no acid generation potential do not require placement of the cover system. In these areas, the NCRP side slopes will remain at angle of repose and with the existing configuration of benches.

The estimated re-slope and construction materials required for the NCRP closure are:

- re-slope area: 835,000 m^2;
- Till Cover: 2,000,000 m^3; and
- Type I waste rock: 3,980,000 m^3.

The design allows for progressive closure during mine operations, with run-of-mine materials generated from the development of the A21 kimberlite pipe used for cover construction.

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1.0 INTRODUCTION

Diavik Diamond Mines (2012) Inc. (DDMI) retained Golder Associates Ltd. (Golder) to prepare a construction package for the closure of the North Country Rock Pile (NCRP) at the Diavik Diamond Mine in the Northwest Territories (NT).

The closure includes re-sloping and covering the areas of the NCRP that contain potentially acid generating materials. The design allows for progressive closure during mine operations, with run-of-mine (ROM) materials generated from the development of the A21 kimberlite pipe used for cover construction.

This report presents a design for closure of the NCRP. The scope of the design includes re-sloping the existing waste rock and placement of a cover, assessment of the physical stability of the closure cover, with quantities, drawings and technical specifications. The cover design geometry was adopted at the direction of DDMI and includes a minimum 1.5 m of till and 3.0 m of non-acid generating waste rock. The scope of the design excludes studies for thermal effects, geochemistry, seepage, surface water management, and environmental components of the NCRP closure design, though work by others is summarized where available.

This report should be read in conjunction with the Study Limitations which follow the text and form an integral part of this document.

1.1 North Country Rock Pile

The Diavik Diamond Mine is located on East Island, a 17 km^2 island in Lac de Gras, approximately 300 km northeast of Yellowknife in the NT. The site is located above the tree line in an isolated area with no permanent access roads. Figure 1 presents the site location plan.

The NCRP provides permanent storage for ROM waste rock generated from the development and operation of the open pits and underground mines at the Diavik Diamond Mine. The NCRP is approximately 60 m high and covers an area of approximately 1,750,000 m^2 on the northern part of East Island. The south side of the NCRP forms the downstream rockfill shell portion of North Dam of the Processed Kimberlite Containment (PKC) Facility.

The NCRP was originally designed by Nishi-Kohn SNC-Lavalin Limited (NKSL), with the design presented in NKSL (1999, 2001). Placement of waste rock within the NCRP commenced at mine start up in 2003 and the majority of the waste rock to be stored in the NCRP was in place by 2012. The waste rock in the NCRP has generally been placed from the bottom up in relatively thin lifts. The NCRP side slopes have been left at angle of repose during operations, at approximately 1.3 horizontal: 1 vertical (1.3H:1V).

The NCRP waste rock is predominantly tonalite (granite), with lesser amounts of pegmatitic granite, biotite schist and diabaise. Waste rock was disposed in different areas of the NCRP based on potential for acid generation. The potential for acid generation was determined from analysis of sulfur content in the waste rock (University of Waterloo, British Columbia and Alberta, 2010). Three waste rock categories were defined based on acid generating potential:

- Type I waste rock (<0.04% sulphur) is clean rock, composed of granite and pegmatitic granite, and is considered to have no acid generating potential.
- Type II waste rock (0.04 to 0.08% sulphur) is predominantly granite with a minor amount of biotite schist, and is considered to have no or low acid generating potential.
- Type III waste rock (>0.08% sulphur) is a mixture of rock with a greater proportion of biotite schist and is considered to be potentially acid generating.

Limits of areas of the NCRP that contain Type II or Type III waste rock are presented in Figure 2.

Re-mining of selected zones of the NCRP was started in 2013 to provide rock to support underground operations and for the construction of the PKC Facility. Re-mining of the NCRP continued in 2015 to provide rockfill for the construction of the A21 dike.

2.0 DESIGN BASIS

2.1 Site Conditions

The site lies within the Arctic Climatic Region in a region of continuous permafrost. The climate is extreme, with long, cold winters and very short, cool summers. The mean daily air temperature is 12°C in July and -29°C in January. The mean annual temperature at the site is approximately -10°C. Site climate is described in more detail in Golder (2008).

2.1.1 Foundation Conditions

The surficial geology on East Island includes varying depths of ablation or glaciofluvial tills consisting of silty sand or sandy silt with gravel and boulders, overlying granitic or metasedimentary bedrock. The surficial till cover is absent and bedrock is exposed at the surface over large areas. Soft lake sediments, generally fine grained cohesionless soils, were also identified in shallow ponds that are distributed across the island. On the north side of the island, an intermittent esker trends in an approximate east-west direction (NKSL 1999).

The near surface soils are generally re-worked by frost action and show evidence of periglacial processes such as solifluction, thermal cracking, and boulder jacking. The active zone, or depth that thaws each year, is approximately 1.5 m to 2.0 m deep in soil deposits, 2.0 m to 3.0 m in well drained granular deposits (eskers), 5.0 m in bedrock, and less than 1 m depth in poorly drained areas including bogs, and areas with thicker vegetation (NKSL 1999).

Typical foundation conditions within the NCRP footprint include between 0 m and 3 m of organic cover, and 0 m to 6 m of primarily till soils overlying bedrock. Soils with greater than 10% visible ice content and 30% moisture content were classified as being 'ice rich' and are present on the island (NKSL 1999).

A plan of the NCRP area presenting the original ground surface topography and foundation conditions is included in Figure 3. The anticipated footprint of the NCRP after re-slope and prior to placement of the cover is also included in Figure 3.

2.1.2 Seismicity

The Diavik site is situated in a region of low seismicity. Interpolated seismic hazard values for the site were determined for coordinates 64.5°N, 110.3°W from fourth generation seismic hazard maps of Canada following the 2010 National Building Code of Canada. Results of the seismic hazard assessment were obtained from Natural Resources Canada (Appendix D). The predicted peak ground accelerations (PGA) for different return periods are summarized in Table 1.

Table 1: Peak Ground Acceleration Values for Diavik Site, 64.5°N, 110.3°W

Return Period (years)	Peak Ground Acceleration (g)
1 in 100	0.003
1 in 475	0.011
1 in 1,000	0.019
1 in 2,475	0.036

Note: For firm ground site class C. Source: Appendix D

2.2 Cover System Design

A closure cover system design geometry for the NCRP was developed by others and provided by DDMI as a design input for the closure of the NCRP. The closure cover design includes:

- a minimum 1.5 m thick till cover (measured perpendicular to the slope) on all areas where the NCRP waste rock is classified as Type II and Type III; and
- a minimum 3 m thick Type I waste rock capping layer over the till cover (measured perpendicular to the slope).

As described in Rio Tinto (2011) and Universities of Waterloo, British Columbia and Alberta (2010), the primary function of the till cover is to reduce infiltration and promote permafrost within the underlying waste rock. The till layer is not designed as a low permeability layer but rather as a permanent frozen layer (or permafrost aggradation layer) as classified in MEND (2012). The 3 m thick Type I waste rock capping layer serves as erosion protection and thermal protection of the 1.5 m thick till layer.

Areas of the NCRP where the waste rock is classified as Type I are not re-sloped and do not include placement of the cover system.

2.3 Physical Stability and Construction Criteria

The re-slope and cover construction for NCRP closure must consider the following criteria:

- The final NCRP footprint is to be minimized.
- The final NCRP closure configuration is to meet or exceed minimum design criteria for Factors of Safety (FoS) for rock pile stability (described in Table 2).
- Access to the perimeter and top of the NCRP is to be maintained during the closure construction, and during post closure. Access road widths for one-way (single) traffic for 220 tonnes (240 US ton) Komatsu 830 haul trucks are to be maintained during closure construction and post closure.
- ROM materials are to be used efficiently as they become available from the A21 open pit operation. Pre-stripping of the A21 pit is scheduled to commence in 2018. Requirement for till and Type I ROM materials from sources other than A21 pit activity are to be minimised.
- The till cover upper surface is to be graded or shaped ditches excavated to limit ponding on the till within the cover, and promote water flow towards the surrounding collection ponds.
- Access for cariboo to the perimeter and top of the NCRP is to be provided for post closure conditions.
- The NCRP closure design must meet minimum requirements of constructability, including access and slopes angles for placement of the cover system.

Selected zones of the NCRP were being re-mined in 2015 to provide rock to other operations on site. The NCRP configuration after re-mining is presented in Figure 2.

Table 2 presents a summary of NCRP closure physical stability and construction design criteria.

Table 2: NCRP Closure Physical Stability and Construction Design Criteria

Criteria	Value	Source/Comment
North Country Rock Pile (NCRP) Footprint	Minimize final NCRP footprint	Undisturbed areas during operation of the mine should remain undisturbed during and after closure (Rio Tinto 2011)
Cover system materials	Till and Type I waste rock for cover and capping layers sourced from A21 Pit stripping operations	DDMI
Design peak ground acceleration (1 in 2,475 year return period seismic event)	0.036 g	Appendix D
Minimum Factor of Safety (FoS) for slope stability	1.3 (static loading) 1.0 (pseudo-static loading)	BCMWRPRC (1991) for static loading CDA (2013) for pseudo-static loading for dams, also common engineering practice for waste piles
Model of largest size of haul truck	Komatsu 830E	DDMI
Maximum Haul Truck Width	8.70 m	830E Operating width including mirrors, Komatsu literature
Tire Size/Diameter	40.00R57/3.6 m	Standard tire, Komatsu literature http://www.komatsuamerica.com
Haul road width for 1-way traffic[a]	31.4 m including shoulder berms on both sides; 24.4 m including shoulder berm on one side only; and 17.4 m with no shoulder berms	NT Mine Health and Safety Regulations S.1.143. Shoulder berms required where a drop of more than 3 m is possible.
Maximum haul road gradient	8%	Adopted, Golder
Minimum height of shoulder barrier [a]	2.7 m	Required where a drop-off greater than 3 m exists

Notes:

[a]: The access road width should comply with the most current NWT Mine Health and Safety Act and Regulations for minimum width of haul roads. For single lane traffic the minimum width is twice the width of the widest haulage vehicle used on the road. A shoulder barrier of at least three-quarters the height of the largest tire on any vehicle using the road is required.

3.0 NCRP CLOSURE DESIGN

The design presented here considers re-sloping of selected areas of the waste rock in the NCRP and placement of a cover system over the Type II and Type III waste rock.

3.1 Design Section and Layout

Two different design sections will be used for closure of the NCRP based on areas requiring cover and areas to remain uncovered.

- Areas of the NCRP where the waste rock is classified as Type II or Type III require placement of the cover system. Limits of the Type II and Type III areas of the NCRP are shown in Figure 2. In these areas, the NCRP side slopes are to be re-sloped from angle of repose to 3H:1V or flatter. The cover system described in Section 2.2 is then placed over the Type II and Type III area. The design slope of 3H:1V was selected to allow till to be placed directly on the slope.
- Areas of the NCRP where the waste rock is classified as Type I do not require placement of the cover system. In these areas, the NCRP side slopes will remain at angle of repose and with the configuration of benches shown in Figure 2.

Areas that are excluded from the initial re-sloping, include:

- Type III waste rock located at the southeast corner of the NCRP (Figure 2) that will be re-mined during construction of the NCRP closure will not require re-slope and placement of the cover system. The side slopes of the NCRP in this area can remain at angle of repose.
- The cover system along the south side of the NCRP will extend to the liner on the crest of the PKC Facility North Dam. The crest elevation and configuration of the liner is to be confirmed during construction of the NCRP closure, following or in conjunction with the development of a design for the raise of the PKC Facility North Dam above elevation 465 m.
- The slope of the cover system over the non-burnable waste dump has been designed flatter than a 3H:1V slope; however, this slope can be steepened to a maximum of 3H:1V.
- Type I areas.

Other constraints on sequencing of re-sloping and placement of the cover system include the area along the north side of the NCRP, where re-sloping of the waste rock will partially cover the airport road and power line. Similarly, a portion of Pond 3 will be covered by the re-slope of the NCRP and placement of the cover system along the west side of the NCRP. Relocation of the airport road and power lines and evaluation of Pond 3 storage capacity is not part of the scope of this design.

The network of roads on the NCRP has been maintained to allow access to the crest and side slopes of the NCRP during re-sloping and cover construction, and following closure. After re-sloping of the side slopes of the NCRP, the minimum width of the access roads and ramps allows for one-way traffic of Komatsu 830 E haul trucks. Additional ramps for cariboo access will be provided at the southeast and northwest corners of the NCRP where the side slopes are not required to be re-sloped based on material types.

3.2 Construction Materials

Re-sloped areas of the NCRP will be re-sloped to 3H:1V with a balanced cut and fill such that no additional rockfill will be sourced for the re-slope.

The till and Type I rockfill used for the construction of the cover system will be obtained during the development of the A21 pipe. Where possible, ROM till and rockfill will be placed directly onto the NCRP for closure cover construction, to limit the requirement for stockpiling and re-handle.

Investigations of the A21 open pit area indicate the till is a well graded silt, sand and gravel mixture with a fines content (particles less than 0.075 mm) generally between 15% and 45% (Golder 2006 and 2007).

The Type I rockfill is expected to be sound, hard, durable rockfill, free from organic matter and debris and other unsuitable material.

The material specifications for construction of the NCRP closure are included in Appendix B.

3.3 Water Management

A system of collection ponds was built around the NCRP to collect surface runoff from the NCRP during operations and following closure. The system of ponds around the NCRP includes Ponds 1 and 13 on the east side, the North Inlet along the north side, and Ponds 2 and 3 on the west side. The North Dam of the PKC Facility forms part of the NCRP south slope and water accumulating within the south side of the NCRP flows east and west to Ponds 1 and 3. During mine operations water reporting to the collection ponds is pumped to the North Inlet or the Process Plant.

Placement of till cover along the south slope of the NCRP will direct water over the PKC Facility North Dam into the PKC Facility from which it can be pumped to the North Inlet or the Process Plant. Surface runoff from other areas of the NCRP continue to report to the surrounding collection ponds and the North Inlet during and after NCRP re-sloping and closure cover construction. The surface of the till layer in the cover will be graded to promote surface runoff, and may include small ditches excavated on the till cover surface. During construction, grading of the till surface will also limit potential erosion. Grading to promote drainage will be defined during construction. Maintenance works may be required during construction to re-grade the till surface and maintain a minimum 1.5 m minimum till layer thickness.

While described here, the management of water reporting to the collection ponds, North Inlet and PKC Facility during closure and post closure are not part of the scope of the design presented in this report.

3.4 Construction Quantities

The estimated quantities for re-slope and construction materials for the NCRP closure are presented in Table 3. The waste rock quantity required to raise the PKC Facility North Dam crest from elevation 465 m to 475 m is not included in Table 3, as this design would be completed separately. The quantities presented in Table 3 are approximate and do not contain a contingency. The actual as-built quantities will likely vary from those listed in Table 3.

Table 3: NCRP Closure Construction Quantities

Material/Activity	Unit	Quantity
Re-sloped Area	m^2	835,000
Till Cover	m^3	2,000,000
Type I ROM Capping	m^3	3,980,000

Notes: Quantities are neat line in-place values.

3.5 Construction Drawings and Specifications

The Issued-for-Construction versions of the construction Drawings and Technical Specifications are presented in Appendix A and B, respectively.

3.6 Geotechnical Instrumentation

Existing geotechnical instrumentation installed in the NCRP includes observation and collection wells, thermistors and a slope inclinometer, summarised in Table 4.

Table 4: NCRP Existing Geotechnical Instrumentation

Instrument Type	Instrument ID	Easting (m)	Northing (m)
Observation Well	NCRP-SCW-W1	532918	7152817
Observation Well	PKCN-SCW-3951	533504	7152587
Collection Well	PKCN-SCW-3123	534508	7152241
Observation Well	PKCN-SCW-3154	534490	7152269
Thermistor	NCRP-TN1A	533350	7153091
Thermistor	NCRP-TN1B	533350	7153091
Thermistor	FD1-T	534211	7152777
Thermistor	FD2-T	534208	7152773
Thermistor	FD3-T	534213	7152773
Inclinometer	NCRP-INN1	533350	7153091

Following construction of the cover, monitoring of the NCRP for deformation should continue to be completed by visual inspection and by aerial survey methods on an annual basis for 5 years, with results reviewed annually. After 5 years the frequency of inspections and survey should be revisited. Required maintenance works should be identified as part of the inspections and monitoring of the NCRP. Maintenance works may include, but not be limited to, re-grading of surfaces and maintenance of geotechnical instrumentation. The instrumentation presented in this report considers geotechnical stability only.

4.0 ANALYSES

Analyses to assess the physical stability of the NCRP closure design are presented here. The original design analyses for the NCRP thermal performance and stability are presented in NKSL (1999, 2001). An additional assessment of the physical stability of the NCRP is presented in Appendix C, and includes a sensitivity analysis to determine the maximum overall slope angle which satisfies the design criteria for minimum Factors of Safety against slope instability, and creep rate.

Analyses completed by others for thermal and seepage performance of the NCRP are summarized here for reference.

4.1 Stability Analyses

4.1.1 Failure Modes

4.1.1.1 Rock Pile Failure Modes

Waste rock piles typically undergo long term settlements which can reach magnitudes of up to 1 to 2% of the pile height. Slumping and settling of the crest and sloughing on the outer surface with occasional boulder roll-out are considered normal behaviour.

A worst case failure mechanism for large waste rock piles is rapid run-out failure involving large scale mass movements. The NCRP waste rock is a high strength frictional material that is not prone to rapid weathering and breakdown. Rapid run-out failures of the NCRP must therefore involve loss of strength in the foundation at the toe area of the pile.

Credible mechanisms for rapid loss of strength in the foundation soils include increase in pore water pressures induced by loading, by rapid thawing of ice rich soils, or by creep rupture of ice rich soils.

- The waste rock in the NCRP has generally been placed from the bottom up in relatively thin lifts which does not cause rapid loading of the foundation. Placement of the cover system is a relatively small additional loading for the NCRP. Therefore, loading induced failure by placement of the cover system and over the long term is not considered to be a credible mechanism of failure.

- Rapid thaw of ice rich soils resulting in excess pore water pressures is not considered to be a credible failure mechanism. The thaw-consolidation ratio for foundation soils at the NCRP is expected to be low (Golder 2007). Therefore, build-up of pore water pressure is not expected to occur in the foundation should thawing conditions prevail.

- The pile will be subject to foundation related creep where founded on ice rich soils. High ice content soils generally carry more load on the ice phase and the ice creeps under load resulting in creep behaviour of the soil. Ice rich soils may be subject to changes in creep through primary, secondary and tertiary phases. The primary phase is characterized by decreasing creep rate, the secondary phase by steady creep rate and the tertiary phase by increase in strain rate up to creep rupture. Creep behaviour of ice rich foundation soils will result in foundation and pile movement over time.

Evidence of ice build-up within the NCRP has been observed from seepage from the PKC Facility. Building of ice within the waste rock can cause slow downslope movements, on the order of centimetres per year, similar to rock glaciers. Slow downslope movements and creep type are not viewed as a credible failure mechanism. Drainage from the NCRP is not included in the scope of this design, and should be considered as part of monitoring for the facility.

4.1.1.2 Cover System Failure Modes

Solifluction, skinflows and bimodal flows are possible failure mechanisms of cover systems in cold regions (MEND 2012). These mass wasting events occur within the active layer (depth of seasonal thaw) and result in the slow movement of non-frozen, saturated soil over the impermeable permafrost layer below. This movement can result in topographic changes to the till cover and Type I ROM layer.

Solifluction, skinflows and bimodal flows occur when frozen soil within the active layer thaws faster than pore water pressure is dissipated, generating excess pore water pressures within the soil layer. The presence of these high pore water pressures increases the potential for slippage of the till cover placed on a slope. Based on temperature measurements collected from a large-scale test pile built at Diavik Mine and numerical simulations (Pham, 2013), the active layer may penetrate the 3 m thick Type I rockfill layer and into the till cover layer. Therefore, solifluction, skinflows and bimodal flows are considering credible failure modes for the cover system.

Similarly, should water pond on the till layer, and the active layer fully penetrate the till, then loss of finer particles from the till into the underlying waste rock is possible due to filter incompatibility.

Failure of the Type I ROM layer of the cover system is not considered to be a credible failure mechanism.

4.1.2 Stability of the Rock Pile

The sensitivity stability analyses presented in Appendix C to determine a maximum pile slope angle while meeting stability criteria are considered to adequately represent the NCRP closure configuration and material properties. Appendix C provides details of the material properties adopted for assessment of the rock pile stability. The peak ground acceleration used in the analyses presented in Appendix C is higher than the peak ground acceleration for a 1 in 2,475 year return period seismic event (Appendix D); therefore, the pseudo-static stability analysis results presented in Appendix C are considered to be conservative.

The sensitivity analyses in Appendix C indicate that side slope angles of 1.3H:1V or shallower will meet or exceed the minimum design criteria factor of safety for slope stability for the range of foundation conditions under both static and pseudo-static loading conditions. The NCRP closure design sections have 3H:1V and 1.3H:1V slopes; therefore the NCRP closure design sections satisfy the design criteria factor of safety.

Limiting creep rate criteria was used in the assessment of the stability analyses for areas of the NCRP with ice rich foundations (Appendix C). A limiting creep rate of 0.01 mm/year was used for the analyses. The results indicated that the side slopes must be no steeper than 2H:1V and 3H:1V for ice-rich foundation conditions at -4°C and -2°C, respectively, to meet the creep rate criteria. The areas of the NCRP where the side slopes will be re-sloped to 3H:1V slopes for placement for the cover system satisfy the creep rate criteria. At the southeast and northwest corners of the NCRP where the design section does not include re-sloping the side slopes, the maximum overall slope angles are approximately 2.8H:1V and 2.6H:1V, respectively. No large displacements have been observed to date in these areas. Creep movements higher than 0.01 mm/year may occur at the lower benches at these areas if the ice-rich foundation temperature is near -2°C; however, rapid run-out failures are not expected. Some tolerable movement is therefore expected, and monitoring has been specified.

4.1.3 Stability of the Cover System

The physical stability of the till layer in the cover was analysed using infinite slope analyses. Infinite slope analysis is considered a suitable assessment method due to the long slope length, and consistent slope angle and materials. Two cases were considered in the analysis:

- Case 1: Stability of the 1.5 m till layer on a 3H:1V slope before placement of the Type I ROM capping layer (short-term condition).
- Case 2: Stability of the 1.5 m till layer on a 3H:1V slope after placement of the 3 m thick Type I ROM capping layer (long-term condition).

The two cases have been analysed using infinite slope analysis methods published in literature that include the effects of increased pore pressures generated during thaw. Case 1 follows the equation by McRoberts and Morgenstern (1973) and Hannah and McRoberts (1988) to calculate the Factor of Safety of the till cover layer. Case 2 uses the equation presented in Pufahl and Morgenstern (1979) and Mageau and Rooney (1984) which allows for the stabilising effect provided by the ROM capping layer. For an infinite slope inclined at angle *i*, with till layer thickness *d* and capping thickness d_0 (see Figure 4), the factor of safety is calculated as follows:

Case 1: (No ROM Capping Layer):

$$FoS = \frac{c'}{\gamma d \sin i \cos i} + \frac{\frac{\gamma'}{\gamma}}{1+2R^2}\,\frac{\tan\varphi'}{\tan i}$$

Case 2: (3 m ROM Capping Layer):

$$FoS = \frac{c'}{(\gamma_0 d_0+\gamma d)\sin i \cos i} + \frac{\gamma_0 d_0(1-R")+\gamma' d(1+2R^2)^{-1}\tan\varphi'}{(\gamma_0 d_0+\gamma d)\tan i}$$

Where $R = \frac{\alpha}{2\sqrt{c_v}}$ $\quad \alpha = 2\sqrt{\alpha_u}\left(\frac{Ste}{2}\right)^{\frac{1}{2}}(1-\frac{Ste}{8})$ $\quad Ste = \frac{c_{vu}T_S}{L}$ $\quad \alpha_u = \frac{k}{c_{vu}}$ $\quad R" = \frac{2.28R^2}{(1+2R^2)}$

And the parameters above are defined as follows:

i	Slope angle	R	Thaw-consolidation ratio for Till cover
ϕ'	Effective friction angle for Till cover	R''	Pore pressure factor for Till cover
c'	Effective cohesion for Till cover	c_{vu}	Unfrozen Volumetric Heat Capacity for Till cover
γ	Bulk unit weight for Till cover	α	(unnamed parameter)
γ_0	Bulk unit weight for Type I layer	α_u	Unfrozen soil thermal diffusivity for Till cover
γ'	Effective unit weight for Till cover	Ste	Stefan's number for Till cover
d	Thickness (vertical) of Till cover	T_s	Constant surface temperature
d_0	Thickness (vertical) of Type I layer	k	Unfrozen thermal conductivity for Till cover
c_v	Coefficient of consolidation for Till cover	L	Volumetric latent heat for Till cover

Calculation of the pore pressure parameter (R) requires an estimate of the constant surface temperature (T_s) acting on the surface of the till layer during thaw. For Case 1, the T_s parameter represents the surrounding air temperature. A T_s value of 12°C has been adopted for Case 1, which is the average maximum air temperature in the Diavik area during the month of July (Golder, 2008). For Case 2, T_s is taken as the temperature near the base of the ROM capping layer. A T_s value of 5°C has been adopted for Case 2, which is based on thermistor data recorded from a large-scale test pile built at Diavik Mine (University of Waterloo, British Columbia and Alberta 2010).

The material properties adopted for the stability assessment of the cover are presented in Table 5. The adopted values are based on the 2006 and 2007 field investigations and laboratory testing result (Golder 2006 and 2007). A bulk unit weight of 20 kN/m^3 was adopted for the Type I ROM capping layer.

Table 5: Material Properties for Till Cover Layer

Property	Till
Physical Properties	
Bulk Unit Weight, γ (kN/m^3)	22.0
Effective Unit Weight, γ' (kN/m^3)	12.0
Effective Friction Angle, φ' (degrees)	34
Effective Cohesion, c' (kPa)	0
Coefficient of Consolidation, c_v (m^2/year)	40
Thermal Properties	
Unfrozen Thermal Conductivity, k (J/s-m-C)	2.7
Unfrozen Volumetric Heat Capacity, c_{vu} (kJ/m^3-C)	2245
Volumetric Latent Heat, L (kJ/m^3)	90000

4.1.4 Stability Analyses Results

Table 6 presents a summary of the factors of safety against solifluction, skinflows and bimodal flows occurring in the till layer within the NCRP cover system.

Table 6: Summary of Results of Stability Analyses for 3H:1V Slope

Case Analysed	Calculated Factor of Safety	Criteria for Minimum Factor of Safety for Static Loading
Case 1: 1.5 m thick till layer, prior to placement of Type I layer	0.9	1.3
Case 2: 1.5 m thick till layer and 3 m thick Type I layer	2.1	

The results of analyses indicate that the design criteria for Factor of Safety are not met for the short term condition where the 3 m thick Type I capping layer has not yet been placed on the till cover. Therefore, if the till cover is left uncapped and allowed to go through freeze-thaw cycling, then some minor slippage of the till layer is expected to occur due to frost action. Maintenance works may be required during construction to re-grade the till surface and maintain a minimum 1.5 m till layer thickness. The design criteria for Factor of Safety are met for the final cover system configuration.

4.2 Thermal Analyses

Pham (2013) presents the results of large-scale test piles and numerical simulations used to model the thermal regimes within the NCRP for various closure configurations and climate scenarios.

Temperature measurements collected from a large-scale test pile constructed with the adopted cover system showed the active layer remained within the Type I ROM capping while the underlying till layer and waste rock remained frozen year round. Under a warming climate scenario, numerical simulations suggest an active zone of up to 3.9 m (i.e., extending into the till layer but still contained within the cover system) for a simulation period of 100 years (Pham, 2013).

Reference should be made to Pham (2013) for further information.

No thermal analyses were undertaken as part of this design report.

4.3 Seepage Analyses

Smith (2013) presents the results of analyses undertaken to predict seepage quality for various closure configurations of the NCRP. The predictions are based on quantitative models and data collected from large-scale test piles constructed at the Diavik Mine.

The results of the analyses suggest that seepage from the adopted closure configuration (with active zone contained within the cover system) would have a near-neutral pH and lower concentration of metals than an uncovered configuration (with active zone within the waste rock). Reference should be made to Smith (2013) for further information.

No seepage analyses were undertaken as part of this design report.

5.0 REPORT CLOSURE

The reader is referred to the Study Limitations, which follows the text and forms an integral part of this Report.

We trust the information in this report meets your requirements at this time. If you have any questions relating to the information contained in this document, please do not hesitate to contact us.

GOLDER ASSOCIATES LTD.

ORIGINAL SIGNED

ORIGINAL SIGNED AND SEALED

John Cunning, P.Eng.
Principal, Senior Geotechnical Engineer

Ben Wickland, Ph.D., P.Eng.
Associate, Senior Geotechnical Engineer

GP/JCC/BEW/rs

o:\final\2015\3 proj\1521339 ddmi surface projects\1521339-1471-r-rev3-7000\1521339-1471-r-rev3-7000-ncrp closure design 29mar_17.docx

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Signature ORIGINAL SIGNED
Date March 29, 2017
PERMIT NUMBER: P 049
NT/NU Association of Professional Engineers and Geoscientists

REFERENCES

BCMWRPRC (British Columbia Mine Waste Rock Pile Research Committee) 1991. *BC Mine Waste Rock Pile Research Committee, Mined Rock and Overburden Piles Investigation and Design Manual.* Interim Report.

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Pufahl, D. E., and Morgenstern, N. R. 1979. Stabilization of planar landslides in Permafrost. Can. Geotech. J. 16(4): 734-47.

Rio Tinto. 2011. *Design Memorandum North Country Rock Pile Closure.* Version 1, dated July 2011.

Universities of Waterloo, British Columbia and Alberta, 2010. *Diavik Waste-Rock Research Program: Phase I Final Report*.

Smith, L. 2013. *Predictions of Seepage Quality from the North Country Rock Pile at Closure*, dated December 13, 2013.

STUDY LIMITATIONS

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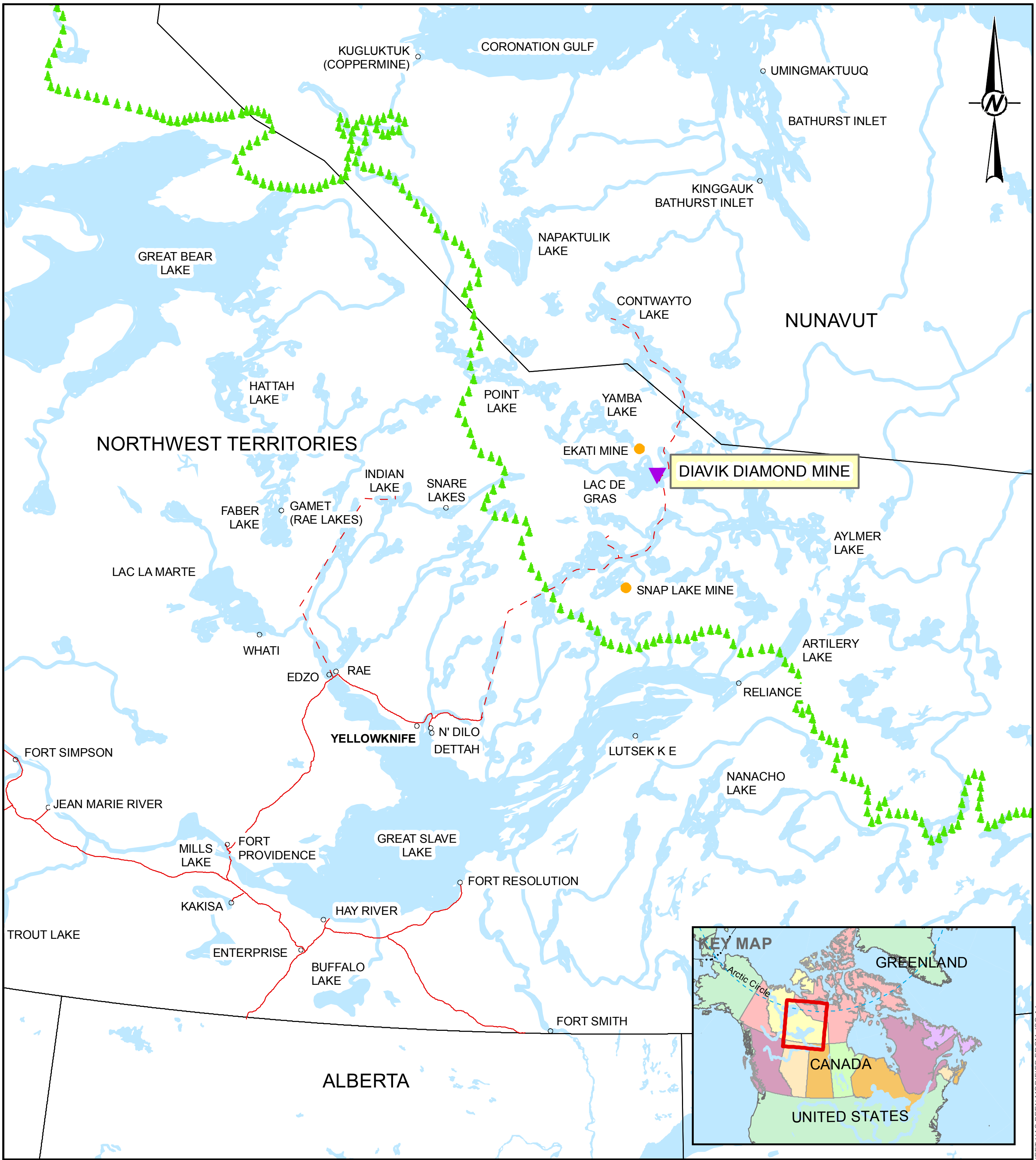

LEGEND

- All Weather Roads
- Winter Roads
- Treeline
- Communities

REFERENCE

DATUM: NAD83 PROJECTION: UTM ZONE 12

CLIENT

DIAVIK DIAMOND MINES INC.

PROJECT

DIAVIK DIAMOND MINES (2012) INC.
NORTH COUNTRY ROCK - PILE CLOSURE

TITLE

KEY PLAN
LOCATION OF DIAVIK DIAMOND MINE

CONSULTANT

Golder Associates

YYYY-MM-DD	2016 JAN 14
PREPARED	CA
DESIGN	GP
REVIEW	GP
APPROVED	BW

PROJECT No.	PHASE	Rev.	FIGURE
1521339	7000	0	1

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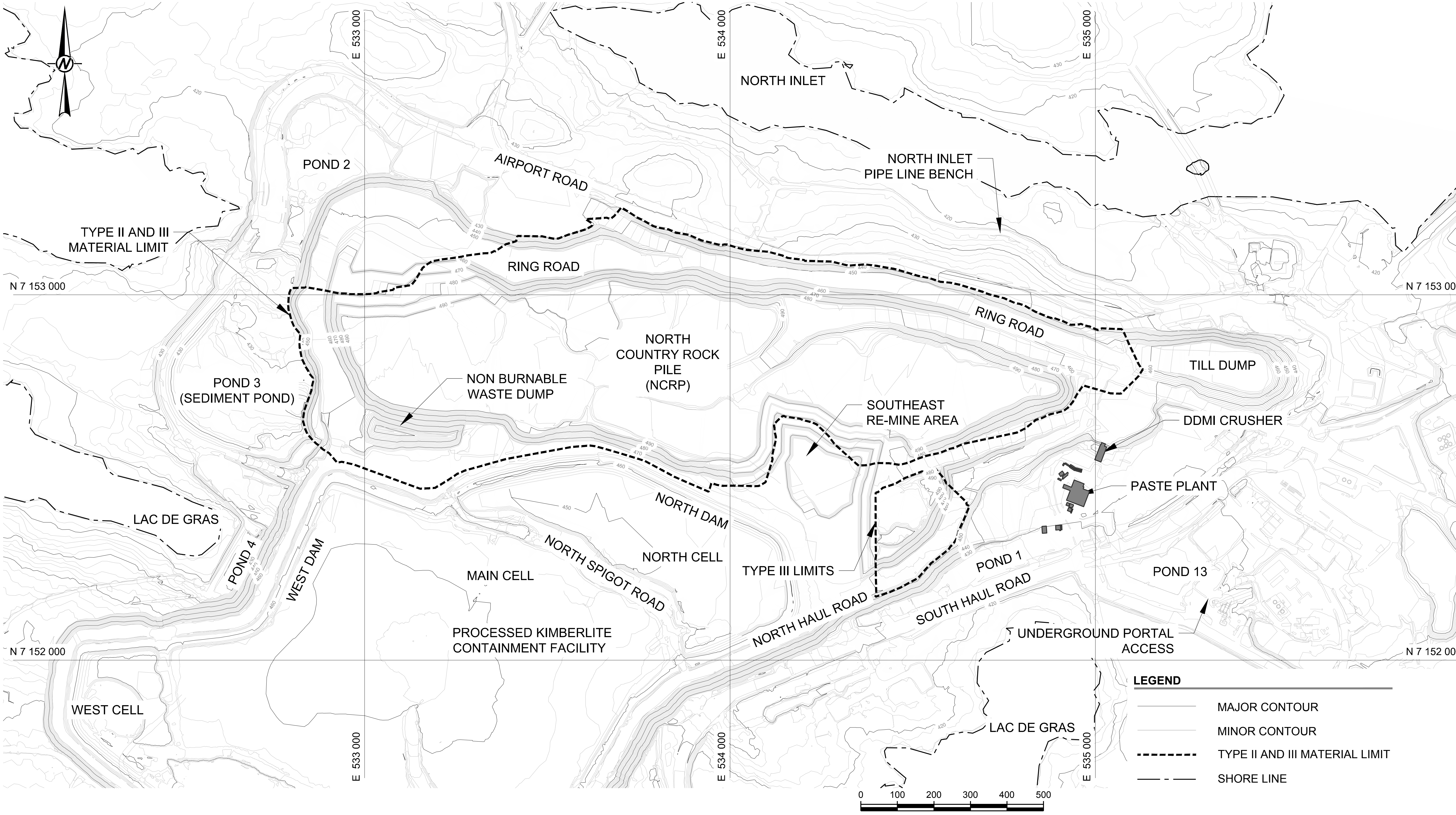

NOTES

1. COORDINATES ARE IN MINE GRID.
2. ALL UNITS IN METRES UNLESS OTHERWISE NOTED.
3. MINOR CONTOUR INTERVAL IS 2 m. MAJOR CONTOUR INTERVAL IS 10 m.

REFERENCES

1. INFRASTRUCTURE AND NOVEMBER 2014 GROUND CONTOURS PROVIDED BY DDMI, DATE: JUNE 1, 2015, DOWNLOADED FROM DDMI FILE TRANSFER PROTOCOL (FTP) SITE.

CLIENT

DIAVIK DIAMOND MINES INC.

CONSULTANT

Golder Associates

YYYY-MM-DD	2016 JAN 15
DESIGNED	GP
PREPARED	AM
REVIEWED	GP
APPROVED	BE

PROJECT

DIAVICK DIAMOND MINES (2012) INC.
NORTH COUNTRY ROCK PILE CLOUSURE

TITLE

NORTH COUNTRY ROCK PILE
PLAN BEFORE RE-SLOPE

PROJECT NO.	PHASE/TASK	REV.	FIGURE
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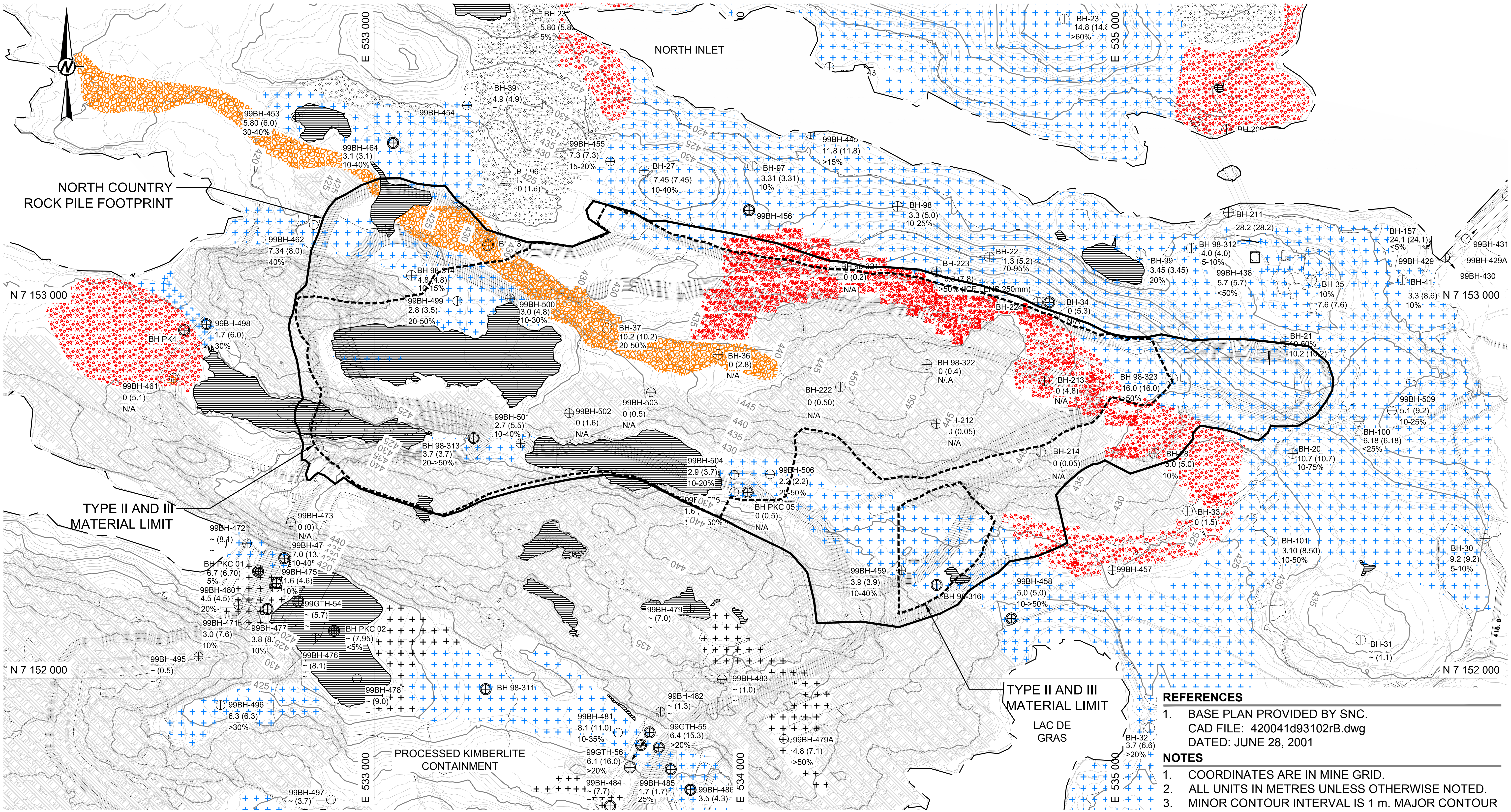

REFERENCES

1. BASE PLAN PROVIDED BY SNC.
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   DATED: JUNE 28, 2001

NOTES

1. COORDINATES ARE IN MINE GRID.
2. ALL UNITS IN METRES UNLESS OTHERWISE NOTED.
3. MINOR CONTOUR INTERVAL IS 1 m. MAJOR CONTOUR INTERVAL IS 5 m.

LEGEND

- ICE POOR GLACIAL TILL (<10% VISIBLE ICE)
- ROCK
- LAKE BED SEDIMENTS
- ESKER
- ICE RICH GLACIAL TILL (>10% VISIBLE ICE)
- CAN NOT BE IDENTIFIED
- NORTH COUNTRY ROCK PILE FOOTPRINT AFTER RESLOPE
- TYPE II AND III MATERIAL LIMIT BEFORE RE-SLOPE
- SHORE LINE
- BOREHOLE
- BOREHOLE WITH THERMISTOR
- 98-320 BOREHOLE NUMBER
- 0.5 (1.75) DEPTH TO ROCK / DEPTH OF SOIL CONTAINING GROUND ICE
- 5% ICE CONTENT (%)

CLIENT
DIAVIK DIAMOND MINES INC.

CONSULTANT
Golder Associates

YYYY-MM-DD	2016 JAN 15
DESIGNED	GP
PREPARED	AM
REVIEWED	GP
APPROVED	BE

PROJECT
DIAVICK DIAMOND MINES (2012) INC.
NORTH COUNTRY ROCK PILE CLOUSURE

TITLE
NORTH COUNTRY ROCK PILE FOUNDATION CONDITIONS

PROJECT NO.	PHASE/TASK	REV.	FIGURE
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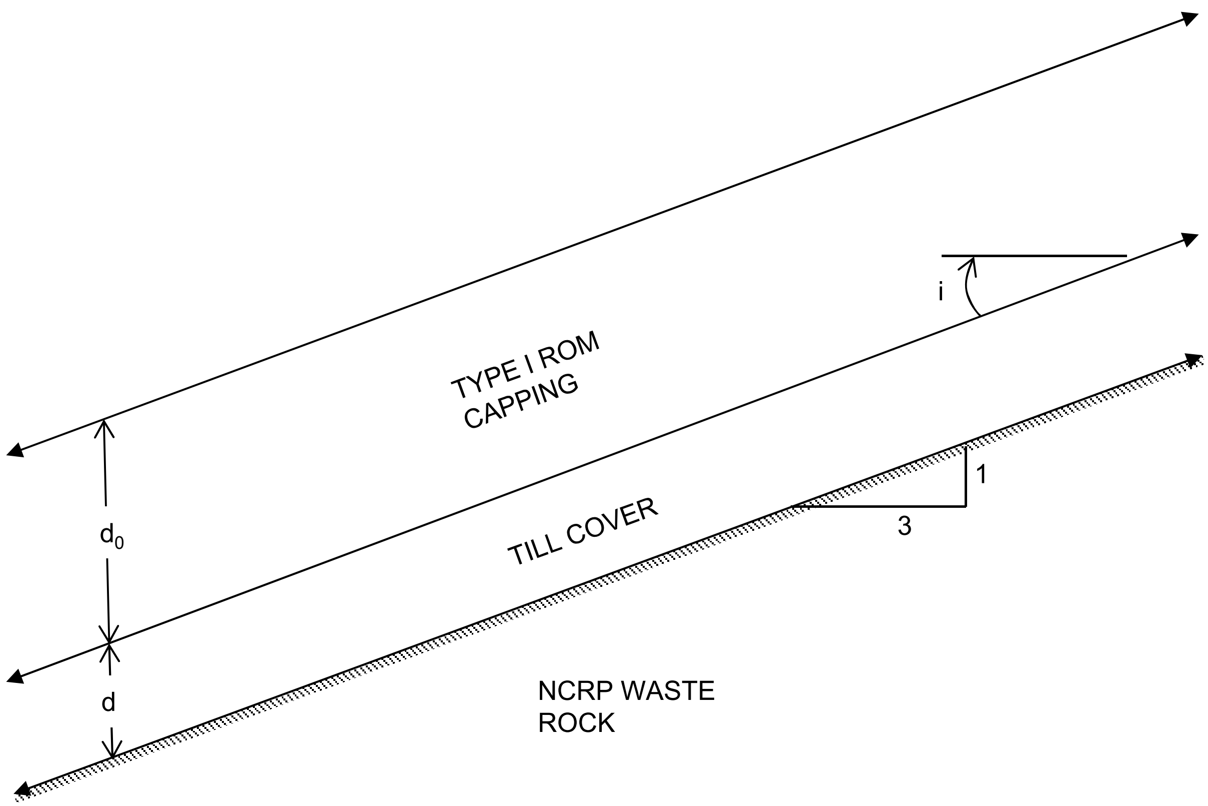


PROJECT

DIAVIK DIAMOND MINES (2012) INC.
NCRP CLOSURE DESIGN REPORT

TITLE

INFINITE SLOPE GEOMETRY FOR COVER STABILITY ASSESSMENT

Golder Associates

PROJECT No. 1521339			PHASE No. 7000	
DESIGN	GP	20 JAN 16	SCALE NTS	REV. 1
CADD	-	-	**FIGURE 4**	
CHECK	GP	20 JAN 16		
REVIEW	BW	20 JAN 16		

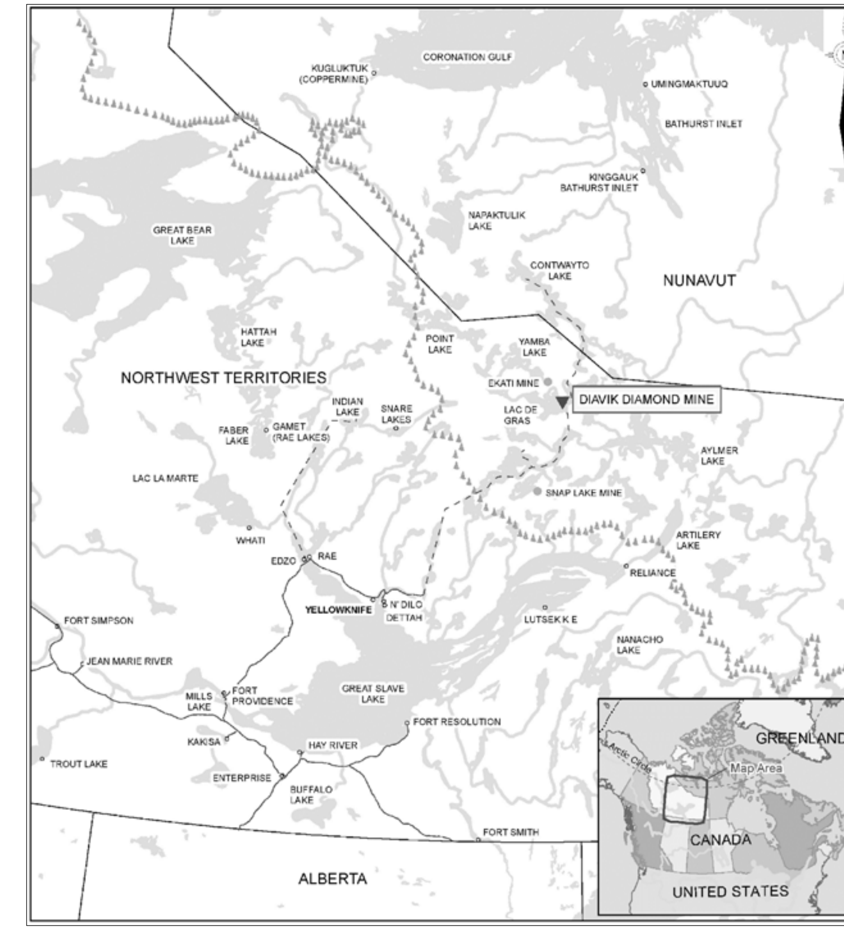
APPENDIX A

Construction Drawings

DIAVIK DIAMOND MINES (2012) INC.
NORTH COUNTRY ROCK PILE CLOSURE CONSTRUCTION DRAWINGS

INDEX TO DRAWINGS

DRAWING NO.	DRAWING TITLE	REVISION
000	DRAWING TITLE SHEET	1
001	DIAVIK DIAMOND MINE SITE PLAN	0
002	NORTH COUNTRY ROCK PILE - PLAN BEFORE RE-SLOPE	0
003	NORTH COUNTRY ROCK PILE - AFTER RE-SLOPE	0
004	NORTH COUNTRY ROCK PILE - FINAL CONFIGURATION	0
005	NORTH COUNTRY ROCK PILE - CROSS SECTIONS	0
006	NORTH COUNTRY ROCK PILE - DETAILS	0
007	NORTH COUNTRY ROCK PILE - GEOTECHNICAL INSTRUMENTATION LAYOUT	0



SITE LOCATION PLAN
NTS

LIST OF SPECIFICATIONS

SPECIFICATION NO.	TITLE	REV. No.	REVISION TITLE
1470	NORTH COUNTRY ROCK PILE CLOSURE CONSTRUCTION TECHNICAL SPECIFICATIONS	2	ISSUED FOR CONSTRUCTION

PERMIT TO PRACTICE
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Signature ORIGINAL SIGNED
Date March 29, 2017
PERMIT NUMBER: P 049
NT/NU Association of Professional Engineers and Geoscientists

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DRAWING REDUCED

REV.	YYYY-MM-DD	DESCRIPTION	DESIGNED	PREPARED	REVIEWED	APPROVED
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B	2015-11-24	ISSUED FOR CLIENT REVIEW	AMF	MGL	GP	BEW
A	2015-11-23	ISSUED FOR INTERNAL REVIEW	AMF	MGL	GP	BEW

SEAL

CLIENT
DIAVIK DIAMOND MINES (2012) INC.

CONSULTANT
Golder Associates
VANCOUVER
SUITE 200 - 2920 VIRTUAL WAY
VANCOUVER, BC
CANADA
[+1] (604) 296 4200
www.golder.com

PROJECT
DIAVIK DIAMOND MINE
NORTH COUNTRY ROCK PILE CLOSURE

TITLE
DRAWING TITLE SHEET

PROJECT NO.	PHASE / TASK	REV.	1 of 8	DRAWING
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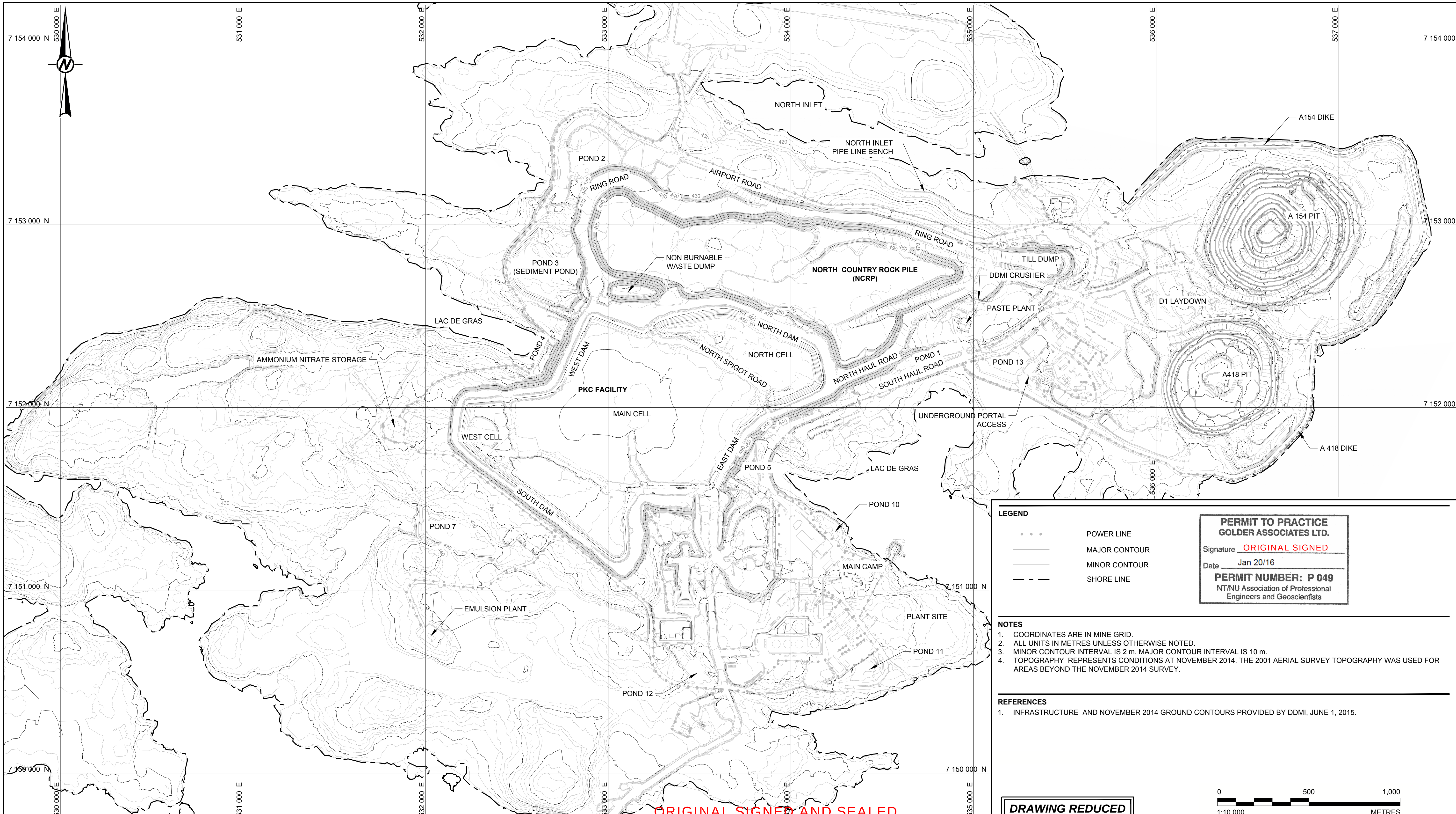

ORIGINAL SIGNED AND SEALED

LEGEND

(dotted line)	POWER LINE
(solid line)	MAJOR CONTOUR
(thin line)	MINOR CONTOUR
(dashed line)	SHORE LINE

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NOTES

1. COORDINATES ARE IN MINE GRID.
2. ALL UNITS IN METRES UNLESS OTHERWISE NOTED.
3. MINOR CONTOUR INTERVAL IS 2 m. MAJOR CONTOUR INTERVAL IS 10 m.
4. TOPOGRAPHY REPRESENTS CONDITIONS AT NOVEMBER 2014. THE 2001 AERIAL SURVEY TOPOGRAPHY WAS USED FOR AREAS BEYOND THE NOVEMBER 2014 SURVEY.

REFERENCES

1. INFRASTRUCTURE AND NOVEMBER 2014 GROUND CONTOURS PROVIDED BY DDMI, JUNE 1, 2015.

DRAWING REDUCED

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B	2015-11-24	ISSUED FOR CLIENT REVIEW	AMF	MGL	GP	BEW
A	2015-11-23	ISSUED FOR INTERNAL REVIEW	AMF	MGL	GP	BEW

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PROJECT
DIAVIK DIAMOND MINE
NORTH COUNTRY ROCK PILE CLOSURE

TITLE
DIAVIK DIAMOND MINE SITE PLAN

PROJECT NO.	PHASE / TASK	REV.	2 of 8	DRAWING
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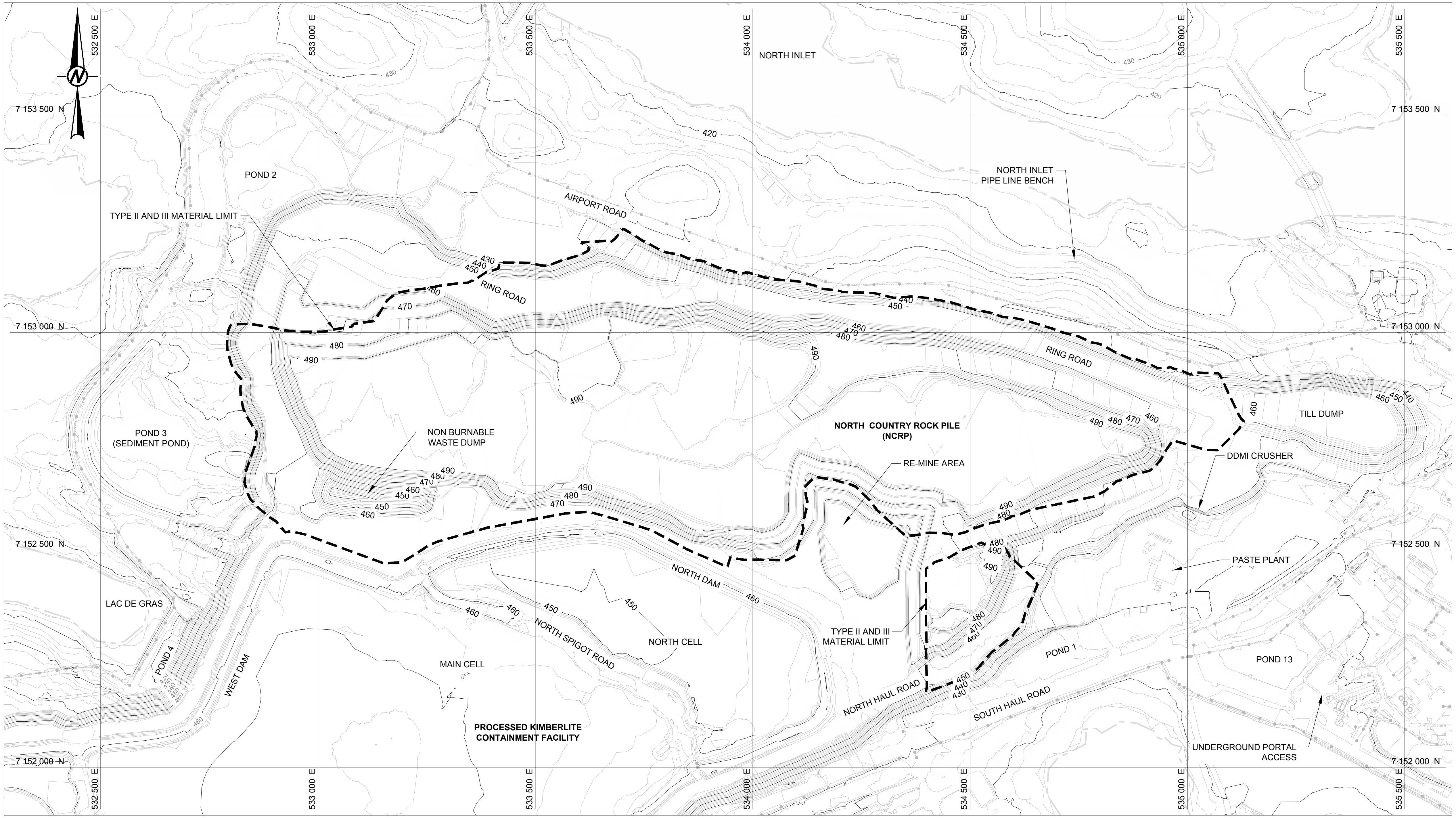

LEGEND

- POWER LINE
- MAJOR CONTOUR
- MINOR CONTOUR
- TYPE II AND III MATERIAL LIMIT
- SHORE LINE

NOTES

1. COORDINATES ARE IN MINE GRID.
2. ALL UNITS IN METRES UNLESS OTHERWISE NOTED.
3. MINOR CONTOUR INTERVAL IS 2 m. MAJOR CONTOUR INTERVAL IS 10 m.
4. SITE INFRASTRUCTURE PROVIDED BY DDMI AND IS SHOWN FOR INFORMATION PURPOSES ONLY.

REFERENCES

1. INFRASTRUCTURE AND NOVEMBER 2014 GROUND CONTOURS PROVIDED BY DDMI, DATE: JUNE 1, 2015, DOWNLOADED FROM DDMI FILE TRANSFER PROTOCOL (FTP) SITE.

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0 200 400
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B	2015-11-24	ISSUED FOR CLIENT REVIEW	AMF	MGL	GP	BEW
A	2015-11-23	ISSUED FOR INTERNAL REVIEW	AMF	MGL	GP	BEW

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PROJECT
DIAVIK DIAMOND MINE
NORTH COUNTRY ROCK PILE CLOSURE

TITLE
NORTH COUNTRY ROCK PILE - PLAN BEFORE RE-SLOPE

PROJECT NO.	PHASE / TASK	REV.	3 of 8	DRAWING
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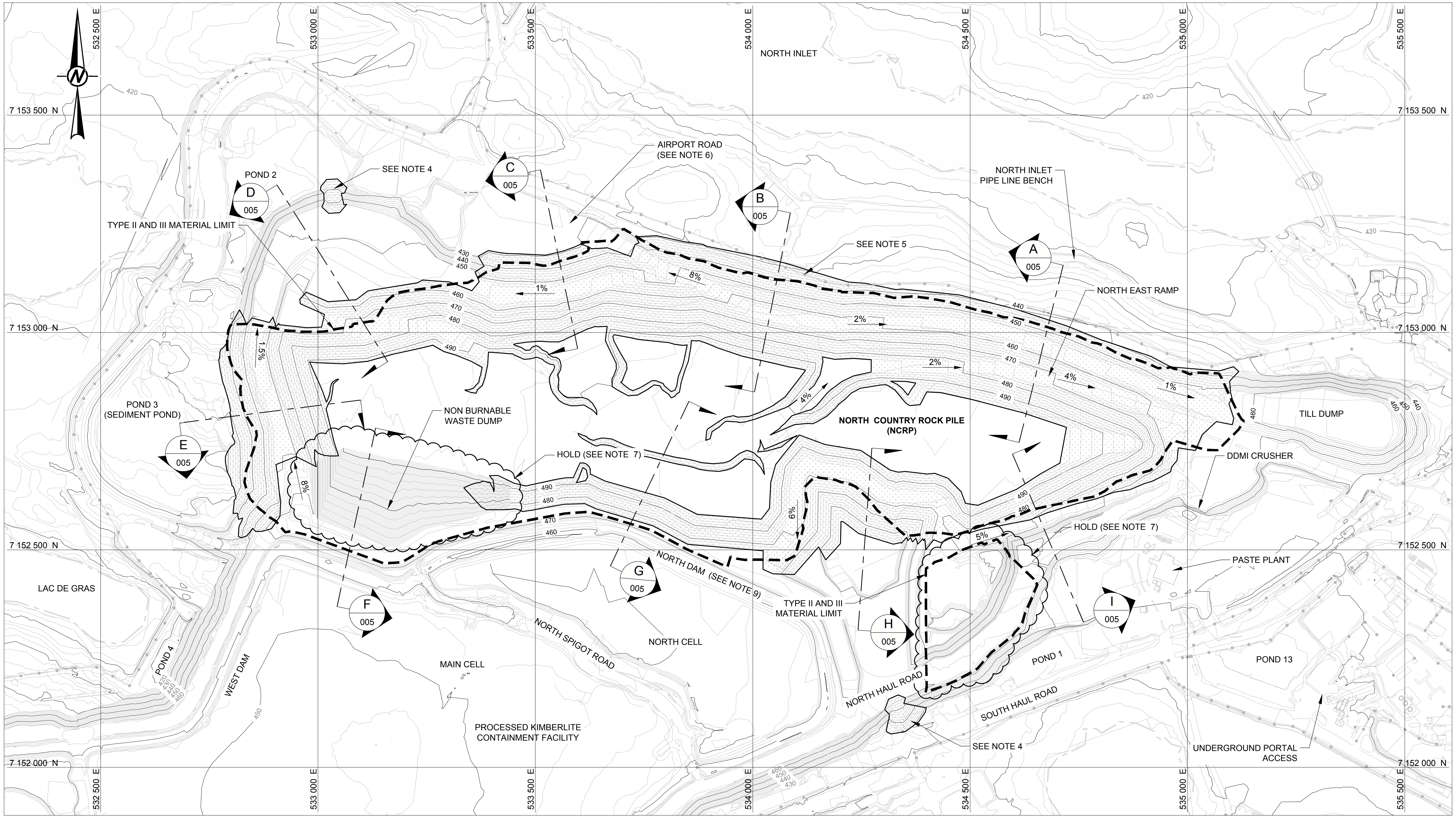

LEGEND

- RE-SLOPE
- POWER LINE
- MAJOR CONTOUR
- MINOR CONTOUR
- TYPE II AND III MATERIAL LIMIT
- SHORE LINE

NOTES

1. COORDINATES ARE IN MINE GRID.
2. ALL UNITS IN METRES UNLESS OTHERWISE NOTED.
3. MINOR CONTOUR INTERVAL IS 2 m. MAJOR CONTOUR INTERVAL IS 10 m.
4. CARIBOU EGRESS SPECIFIED BY DDMI (CONCEPT).
5. POWERLINE TO BE RE-LOCATED.
6. AIRPORT ROAD TO BE RE-LOCATED.
7. AREA FOR FUTURE CONSIDERATION.
8. SITE INFRASTRUCTURE PROVIDED BY DDMI AND IS SHOWN FOR INFORMATION PURPOSES ONLY.
9. PKC FACILITY DAM RAISE ABOVE ELEVATION 465 m TO BE CONFIRMED. TIE-IN TO PKC FACILITY LINER TO BE CONFIRMED.

REFERENCES

1. INFRASTRUCTURE AND NOVEMBER 2014 GROUND CONTOURS PROVIDED BY DDMI, JUNE 1, 2015.

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0 200 400
1:5,000 METRES

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B	2015-11-24	ISSUED FOR CLIENT REVIEW	AMF	MGL	GP	BEW
A	2015-11-23	ISSUED FOR INTERNAL REVIEW	AMF	MGL	GP	BEW

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VANCOUVER, BC
CANADA
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PROJECT
DIAVIK DIAMOND MINE
NORTH COUNTRY ROCK PILE CLOSURE

TITLE
NORTH COUNTRY ROCK PILE - AFTER RE-SLOPE

PROJECT NO.	PHASE / TASK	REV.		DRAWING
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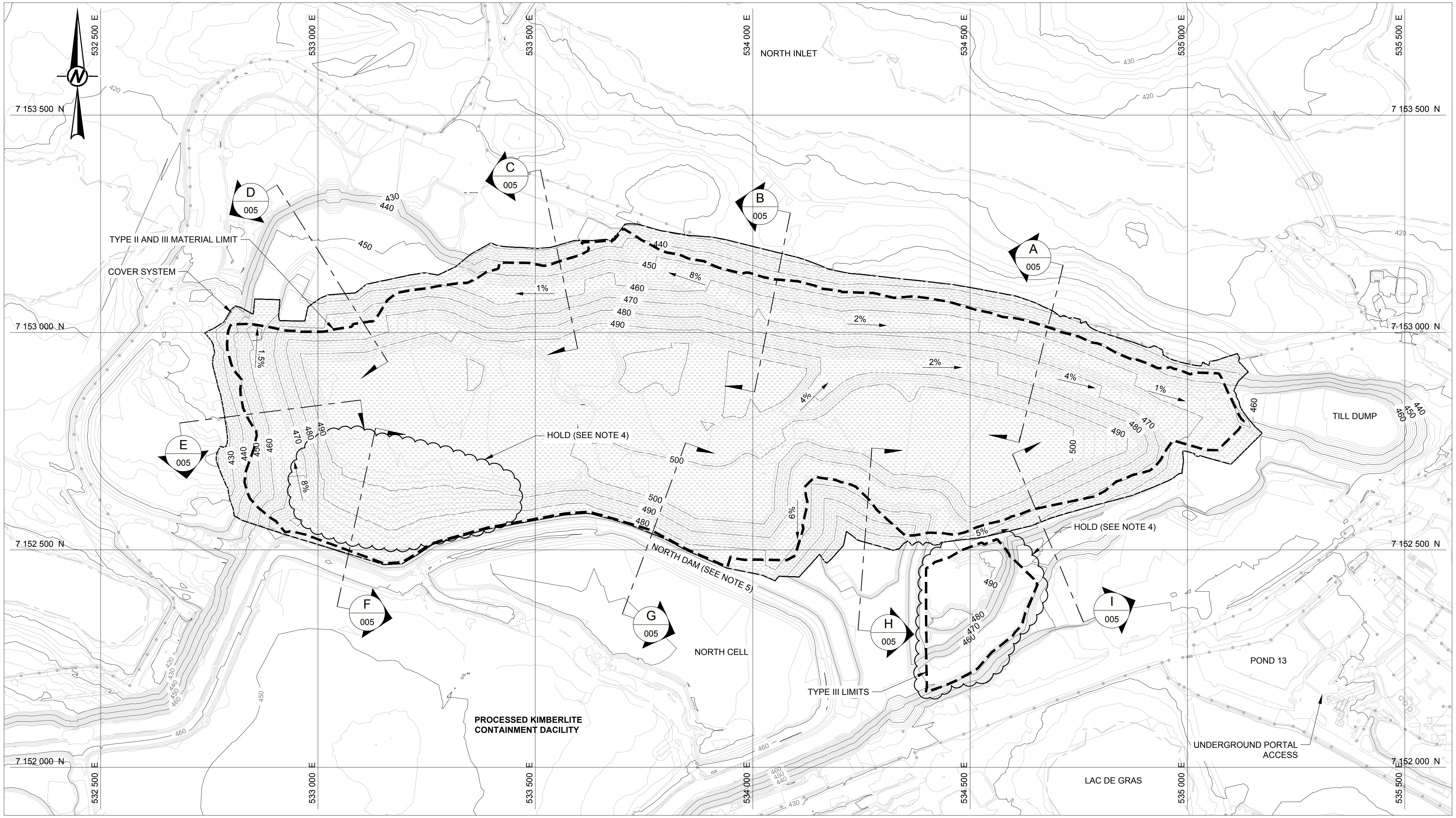

LEGEND

- COVER SYSTEM
- POWER LINE
- MAJOR CONTOUR
- MINOR CONTOUR
- TYPE II AND III MATERIAL LIMIT
- SHORE LINE

NOTES

1. COORDINATES ARE IN MINE GRID.
2. ALL UNITS IN METRES UNLESS OTHERWISE NOTED.
3. MINOR CONTOUR INTERVAL IS 2 m. MAJOR CONTOUR INTERVAL IS 10 m.
4. AREA FOR FUTURE CONSIDERATION.
5. PKC FACILITY DAM RAISE ABOVE ELEVATION 465 m TO BE CONFIRMED. TIE-IN TO PKC FACILITY LINER TO BE CONFIRMED.

REFERENCES

1. INFRASTRUCTURE AND NOVEMBER 2014 GROUND CONTOURS PROVIDED BY DDMI, JUNE 1, 2015.

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Date Jan 20/16

PERMIT NUMBER: P 049

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DRAWING REDUCED

0 200 400
1:5,000 METRES

REV.	YYYY-MM-DD	DESCRIPTION	DESIGNED	PREPARED	REVIEWED	APPROVED
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B	2015-11-24	ISSUED FOR CLIENT REVIEW	AMF	MGL	GP	BEW
A	2015-11-23	ISSUED FOR INTERNAL REVIEW	AMF	MGL	GP	BEW

SEAL

CLIENT
DIAVIK DIAMOND MINES (2012) INC.

CONSULTANT
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VANCOUVER
SUITE 200 - 2920 VIRTUAL WAY
VANCOUVER, BC
CANADA
[+1] (604) 296 4200
www.golder.com

PROJECT
DIAVIK DIAMOND MINE
NORTH COUNTRY ROCK PILE CLOSURE

TITLE
NORTH COUNTRY ROCK PILE - FINAL CONFIGURATION

PROJECT NO. 1521339 | PHASE / TASK 7000/7300 | REV. 0 | 6 of 8 | DRAWING 004

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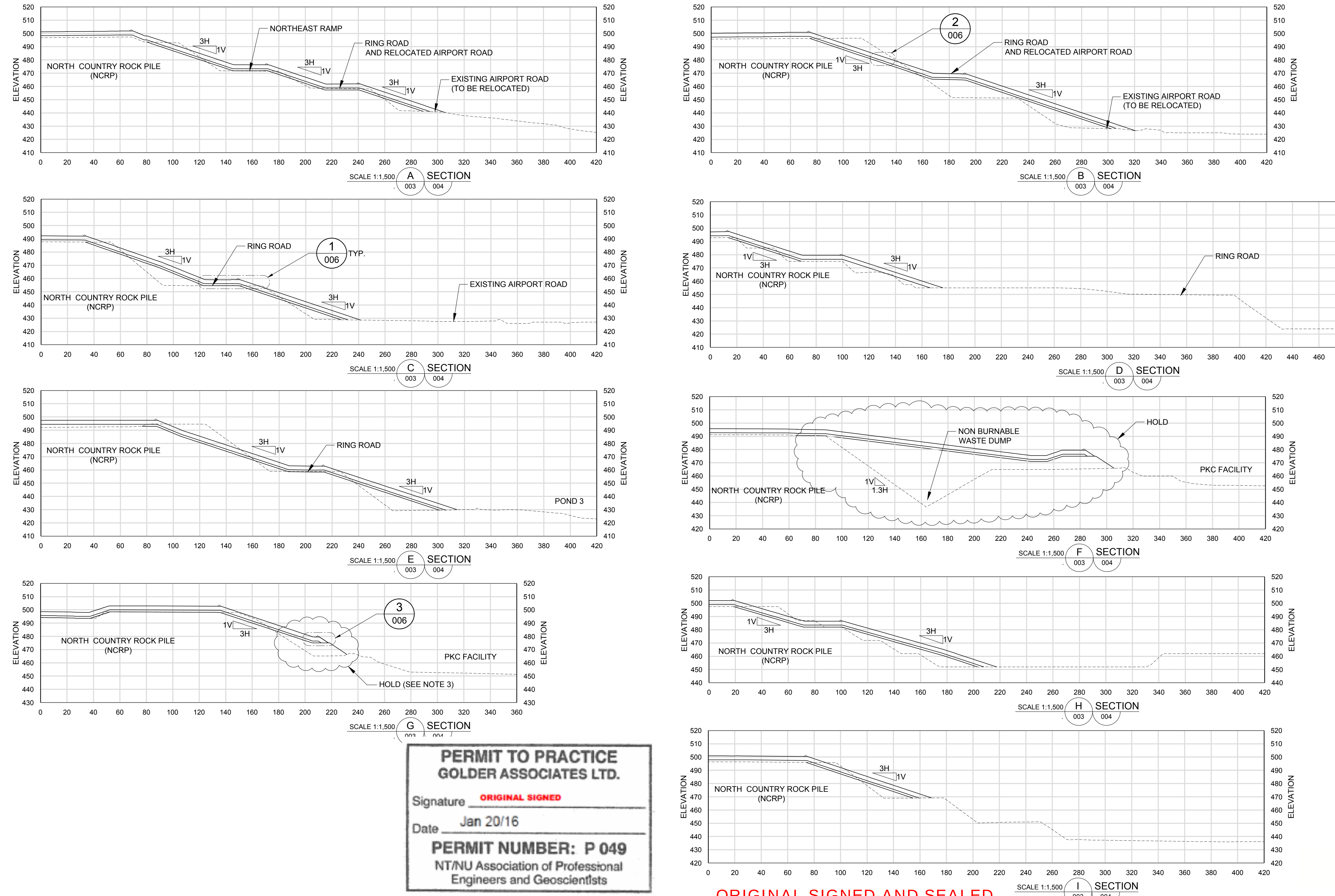

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GOLDER ASSOCIATES LTD.

Signature ORIGINAL SIGNED

Date Jan 20/16

PERMIT NUMBER: P 049

NT/NU Association of Professional Engineers and Geoscientists

ORIGINAL SIGNED AND SEALED

LEGEND

EXISTING GROUND (NOVEMBER 2014)

NOTES

1. COORDINATES ARE IN MINE GRID.
2. ALL UNITS IN METRES UNLESS OTHERWISE NOTED.
3. PKC FACILITY DAM RAISE ABOVE ELEVATION 465 m TO BE CONFIRMED. TIE-IN TO PKC FACILITY LINER TO BE CONFIRMED.

REFERENCES

1. NOVEMBER 2014 GROUND CONTOURS PROVIDED BY DDMI, JUNE 1, 2015.

DRAWING REDUCED

0 50 100

1:1,500 METRES

REV.	YYYY-MM-DD	DESCRIPTION	DESIGNED	PREPARED	REVIEWED	APPROVED
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B	2015-11-24	ISSUED FOR CLIENT REVIEW	AMF	MGL	GP	BEW
A	2015-11-23	ISSUED FOR INTERNAL REVIEW	AMF	MGL	GP	BEW

SEAL

CLIENT
DIAVIK DIAMOND MINES (2012) INC.

CONSULTANT
Golder Associates
VANCOUVER
SUITE 200 - 2920 VIRTUAL WAY
VANCOUVER, BC
CANADA
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www.golder.com

PROJECT
DIAVIK DIAMOND MINE
NORTH COUNTRY ROCK PILE CLOSURE

TITLE
NORTH COUNTRY ROCK PILE - CROSS SECTIONS

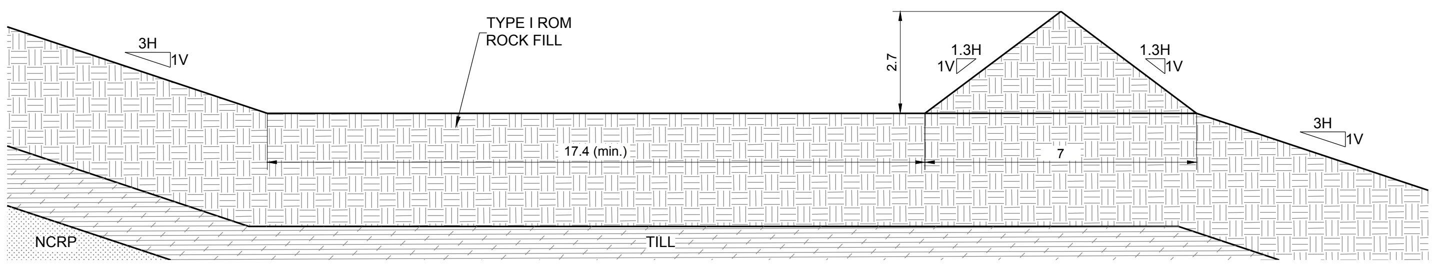
PROJECT NO.	PHASE / TASK	REV.		DRAWING
1521339	7000/7300	0	7 of 8	005

LEGEND

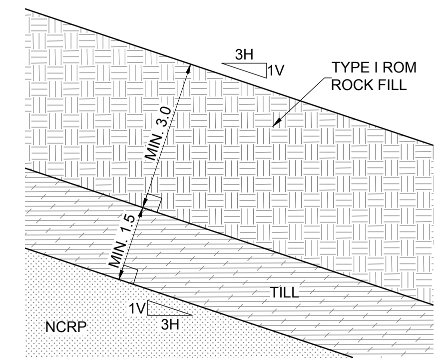
- TYPE I ROM ROCK FILL
- TILL
- NCRP
- TRANSITION
- 50 mm MINUS CRUSH
- COARSE PROCESSED KIMBERLITE
- ES2 COLETANCHE BITUMINOUS LINER

NOTES

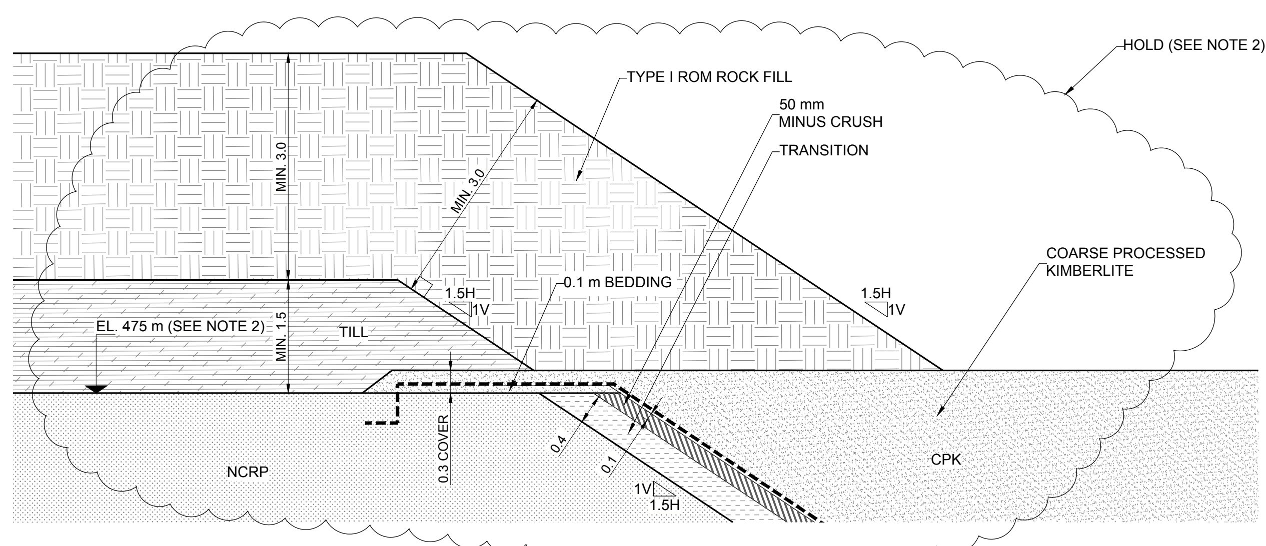
1. ALL UNITS IN METRES UNLESS OTHERWISE NOTED.
2. PKC FACILITY DAM RAISE ABOVE ELEVATION 465 m TO BE CONFIRMED. TIE-IN TO PKC FACILITY LINER TO BE CONFIRMED.



SCALE 1:100 1 / 005 TYPICAL HAUL ROAD



SCALE 1:75 2 / 005 DETAIL - COVER SYSTEM



SCALE 1:50 3 / 005 DETAIL - LINER TIE-IN

PERMIT TO PRACTICE
GOLDER ASSOCIATES LTD.
Signature ORIGINAL SIGNED
Date Jan 20/16
PERMIT NUMBER: P 049
NT/NU Association of Professional Engineers and Geoscientists

ORIGINAL SIGNED AND SEALED

DRAWING REDUCED

0 5 10
1:100 METRES

0 2.5 5
1:75 METRES

0 2 4
1:50 METRES

REV.	YYYY-MM-DD	DESCRIPTION	DESIGNED	PREPARED	REVIEWED	APPROVED
0	2015-01-20	ISSUED FOR CONSTRUCTION	AMF	MGL	GP	BEW
B	2015-11-24	ISSUED FOR CLIENT REVIEW	AMF	MGL	GP	BEW
A	2015-11-23	ISSUED FOR INTERNAL REVIEW	AMF	MGL	GP	BEW

SEAL

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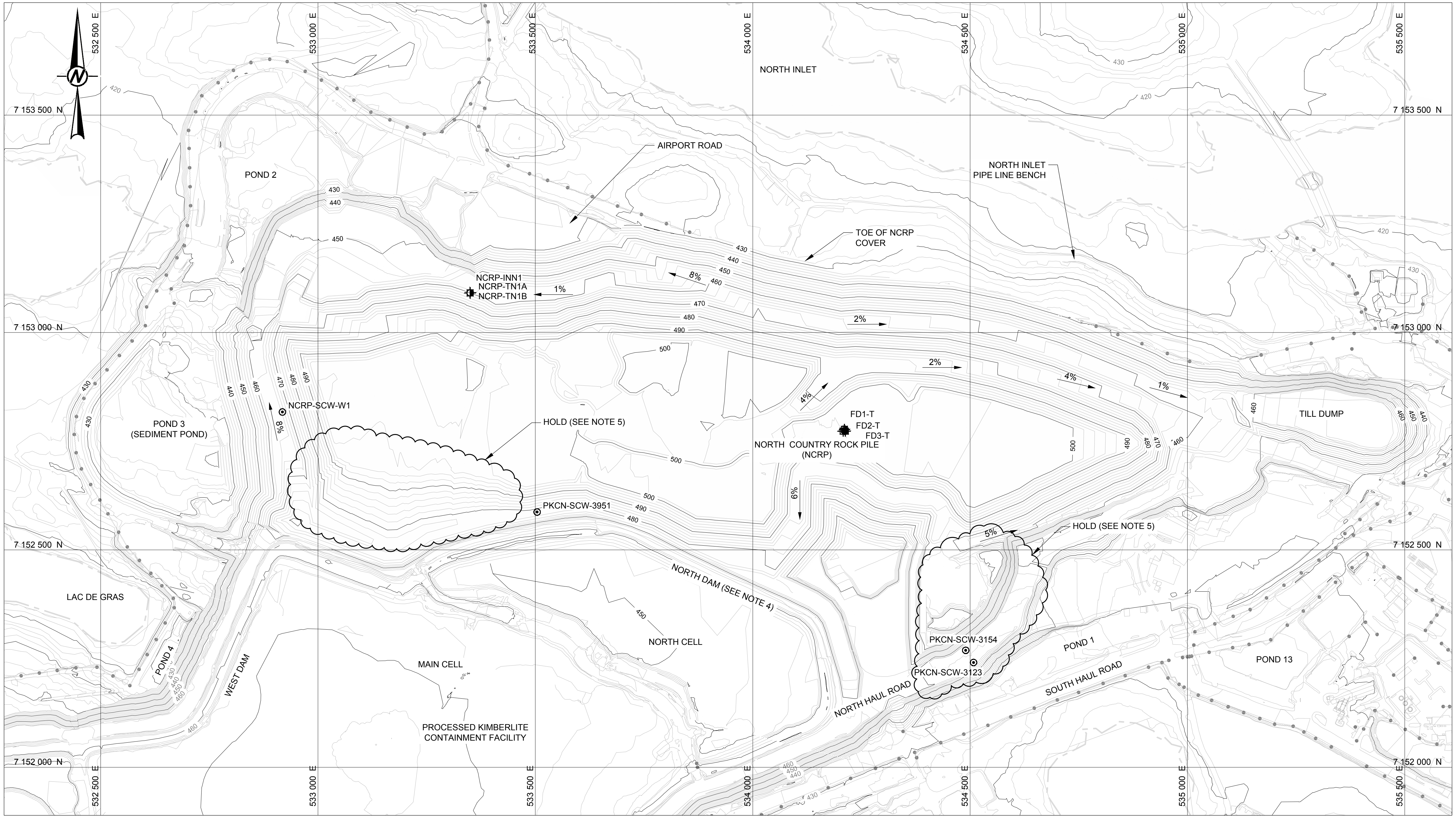
PROJECT
DIAVIK DIAMOND MINE
NORTH COUNTRY ROCK PILE CLOSURE

TITLE
NORTH COUNTRY ROCK PILE - DETAILS

PROJECT NO.	PHASE / TASK	REV.		DRAWING
1521339	7000/7300	0	5 of 8	006

Path: \\van1-v-fshare\proyectos\159\Abiertos\INGENIERIA\1592\1C1022 BWOs from Golder Canada\Informacion tecnica\CG-Civil\01_En Trabajo\01_DIAVIK_NCRP_CLOSURE\ | File Name: 1022-006.dwg

IF THIS MEASUREMENT DOES NOT MATCH WHAT IS SHOWN, THE SHEET SIZE HAS BEEN MODIFIED FROM: ANSI D
25 mm
0



LEGEND

- POWER LINE
- MAJOR CONTOUR
- MINOR CONTOUR
- SHORE LINE
- INCLINOMETER AND THERMISTOR LOCATION
- WELL LOCATION

NOTES

1. COORDINATES ARE IN MINE GRID.
2. ALL UNITS IN METRES UNLESS OTHERWISE NOTED.
3. MINOR CONTOUR INTERVAL IS 2 m. MAJOR CONTOUR INTERVAL IS 10 m.
4. PKC FACILITY DAM RAISE ABOVE ELEVATION 465 m TO BE CONFIRMED. TIE-IN TO PKC FACILITY LINER TO BE CONFIRMED.
5. AREA FOR FUTURE CONSIDERATION.

REFERENCES

1. INFRASTRUCTURE AND NOVEMBER 2014 GROUND CONTOURS PROVIDED BY DDMI, JUNE 1, 2015.

EXISTING WELL LOCATION TABLE

NAME	STRUCTURE	EASTING (m)	NORTHING (m)	DATE INSTALLED
NCRP-SCW-W1	NORTH COUNTRY ROCK PILE	532918	7152817	2013
PKCN-SCW-3951	NORTH DAM	533504	7152587	2013
PKCN-SCW-3123	NORTH DAM	534508	7152241	2013
PKCN-SCW-3154	NORTH DAM	534490	7152269	2013

EXISTING THERMISTOR LOCATION TABLE

NAME	STRUCTURE	EASTING (m)	NORTHING (m)	DATE INSTALLED
NCRP-TN1A	NORTH COUNTRY ROCK PILE	533350	7153091	2013
NCRP-TN1B	NORTH COUNTRY ROCK PILE	533350	7153091	2013
FD1-T	NORTH COUNTRY ROCK PILE	534211	7152777	2010
FD2-T	NORTH COUNTRY ROCK PILE	534208	7152773	2010
FD3-T	NORTH COUNTRY ROCK PILE	534213	7152773	2010

EXISTING INCLINOMETER LOCATION TABLE

NAME	STRUCTURE	EASTING (m)	NORTHING (m)	DATE INSTALLED
NCRP-INN1	NORTH COUNTRY ROCK PILE	533350	7153091	2013

ORIGINAL SIGNED AND SEALED

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Signature ORIGINAL SIGNED
Date Jan 20/16
PERMIT NUMBER: P 049
NT/NU Association of Professional Engineers and Geoscientists

DRAWING REDUCED

0 200 400
1:5,000 METRES

REV.	YYYY-MM-DD	DESCRIPTION	DESIGNED	PREPARED	REVIEWED	APPROVED
0	2015-01-20	ISSUED FOR CONSTRUCTION	AMF	MGL	GP	BEW
B	2015-11-24	ISSUED FOR CLIENT REVIEW	AMF	MGL	GP	BEW
A	2015-11-23	ISSUED FOR INTERNAL REVIEW	AMF	MGL	GP	BEW

SEAL

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PROJECT
DIAVIK DIAMOND MINE
NORTH COUNTRY ROCK PILE CLOSURE

TITLE
NORTH COUNTRY ROCK PILE - GEOTECHNICAL INSTRUMENTATION LAYOUT

PROJECT NO.	PHASE / TASK	REV.		DRAWING
	7000/7300	0	8 of 8	007

APPENDIX B

Technical Specifications

PREPARED FOR

DIAVIK DIAMOND MINES INC.

PREPARED BY:

PERMIT TO PRACTICE
GOLDER ASSOCIATES LTD.
Signature ORIGINAL SIGNED
Date March 29, 2017
PERMIT NUMBER: P 049
NT/NU Association of Professional Engineers and Geoscientists

Golder Associates

ORIGINAL SIGNED AND SEALED ORIGINAL SIGNED

2	29MAR17	ISSUED FOR CONSTRUCTION	BW	JC
1	06FEB17	ISSUED FOR REVIEW	GP	BW
0	20JAN16	ISSUED FOR CONSTRUCTION	GP	BW
C	15JAN16	ISSUED FOR REVIEW	GP	BW
B	22DEC15	ISSUED FOR REVIEW	GP	BW
REV.	DATE	REASON FOR REVISION	CHK	APP

TABLE OF CONTENTS

SECTION PAGE

LIST OF TABLES

LIST OF FIGURES

1.0 GENERAL

These Technical Specifications provide the technical information related to physical stability for the North Country Rock Pile (NCRP) closure construction at the Diavik Diamond Mines (2012) Inc.'s (DDMI) Diavik Mine in the Northwest Territories of Canada. These Technical Specifications do not cover technical information to fulfill the geochemistry, seepage, surface water management, and environmental requirements of the design.

These Technical Specifications shall be read and interpreted in combination with the latest revision of the Drawings. Table 1-1 presents a summary of the current Drawings. The user is responsible to confirm that they are using the most recent version of the Technical Specifications and Drawings.

Table 1-1: Drawings

Drawing Number	Drawing Title	Revision
000	Drawing Title Sheet	1
001	Diavik Diamond Mine Site Plan	0
002	North Country Rock Pile – Plan Before Re-Slope	0
003	North Country Rock Pile – After Re-Slope	0
004	North Country Rock Pile – Final Configuration	0
005	North Country Rock Pile – Cross-Sections	0
006	North Country Rock Pile – Details	0
007	North Country Rock Pile – Geotechnical Instrumentation Layout	0

Where a discrepancy exists between these Technical Specifications and the Drawings, the Project Engineer and or QA Manager shall be notified upon discovery of the discrepancy, and a formal written Request for Information (RFI) shall be issued to the Project Engineer and or QA Manager to clarify the design intent.

1.1 Definitions

Table 1-2: List of Definitions

Owner	Diavik Diamond Mines (2012) Inc.
Approval	A written engineering or geotechnical opinion, concerning the progress and completion of the work.
Quality Assurance	Planned and systematic activities that provide adequate confidence to the Owner (DDMI) and various stakeholders that quality control is being implemented effectively.
Quality Control	A planned system of inspection and testing carried out according to accepted standard specifications to ensure the quality of work meets the design requirements.
Type I ROM Rockfill	Non-acid generating run-of-mine rockfill, meeting the design specifications.
Till	Unfrozen till soil meeting the design specifications. To be placed as cover over the re-sloped surface and crest of the NCRP.
Waste Rock	Rockfill excavated during the development and operation of the open pits and underground pits and placed within the NCRP.
Fill	A general terminology to describe soil or rockfill materials used for construction.
Drawings	The most recent issued version of the Drawings prepared by the Engineer for the NCRP Closure Construction.
Specifications	NCRP Closure Construction Technical Specifications, prepared by the Engineer.
Work	The construction works required to complete the NCRP Closure as defined by the DDMI Construction Manager's scope of work and Engineers Drawings and Specifications.

1.2 Standards and Codes

Work shall conform to, but not be limited to, the requirements of the latest editions of the following standards and codes which are part of the Specifications (Table 1-3).

Table 1-3: List of Standards

Standard	Description
ASTM D422	Standard Test Method for Particle-Size Analysis of Soils
ASTM D2216	Standard Test Method for Laboratory Determination of Water (Moisture) Content of Soil and Rock by Mass
	Mine Health and Safety Act of the Northwest Territories
	Mine Health and Safety Regulations of the Northwest Territories

1.3 Roles and Responsibilities

Figure 1 presents the project organization chart, including key communication links between the DDMI Construction Management (CM) team and the engineer. Table 1-4 presents a summary of the key roles, responsibility and authorities of the key positions included in the organizational chart. The roles and responsibilities shall be reviewed, documented and agreed upon prior to the start of work for each stage of the project.

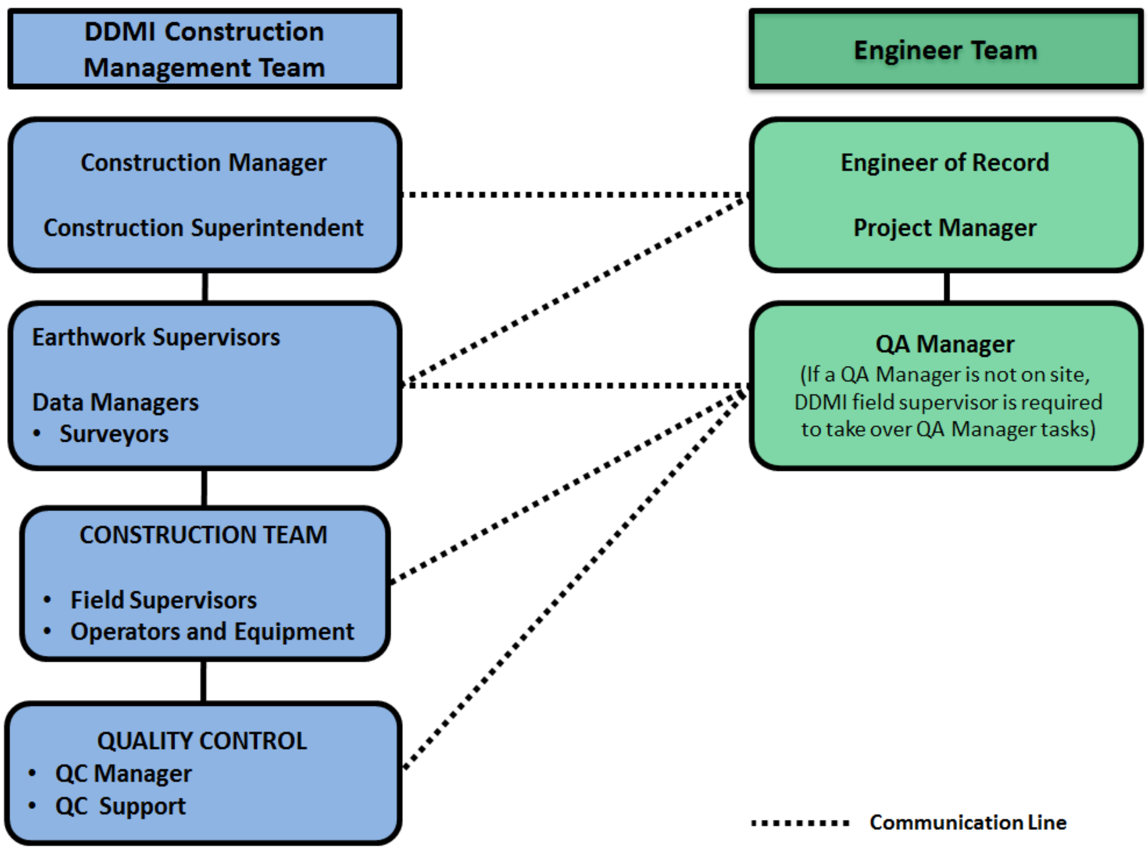

Figure 1: North Country Rock Pile Closure Construction Project Organization Chart

Table 1-4: Roles and Responsibilities

Role	Responsibilities	Authority
DDMI Construction Management (CM) Team		
DDMI Construction Manager	▪ Defines the scope of the project. ▪ Responsible for providing items noted in the contract package as being supplied by DDMI, obtaining all relevant permits, and providing reasonable access to the general open areas surrounding the work site.	▪ Represents the Owner, DDMI. ▪ Retains the DDMI Construction Management Team and the Engineer to complete the project.
DDMI Construction Superintendent	▪ The DDMI Construction Superintendent shall represent the DDMI Construction Manager on site. ▪ The DDMI Construction Superintendent is responsible for coordinating all project communications, arranging daily and weekly meetings, as required, and holding problem resolution meetings for QC and QA issues.	▪ Directs scope of work to complete the project. ▪ Can stop work because of non-compliance, which shall resume only after a plan for corrective action has been prepared and has been approved by the QC Manager and QA Manager.
DDMI Earthworks Supervisors	▪ Reports to the DDMI Construction Superintendent on site and liaises directly with the DDMI Field Supervisors, QC Manager and QA Manager.	▪ Assistance in resolving QC issues.
DDMI Field Supervisors	▪ Retained by the DDMI Construction Manager and will be assigned tasks by the DDMI CM Team to bring the NCRP Closure Construction works or additional work, as requested by the DDMI Construction Manager, to final completion. ▪ Responsible for proper construction and to ensure that the works are constructed in accordance with the Drawings and Specifications. ▪ Shall follow the Quality Control Plan as directed by QC Manager or designate by the CM team, to ensure the quality of the construction works it undertakes. ▪ Reports to the DDMI CM Team. ▪ Request information, as required, to clarify the design intent.	▪ Directs work.

Role	Responsibilities	Authority
Quality Control Manager	▪ Qualified Geotechnical Engineer or Technician. ▪ Represents DDMI Construction Team in quality control to ensure the Work meets the requirements of the Drawings and Specifications. ▪ Responsible for QC tasks outlined in Section 5. ▪ Responsible for material testing on site. ▪ Preparation of the required construction checklists. ▪ Reports to the DDMI CM Team.	▪ Directs work with respect to quality such that the work is carried out to meet the requirements of the Drawings and Specifications. ▪ Approves work conducted in accordance with the Drawings and Specifications. ▪ Signs as the site QC Manager on the construction checklists. ▪ Rejects work which fails to meet the design intent of the Drawings and Specifications. ▪ Reports non-conformance of work to the QA Manager, and DDMI Construction Superintendent, and resolves QC issues.
Engineer Team		
Engineer of Record (Engineer)	▪ Ensures that the works are constructed in accordance with the design intent. ▪ Conducts site visits to check on progress of work and support the QA team. ▪ Responsible to oversee the Quality Assurance program to confirm the works are constructed in accordance with the design intent. ▪ Responsible for the design and preparation of the Drawings and Specifications. ▪ Preparation of construction record report from information provided by the DDMI Construction Superintendent. ▪ Responsible for all design and specification changes, modifications, or clarifications required during construction.	▪ Overall supervision and coordination of QA team. ▪ Signs construction record.

Role	Responsibilities	Authority
Quality Assurance Manager	▪ Reports to the Engineer, and represents the Engineer on site. ▪ Conducts site visits to check on progress of work. ▪ The QA Manager shall be responsible for performing the QA tasks outlined in Section 5, including collection of QA samples. ▪ Shall be responsible to monitor the Quality Control program to confirm the works are constructed in accordance with the design intent. ▪ Review of QC data and sign-offs provided for compliance with design intent, Drawings and Specifications. ▪ Attends site meetings and site inspections with the CM Team when necessary. ▪ Assesses site design/construction issues or non-compliances and prepares recommendations for review by the Engineer. ▪ Prepares draft site documentation for review by the Engineer.	▪ Approves work conducted in accordance with the Drawings and Specifications. ▪ Signs as the site QA Manager on the construction checklists. ▪ Rejects work which fails to meet the design intent laid out on the Drawings and Specifications. ▪ Reports non-conformance of Work to the DDMI CM team and the Engineer.

2.0 CARE OF WATER

This Section describes Care of Water during the construction, which consists of all work required to control water from any sources, including groundwater, surface water, snowmelt and precipitation, in order to complete the Work in accordance with the Drawings and Specifications.

Surface water shall be temporarily diverted and managed during construction of the Work. Appropriate channel, ditch, dike, and other facilities required to divert surface water from any area required to complete the Work shall be constructed.

Temporary diversion and protective works and pumping stations shall be adequately operated and maintained. These shall also be readily accessible at all times. Temporary dikes and other temporary works shall be removed when they are no longer required.

3.0 EARTHWORKS SPECIFICATIONS

3.1 General

This section defines the requirements for the NCRP re-slope and fill placement according to the lines and grades shown on the Drawings.

Prior to starting any NCRP closure earthworks, a photographic record of the existing conditions of the NCRP shall be compiled. Use of drones for aerial survey is allowed.

3.2 Scope of Work

The scope of work as defined by the Drawings and Specifications is as follows:

- re-sloping of NCRP slopes to 3 horizontal to 1 vertical (3H:1V) slopes, or flatter, to the limits defined in the Drawings;
- placement of the cover system consisting of Till and Type I ROM Rockfill; and
- geotechnical instrumentation raises.

Quality control and quality assurance shall be completed for the placement of the cover system, as described in Section 5.

3.3 Materials

3.3.1 Waste Rock

Waste rock in different areas of the NCRP is categorized based on acid generating potential:

- Type I waste rock (<0.04% sulphur) is clean rock, composed of granite and pegmatitic granite, and is considered to have no acid generating potential.
- Type II waste rock (0.04 to 0.08% sulphur) is predominantly granite with a minor amount of biotite schist, and is considered to have no or low acid generating potential.
- Type III waste rock (>0.08% sulphur) is a mixture of rock with a greater proportion of biotite schist and is considered to be potentially acid generating.

3.3.2 Till

Till will be used to cover the Type II and Type III waste rock as shown in the Drawings.

The Till to be placed on the re-sloped and crest surfaces of the NCRP shall be a silt, sand and gravel mixture with a maximum particle size of 1.5 m or lift thickness, with 30 to 70% passing the No. 40 sieve (0.42 mm). The maximum water content of placed till is 25%.

Till used in construction of the cover shall be unfrozen. Till is to be free of organic matter, debris, cinders, ash, refuse, snow, ice and other unsuitable material.

Till is to be kept well graded during loading, transport, stockpiling and placement.

3.3.3 Type I ROM Rockfill

Type I ROM Rockfill is required for capping of the Till layer.

The Type I ROM Rockfill will be sound, hard, durable Type I waste rock free from organic matter and debris and other unsuitable material.

Snow, ice and frozen chunks within the Type I ROM Rockfill shall be avoided. Areas with significant quantities of snow, ice or frozen chunks may require removal and replacement, as determined by the QC Manager and the QA Manager.

3.4 Fill Placement

3.5.1 General

The placement of fill material includes loading, transporting, unloading, storing, and additional handling when necessary.

Fill placement shall be to the lines and grades shown on the Drawings and in accordance with the Specifications.

Fill materials shall not be placed until the foundation or surface has been inspected and approved by the QC Manager and QA Manager.

Structures shall be constructed to the lines, grades and cross-sections shown on the Drawings using only suitable materials as defined within the Specifications and/or approved by the QA Manager.

Equipment suitability, methods of working and quality of work shall be demonstrated during the initial stages of the work or at such time as requested by the DDMI Construction Superintendent or the QA Manager. In the event that the work performance is unsatisfactory, changes shall be implemented to ensure the required quality.

During placement and spreading, unsuitable materials shall be removed.

Materials shall be transported, dumped and spread to limit segregation to the extent possible, so that each zone is homogeneous, free of stratification, lenses, pockets, ruts or layers of material of different texture or grading not conforming to the requirements specified for the material of each zone.

Fill material shall not be placed on a surface where an excess of ice, snow or water has accumulated. Measures shall be taken to minimize collection of water, snow, ice or other unsuitable material(s) on the surface of the fill or foundations. If these materials collect on the surface remove them prior to placement of subsequent lifts of material.

The thickness of placed layers of fill shall be measured perpendicular to the slope, unless stated otherwise.

3.5.2 Waste Rock Re-slope

The side slopes of the NCRP are to be re-sloped from the current angle of repose to 3H:1V, or less, within the limits shown on the Drawings.

Prior to re-slope, snow, ice or other deleterious materials shall be removed to the extent possible. Accumulations of snow or ice at the toe of the NCRP side slopes shall be removed prior to re-slope. Trapping of snow and ice during re-sloping shall be avoided.

Snow shall not be allowed to build up in areas of active re-sloping to the extent where snow could potentially support Waste Rock particles.

3.5.3 Till

Prior to Till placement, significant accumulations of water, snow, ice, or other unsuitable materials shall be removed from the approved foundation surface, which includes the re-sloped NCRP surface.

The QC Manager shall prepare and sign the re-sloped Waste Rock construction checklist, and the QA Manager shall review and sign the checklist prior to placement of Till.

Till shall be placed in a layer with a minimum thickness of 1.5 m (measured perpendicular to the slope) on the approved surface.

The Till may be nominally compacted by the dozer during spreading of the Till.

If the Till is left exposed, the surface shall be graded to manage surface water and limit erosion. The Till surface shall be smoothed if erosion gullies larger than 0.3 m develop. The Till layer shall be covered with Type I ROM if erosion gullies larger than 0.5 m develop. The Till layer shall be rebuilt to a minimum 1.5 m thickness prior to covering with Type I ROM.

3.5.4 Type I ROM Rockfill

Prior to Type I ROM Rockfill placement, accumulations of water, snow, ice, or other deleterious material shall be removed from the approved Till surface. When the approved Till surface has been left exposed for more than 2 months, the Till surface shall be re-inspected and re-approved by the QC Manager and QA Manager prior to Type I ROM Rockfill placement. Additional Till shall be placed if erosion, downslope movement, or other mechanism decreases the Till layer thickness to less than 1.5 m.

The QC Manager shall prepare and sign the Till placement construction checklist, and the QA Manager shall review and sign the checklist prior to placement of Type I ROM Rockfill.

The Type I ROM Rockfill may be nominally compacted by the dozer during placement of the Type I ROM Rockfill.

The minimum thickness of Type 1 ROM Rockfill cover is 3 m (measured perpendicularly to the slope) on the approved surface.

3.5 Borrow and Stockpile Sources

The suitability of the Till and Type I ROM Rockfill placed as cover materials shall be monitored as part of the QC Plan with the QC Manager to confirm that materials meet the requirements of the Specifications prior to use. The quality control will be checked by the QA Manager for conformance with the Specifications.

Till and Type I ROM Rockfill may be hauled to the NCRP directly from the A21 open pit operation, or from stockpiles.

4.0 GEOTECHNICAL INSTRUMENTATION

Table 4-1 presents a summary of the existing geotechnical instrumentation that requires raising as part of the NCRP closure construction. All geotechnical instruments can be raised vertically through the cover layers of the Till and Type I ROM Rockfill. The location of the existing geotechnical instrumentation is shown on Drawing 007, North Country Rock Pile – Geotechnical Instrumentation Layout.

Table 4-1: Existing Geotechnical Instrumentation Summary

Instrument Type	Instrument ID	Easting (m)	Northing (m)
Observation Well	NCRP-SCW-W1	532918	7152817
Observation Well	PKCN-SCW-3951	533504	7152587
Collection Well	PKCN-SCW-3123	534508	7152241
Observation Well	PKCN-SCW-3154	534490	7152269
Thermistor	NCRP-TN1A	533350	7153091
Thermistor	NCRP-TN1B	533350	7153091
Thermistor	FD1-T	534211	7152777
Thermistor	FD2-T	534208	7152773
Thermistor	FD3-T	534213	7152773
Inclinometer	NCRP-INN1	533350	7153091

5.0 QUALITY CONTROL AND QUALITY ASSURANCE PLAN

5.1 General

This Section of the Specification defines the requirements for a Quality Control (QC) and Quality Assurance (QA) Plan for the NCRP closure construction.

This QC and QA Plan includes:

- QC requirements;
- QA requirements;
- site inspections and testing requirements;
- construction record information requirements; and
- construction checklists (sign-off sheets) for the major construction activities.

5.1.1 QC Requirements

A QC Plan shall be developed for each stage of work and agreed upon by the CM team. Appropriate QC during the construction shall include:

- an experienced and qualified QC Manager, who is a qualified Geotechnical Engineer, Technician or designate by the CM team;
- QC inspection and testing to ensure compliance with the Drawings and Specifications, which includes visual inspection of layer thicknesses, surface conditions, surface erosion and fill material gradations, and testing for material gradation and water content of Till;
- QC review of as-built survey to ensure compliance with the Drawings, which includes lift thickness, elevation and layout; and
- a suitable laboratory facility location and equipment for performing the required testing.

The QC Manager monitors construction activities through the QC Plan such that the Work meets the requirements of the Drawings and Specifications. The QC Manager prepares the required construction checklists, and signs to approve that the Work has been completed in accordance with the Drawings and Specifications.

The QC Manager rejects Work which fails to meet the design intent of the Drawings and Specifications, reports any non-conformance of work to the QA Manager, and DDMI Construction Superintendent, and resolves QC issues.

5.1.2 QA Requirements

A QA Manager shall make periodic trips to site timed to inspect completed NCRP re-slope surfaces, during placement of Till and Type I ROM Rockfill layers of the cover, and to inspect placed Till surfaces. When the QA Manager is not on site, a DDMI representative is required to take over the role of the QA Manager and report observations to the Engineer.

The QA Manager shall carry out planned and systematic activities that provide adequate confidence to the Owner (DDMI) and various stakeholders that quality control is being implemented effectively such that construction is in accordance with the Drawings and Specifications.

The Engineer is responsible for all changes to the design. As the Engineer's representative, the QA Manager shall review any deviations from the design, including field changes or adjustments, to confirm conformance with the design intent. Any such modifications are subject to approval by the Engineer or the Engineer's representative.

QA inspection and testing shall be carried out by the QA Manager to confirm the QC program is ensuring the work meets the design intent, as set out in the Drawings and the Specifications, including layer thicknesses, surface conditions, surface erosion, material gradations, and water content of Till material.

Survey QA to confirm construction grades and limits will be coordinated by the DDMI Construction Superintendent. The QA Manager may request QA survey at any time.

5.2 QC/QA Requirements for Construction Activities

The QC Manager is responsible for compiling a photographic record of the conditions at the NCRP prior to starting construction, during intermediate stages of construction, and immediately after construction. Use of drones and aerial photography is allowed. The criteria and responsibilities of both the QC Manager and the QA Manager for the construction activities are listed in Table 5-1.

Table 5-1: QA/QC Responsibilities for Construction Activities

Activities	Responsibility	
	QC Manager	**QA Manager**
Survey	▪ Confirm as-built survey has been completed by DDMI and matches the Drawings. ▪ Determine need for adjustments in field. ▪ Provide protection of the survey set out stakes.	▪ Review survey alignments, elevation and layout, provided by surveyor. ▪ Confirm as-built survey has been completed by DDMI and matches the Drawings. ▪ Approve field adjustments.
Construction Materials	▪ Ensure materials meet the Specifications. ▪ Perform visual inspection of all construction material. ▪ Ensure laboratory equipment is calibrated. ▪ Have all applicable ASTM testing standards easily available. ▪ Perform, document and make available to the QA Manager the testing results as required in these Specifications. ▪ Provide photographic records.	▪ Verify QC equipment is calibrated and testing is to standards. ▪ Perform visual inspection of the materials to confirm QC program is controlling material. ▪ Perform QA sampling and testing to confirm QC testing. ▪ Review testing results. ▪ Provide photographic records.
Waste Rock Re-Slope	▪ Inspect the slope and specify removal methods for unsuitable materials as defined in the Specifications prior to re-sloping. ▪ Monitor waste rock re-slope activities. ▪ Perform regular inspections during re-sloping to ensure extent are as per the Drawings. ▪ Visual inspection of final re-slope surface. ▪ Ensure re-slope is approved and checklist signed-off. ▪ Provide photographic record.	▪ Inspect and approve removal of unsuitable materials prior to re-sloping. ▪ Visual inspection of final re-slope surface. ▪ Provide photographic record. ▪ Approve re-slope, and sign checklist.

Activities	Responsibility	
	QC Manager	**QA Manager**
Fill Placement	▪ Monitor fill material quality activities. ▪ Perform regular inspections during placement. ▪ Ensure placement and extent are as per the Drawings. ▪ Ensure fill lift thickness meets the Specifications. ▪ Ensure water content of the till meets the Specifications. ▪ Visual inspection of final lift surface. ▪ Ensure layer is approved and checklist signed-off. ▪ Provide photographic record.	▪ Monitor fill material quality to confirm Specifications are being met. ▪ Visual inspection of final lift surface. ▪ Perform checks of lift thickness. ▪ Review QC testing results and reports. ▪ Provide photographic record. ▪ Approve placement, and sign checklist.
Geotechnical Instrumentation Raising	▪ Prepare geotechnical instrumentation raise plan. ▪ Protect existing geotechnical instrumentation where required. ▪ Identify and clearly signpost geotechnical instrumentation locations. ▪ Coordinate and provide support for instrument raises to be completed by DDMI staff. ▪ Coordinate raises and splicing requirement through DDMI geotechnical department. ▪ Provide photographic record.	▪ Confirm that the geotechnical instrumentation has been sufficiently protected and signposted. ▪ Monitor geotechnical instrumentation raises. ▪ Provide photographic record.

5.3 Quality Control and Quality Assurance Requirements

5.3.1 Fill Material

The QC and QA testing requirements and frequency are listed in Table 5-2.

The QC Manager shall, at a minimum, perform all the required testing to document the construction quality. The material QC testing listed in Table 5-2 shall be completed on Till samples taken from placed material on the slope to evaluate suitability of Till.

The QA Manager will conduct independent sampling and testing to confirm the QC testing results. Samples may be requested to be taken from placed material on the slope or from borrow sources, or stockpiles.

Table 5-2: Construction QC and QA Testing

Material	Testing		
	Type of Testing	**QC Frequency**	**QA Frequency**
Till	Visual Gradation	Continuously	Periodically
	Gradation	1 every 50,000 m³	1 every 10 QC tests
	Water Content	1 every 50,000 m³	1 every 10 QC tests
	Lift Thickness	Measured continuously and survey data reviewed periodically	Measured periodically and survey data reviewed periodically
Type I ROM Rockfill	Visual Gradation	Continuously	Periodically
	Lift Thickness	Measured continuously	Measured periodically

5.3.2 Quality Control Documentation and Reporting

The QC Manager shall prepare and submit daily a Daily Field Construction Summary Report, in electronic and hardcopy formats, which at a minimum shall include the following:

- summary of work completed, including photographic record;
- summary of testing and measurements with details (coordinates, elevation, lift) of locations of acquired samples;
- record of problems/issues and resolutions;
- record of documents issued; and
- record of weather conditions.

A copy of this information shall be provided to the DDMI Construction Superintendent and QA Manager for approval.

5.4 Construction Record Report

Upon completion of construction activities at the end of each calendar year, DDMI and the Engineer shall prepare a Construction Record summary report.

The Construction Record summary report shall include, but not be limited to, the following information:

- daily construction reports;
- all testing records including a summary of all test sample locations and test results;
- photographic documentation of conditions prior to construction, construction works, and post construction conditions;
- summary of construction problems and resolutions; and
- completed construction checklists.

The DDMI CM team shall provide as-built survey of surfaces of materials placed and as-built construction quantities.

Upon completion of the NCRP closure construction, a single comprehensive NCRP closure construction record report will be prepared by the CM team and Engineer for submission to the regulators by DDMI.

5.5 Construction Checklists

Construction checklist forms to be completed by the QC Manager and approved by the QA Manager for documenting the quality control and other aspects of the construction activities are attached by work area which includes:

- NCRP Closure Construction Checklist – NCRP Re-slope; and
- NCRP Closure Construction Checklist – Fill Placement.

The checklists are to be signed by all parties prior to acceptance of each lift, and before the following lift can be placed. The construction checklist shall be re-signed by all parties prior to placement of Type I ROM if the Till surface has been left exposed for more than 2 months.

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NCRP CLOSURE CONSTRUCTION CHECKLIST- NCRP RE-SLOPE

CONTRACTOR :		DATE :	SHIFT: day - night
LOCATION	From :	To :	
NO.	**ITEMS TO BE INSPECTED**	**INSPECTED BY QC MANAGER**	**INSPECTED BY QA MANAGER**
1.	Survey lines and layout conform to the Drawings (survey by DDMI)		
2.	Accumulations of snow, ice and unsuitable materials removed prior to re-slope		
3.	Required visual inspection of materials performed		
4.	As-built survey conducted (survey by DDMI)		

REMARKS :

DEVIATIONS : (Attach list if necessary)

DATE OF RECTIFICATION :

ACCEPTED BY: QC Manager	ACCEPTED BY: QA Manager
NAME:	NAME:
SIGNATURE :	SIGNATURE :
DATE :	DATE :

NCRP CLOSURE CONSTRUCTION CHECKLIST- FILL PLACEMENT

CONTRACTOR :		DATE :	SHIFT (day/night):
LOCATION : From :		To :	
MATERIAL TYPE:			
NO.	**ITEMS TO BE INSPECTED**	**INSPECTED BY QC MANAGER**	**INSPECTED BY QA MANAGER**
1.	Survey lines and layout conform to the Drawings (survey by DDMI)		
2.	Required visual inspection of stockpiled materials performed		
3.	Required soil tests performed		
4	Water content of fill materials meet the Specification requirements		
5.	Fill materials meet the Specification requirements		
6.	Lift thickness meets the Specification		
7.	Dewatering measures provided if required		
8.	Required visual inspection of placed materials performed		
9.	Fill materials contain no excess snow, ice, or frozen chunks		
10.	Lift inspected and acceptable		
11.	Snow and loose or saturated materials removed prior to placement of the lift		
12.	As-built survey conducted (survey by DDMI)		
REMARKS :			
DEVIATIONS : (Attach list if necessary) DATE OF RECTIFICATION :			

ACCEPTED BY: QC Manager	ACCEPTED BY: QA Manager
NAME :	NAME :
SIGNATURE :	SIGNATURE :
DATE :	DATE :

APPENDIX C

North Country Rock Pile Slope Stability Assessment

TECHNICAL MEMORANDUM

DATE	November 19, 2009	**PROJECT No.**	09-1328-0006/4000
TO	Lianna Smith Diavik Diamond Mines Inc.	**DOC. No.**	890 Ver. 0
CC		**PO No.**	D01302 Line 1
FROM	Ben Wickland and John Cunning	**EMAIL**	bwickland@golder.com; jcunning@golder.com

NORTH COUNTRY ROCK PILE SLOPE STABILITY ASSESSMENT
DIAVIK DIAMOND MINES, LAC DE GRAS, NWT

1.0 INTRODUCTION

Diavik Diamond Mines Inc. (DDMI) is reviewing options related to the closure configuration of the North Country Rock Pile (NCRP). Golder Associates Ltd. (Golder) was requested to carry out a slope stability assessment for various configurations of the NCRP. Geotechnical data were reviewed and two dimensional (2D) stability analyses conducted for selected sections through the NCRP for existing pile configurations. A sensitivity analysis was completed to investigate maximum pile slope angle which achieves a minimum slope stability Factor of Safety (FoS) of 1.3.

1.1 Background

1.1.1 Summary of Site Conditions

The Diavik Diamond Mine is located on East Island, a 17 km^2 island in Lac de Gras, NWT (64^{o}31' North, 110^{o}20' West). The mine site is located in the zone of continuous permafrost and is located in the Arctic Climatic Region with long, cold winters and very short, cool summers. Site climate is described in more detail in NKSL (2001).

The site is in a zone of low seismicity. Interpolated seismic hazard values based on the fourth generation seismic hazard maps of Canada following the 2005 National Building Code of Canada have been used to design other facilities at the site (Golder 2007a). The peak predicted ground acceleration (PGA) at corresponding probabilities of occurrence is summarized in Table 1.

Table 1: Peak Ground Acceleration Values for Diavik Mine Site 64.52^{o}N 110.33^{o}W

Return Period (years)	PGA (g)
1 in 100	0.007
1 in 475	0.02
1 in 1,000	0.04
1 in 2,475	0.06

Note: For firm ground, soil class C.

NKSL (1999) reported that surficial geology on the island generally includes 0 m to 2 m of organic cover and 0 m to 6 m soil, primarily till, comprised of silty sand with averages of 17% gravel and 15% cobble and boulders overlying granitic and metasedimentary country rock. Near surface soils are generally re-worked by frost action and show evidence of periglacial processes such as solifluction, thermal cracking and boulder jacking. The active zone is approximately 1.5 m to 2.0 m deep in soil deposits, 2.0 m to 3.0 m in well drained granular deposits (eskers), 5 m in bedrock, and less than 1 m depth in poorly drained areas including bogs, and areas with thicker vegetation (NKSL 1999).

NKSL (1999) classified the soils with greater than 10% visible ice content and 20% moisture content as ice rich. Temperature measurements obtained from thermistors strings installed in the NCRP area prior to mine operations showed that the active layer was approximately 2 m deep in soil deposits with stable ground temperatures between -3.5 and -5°C below depths of 12 m to 17 m. Thermistor data collected in the area of the NCRP between about 1996 and 1999 are included in Appendix I.

Figure 1 presents a plan of the NCRP area, including the original ground surface topography, the existing footprint, foundation soil conditions with ice rich / ice poor classification, borehole and thermistor locations. The existing configuration of the NCRP is shown in Figure 2. The NCRP has been in use since mine start up and the majority of mine rock to be stored in the NCRP is already in place. As open pit mining in both the A154 and A418 pits near completion, the rate of contribution of additional material to the NCRP is expected to be reduced.

Foundation conditions of the NCRP are summarized from NKSL (1999) as follows:

- North area foundation included bedrock at surface transitioning to ice poor soil and then to ice rich soil near the North Inlet with visible ice contents as high as 95%. Total soil thickness varied from 0.2 m to 7.8 m.
- Northwest area foundation included both ice poor soil and ice rich soil with visible ice contents as high as 50%, esker (sand and gravel) and bedrock at surface. Soil thickness, where present, varied from 1.6 m to 7.6 m.
- South area foundation conditions include mainly bedrock, lakebed sediments, ice rich soils had visible ice contents greater than 50%. Total soil thickness, where present, varied from 1.6 m to 6.0 m.
- Southeast area foundation conditions included ice rich soil with visible ice content greater than 50% and ice poor soil. Total soil thickness varied from 2.8 m to 9.1 m.

1.1.2 NCRP Design

The design of the NCRP was presented in two reports by NKSL (1999 and 2001). The original design considered final overall outer pile slopes of 4H:1V where founded on ice rich soil, and 2.5H:1V slopes where founded on ice poor soil. Foundations were modelled as frozen, partially thawed and thawed for static and pseudo-static loading conditions. For frozen soils, NKSL (2001) defined a limiting creep rate for ice rich foundations of 0.01 m/m/year. The current review was undertaken to confirm that the existing as-built NCRP configuration meets the design intent for stability using updated criteria and material properties.

2.0 STABILITY ANALYSIS

Stability analyses were performed to evaluate the existing NCRP configuration, and also to determine maximum pile slope angles that achieve the design criteria FoS for slope stability. Slope stability analyses for two dimensional sections were completed using SLOPE/W 2007 (GEO-SLOPE, 2008). Models considered ice poor foundation soils as both frozen and thawed, ice rich foundation soils at temperatures of -2^{o}C and -4^{o}C, partially thawed and thawed as an upset case for climate change.

2.1 Failure Modes

Mine rock piles typically undergo long term settlements which can reach magnitudes of up to 1 to 2% of the pile height. Slumping and settling of the crest and sloughing on the outer surface with occasional boulder roll-out are considered normal behaviour. A worst case failure mechanism for large rock piles is rapid run-out failure involving large scale mass movements. Rapid run-out failures are typically caused by loss of strength in the foundation at the toe area of the pile. Mine rock at Diavik is a high strength frictional material that is not prone to rapid weathering and breakdown; therefore the credible failure modes for the NCRP must therefore involve failure of the foundation soils.

Mechanisms for rapid loss of strength in foundation soils can include increase in pore water pressures induced by loading during operations, by increase in head of ponded water during operations or throughout closure, by rapid thawing of ice rich materials, or by creep rupture of ice rich soils.

Loading by deposition of additional rock on the pile can induce failure of the foundation materials. Loading induced failures are typically controlled by monitoring movement rates of actively loaded piles and are a concern for operations, rather than over the long term. In contrast to piles with large tip faces that require monitoring for crest displacement, DDMI operates the NCRP by placing rock from the bottom up in relatively thin lifts. Loading induced failure is therefore not expected to be a credible mechanism of failure over the long term.

For areas of the NCRP founded on bedrock and on ice poor permafrost soils, behaviour of the pile and foundation are expected to be similar to piles at warmer sites. The key factor for large run-out failures in mountainous terrain at warmer sites is foundation slope angle. Piles on steep slopes are more prone to failure than piles on flat slopes. The NCRP foundation conditions are generally flat, as illustrated in Figure 1. Foundation slope is therefore not considered to be a key element in slope stability failure.

For piles founded on ice rich permafrost soils, the pile will be subject to foundation related creep. Strength behaviour of ice rich permafrost soils is generally dependent on ice content, ice distribution, air content, soil particle size, soil temperature, total load and loading rate. Lower temperature frozen soils generally have higher strength. Higher ice content soils will generally carry more load on the ice phase and the ice will creep or flow under load resulting in creep behaviour of the soil. If creep rate is limited, creep related movement occurs relatively slowly and is not a credible mechanism for rapid loss of strength resulting in run-out failure.

Creep rate in frozen soils is dependent on loading and temperature with higher creep rates observed at higher temperatures and under higher loading conditions. Maximum creep rates occur at near thawed conditions. Ice rich soils at near thawed conditions contain a water phase that may influence the strength of the soil, but the process of deformation is not well understood. Seasonally higher creep rates are therefore expected to be correlated with seasonal warming of ice rich soil foundations.

A possible failure mechanism for foundation materials is the rapid thaw of ice rich soils resulting in consolidation related excess pore water pressures. The thaw-consolidation ratio is expected to be low for soils at Diavik (Golder 2007b), and therefore build up of pore water pressure is not expected to occur in the foundation should thawing conditions prevail. For thawed conditions load is carried by the soil grains and the soil acts as a frictional material.

The strength of ice rich soils also depends on rate of loading. Deformation of ice rich soils subjected to rapid, dynamic loading, such as seismic loading, is less than if subjected to long term static loading that induces creep related to ice content.

Creep rate is, without other influences, variable with time. Ice rich soils may be subject to changes in creep through primary, secondary and tertiary phases. The primary phase is characterized by decreasing creep rate, the secondary phase by steady creep rate and the tertiary phase by increase in strain rate up to creep rupture. Tertiary creep is generally associated with high stress conditions.

Creep behaviour of ice rich foundation soils will result in foundation and pile movement over time. Limiting creep rate criteria has been used in the assessment of the stability analyses results.

2.2 Analysis Criteria

Criteria for slope stability in the present analyses are presented in Table 2.

Table 2: Summary of Slope Stability Analysis Criteria

Criteria	Value	Reference
Method of Analysis	Morgenstern-Price	
Minimum Factor of Safety		
Static Loading Condition	1.3	Draft Rio Tinto Guidance Note – *Surface Mining: Design and Management of Stockpiles, Spoil and Waste Dumps*, BC MWRPRC 1991.
Pseudo-static Loading Condition	1.0	CDA (2007) for dams, common engineering practice for piles.
Peak Ground Acceleration (1/1,000 year earthquake occurrence)	0.04 g	2005 National Building Code Seismic Hazard Calculation.
Horizontal Ground Acceleration used in the Model	0.02 g	Kyrou (2002), Hynes-Griffin and Franklin (1984).
Foundation Temperatures	-2 to -4°C	NKSL (2001).

Design analyses used a value of horizontal seismic coefficient of 0.02 g, based the Geological Survey of Canada 1995 National Building Code assessment, factored for embankment height (NKSL 2001). Since the design was completed, an updated seismic assessment has been completed for the 2005 National Building Code Seismic Hazard Calculation, and is used in the present analysis. The 1 in 1000 year return period peak ground acceleration (PGA) for Diavik area is 0.04 g (Table 1). The value of horizontal seismic coefficient used in the pseudo-static loading cases analyses was half the PGA, or 0.02 g, based on the method presented by Hynes and Franklin (1984).

The present study adopts the criteria that a value of FoS equal to 1 is required for pile stability under pseudo-static loading conditions. The design value of FoS for pseudo-static loading was set at 1.1 by NKSL (2001) based on engineering practice for earth fill dams. Earthquake related failure of embankments will typically result in increased rate of settlement, rather than a major rotational failure. Pseudo-static loading may result in shallow ravelling or boulder roll-out from the face of a waste rock pile. While settlement of a dam can result in overtopping and erosion related catastrophic failure, there is no danger of overtopping for a rock pile. A design value of FoS for pseudo-static loading of 1.0 is adopted for the present analysis and is consistent with guidelines for the design of rock piles (BC MWRPRC 1991), and also for dams under the latest guidelines from the Canadian Dam Association (CDA 2007).

2.3 Creep Rate Criteria

Criteria for the present analysis for pile stability on ice rich permafrost foundations soil are adopted from NKSL (2001). The creep rate criteria is a limit of 0.01 m/m/years, and is evaluated based on a limit equilibrium stability FoS of 1.3 using ice rich soil creep strength corresponding to a rate of 0.1 m/m/year.

While limit equilibrium methods for slope stability are used here to evaluate creep of ice rich foundations, the results are an indication of creep rate, rather than slope stability failure. Failure of the design criteria implies that the slope and ice rich foundation will creep at a higher rate than the criteria.

2.4 Model Geometry

Model sections for stability analysis of the existing pile configuration were selected based on section locations shown in plan on Figures 1 and 2. Modeled sections are summarized in Table 3 and illustrated in Figure 3. Maximum pile elevation for the existing pile configuration was 496 masl. Sensitivity analyses for maximum allowable pile slopes are based on the North Section ground surface topography. The North Section ground surface has ground surface that slopes down away from the pile, and is considered the least favourable for stability of the pile.

The NCRP was built from the bottom up by placing rockfill at angle of repose in lifts and includes wide haul road benches incorporated along the outside pile slopes. The method of construction produces more stable configurations than top down construction with rock tipped down a larger face. Accounting for the benched configuration, the existing slopes have average overall or composite slopes varying from 3H:1V at the South Section to near 8H:1V at the Southeast Section location, as indicated in Table 3. For unfrozen foundation conditions, phreatic surfaces were assumed to be at the top the soil layer.

Table 3: Summary of Model Geometry

Section Location	North NCRP	Northwest NCRP	South NCRP	Southeast NCRP
Soil Thickness (m)	8	8	6	9
Existing Pile Crest Elevation (masl)	493	496	494	494
Average Slope Angle	3.8H:1V	3.4H:1V	3.0H:1V	7.7H:1V

Slopes of the NCRP at the South section location shown in Figure 2 are bounded by the Processed Kimberlite Containment (PKC) Facility. The stability of the NCRP is considered independently of the PCK Facility, which will tend to increase the stability of the NCRP against slope stability failure.

Thickness of soil in the foundation of the PKC is variable. Modelled sections assume a continuous thickness of soil below the NCRP, typically near the maximum thickness encountered in boreholes adjacent the sections. The assumption is conservative with respect to foundation strength and slope stability.

2.5 Material Properties

Material properties used in the stability analyses are listed in Table 4.

Table 4: Summary of Material Properties

Material	Unit Weight (kN/m^3)	Friction Angle, ϕ (deg.)	Cohesion, c (kPa)	Reference
Mine Rock	20	45	0	Golder (2007a and b)
Soil – Ice Rich				
Thawed	16	29	5	Golder (2007 a and b)
Partly Thawed	16	24	0	NKSL (1999 and 2001)
Frozen T= -2°C	14	0	110	Golder (2007 a and b); NKSL (1999 and 2001)
Frozen, T= -4°C	14	0	150	Golder (2007 a and b); NKSL (1999 and 2001)
Soil - Ice Poor				
Thawed	18	32	0	Golder (2007 a and b)
Frozen	18	0	400	NKSL (1999 and 2001)

The value of 45 degrees used in the present analysis is considered representative of the high angularity, high strength rock at Diavik.

It is noted that the distribution of ice rich permafrost soil is variable. Foundation conditions were modeled for a range of soil foundation conditions, including profiles of all ice rich, all ice poor, partly thawed and thawed. The assumptions of soil foundation conditions therefore bound the expected best and worst credible cases for foundation soil strength.

In addition to values in Table 5, strengths for ice rich soils and corresponding creep rates used for sensitivity analyses are presented in Figure 4, based on NKSL (2001).

3.0 RESULTS

Rock at Diavik is generally considered to be durable, with high strength and friction angle and credible failure mechanisms must therefore consider failure through the foundation below the pile. Models considered the near maximum soil thickness and also assume ice rich soil conditions for each section. The assumptions are conservative with respect to slope stability. For foundations including ice rich permafrost soils, the creep rate of the pile is expected to vary with foundation temperature.

3.1 Existing Pile Configuration

Results of stability analyses for the existing pile configurations are summarized in Table 5. Typical failure surfaces analyzed are illustrated in Figure 5. Results of stability analyses involving creep rate of ice rich permafrost foundations are summarized in Table 6.

Table 5: Summary of Stability Analysis Results for Existing NRCP Configuration

Section Location		North		Northwest		South		Southeast	
		FoS for Static and (Pseudo-Static) Loading Conditions							
Foundation Condition		Circular	Block	Circular	Block	Circular	Block	Circular	Block
Soil - Ice Rich	Partly Thawed	1.6 (1.4)	2.0 (1.9)	1.5 (1.5)	1.8 (1.7)	2.6 (2.4)	2.5 (2.4)	1.4 (1.4)	1.5 (1.4)
	Thawed	1.5 (1.4)	2.1 (2.0)	1.5 (1.5)	2.0 (1.8)	2.7 (2.5)	2.9 (2.7)	1.4 (1.4)	1.5 (1.4)
Soil - Ice Poor	Frozen	3.6 (3.3)	3.7 (3.4)	2.7 (2.6)	2.7 (2.5)	3.0 (2.8)	3.0 (2.8)	3.1 (3.0)	3.4 (3.3)
	Thawed	1.7 (1.6)	2.2 (2.1)	1.7 (1.6)	2.1 (2.0)	3.1 (2.9)	3.1 (2.9)	1.6 (1.5)	1.6 (1.5)

Table 6: Summary of Creep Rate Analysis Result for Existing NRCP Configuration

Section Location	North		Northwest		South		Southeast	
Foundation Condition	Circular	Block	Circular	Block	Circular	Block	Circular	Block
Soil – Ice Rich, Frozen T= -2°C	1.5	1.5	1.1	1.2	1.5	1.5	1.3	1.4
Soil – Ice Rich, Frozen T= -4°C	1.5	1.8	1.3	1.4	1.7	1.9	1.6	1.8

Results in Table 5 indicate that the existing pile configuration meet or exceeds the criteria for slope stability for circular and block failure modes under static and pseudo-static loading conditions under partly thawed and thawed conditions for ice rich and ice poor foundation conditions. Results in Table 6 indicate that the existing pile configuration would meet the creep rate criteria for ice rich permafrost soils, with the exception of the Northwest section. The models of the existing pile at the Northwest Section fail the current creep criterion. For the Northwest section the creep rate is expected to be higher than the design criteria of 0.01 mm/year for frozen conditions around -2 degrees Celsius. The current analyses do not allow this higher creep rate to be quantified.

3.1.1 Maximum Pile Slope Angle

Sensitivity analyses were conducted to determine the maximum pile slope angle that achieves the design criteria for slope stability for the North Section foundation surface topography. Results are presented in Figure 6 and Tables 7 and 8. For thawed conditions, the steepest overall pile slope found to sustain the design FoS of 1.3 was near 1.3H:1V, or angle of repose for rockfill, for the limiting case of partly thawed foundation conditions. For ice rich permafrost foundation soils, the maximum pile slopes were 2H:1V for a -4°C ice rich soil permafrost foundation and 3H:1V for a -2°C ice rich soil permafrost foundation. In comparison, the existing pile configuration had overall slope angles near 3H:1V at the South Section, which is buttressed by the PKC facility, and flatter than 3H:1V slopes at the other critical section locations.

In general, a pile with slopes near angle of repose for rock fill will meet the design criteria for stability, but may undergo creep at rates higher than the design criteria of 0.01 m/m/year if founded on ice rich soils.

Table 7: Summary of Stability Analysis Results for Maximum Pile Slope Angle

Foundation	Thermal Condition of Foundation	FoS for 1.3H:1V Slope for Static and (Pseudo-Static) Loading
Soil – Ice Rich	Partly Thawed	1.4 (1.3)
	Thawed	1.5 (1.5)
Soil – Ice Poor	Frozen	2.1(2.0)
	Thawed	1.6 (1.5)

Table 8: Summary of Creep Rate Analysis Results for Maximum Pile Slope Angle

Section	2H:1V Slope	3H:1V Slope
Foundation	**FoS**	**FoS**
Soil – Ice Rich, Frozen T= -2°C	1.0	1.3
Soil – Ice Rich, Frozen T= -4°C	1.3	1.6

4.0 SUMMARY

A stability assessment for the NCRP has been carried out for existing slope configurations. A range of foundation conditions have been considered including maximum thickness of foundation soils near analysis sections, ice poor permafrost soil in frozen and thawed states, ice rich permafrost soil foundations at -2°C and -4°C, partially thawed and thawed conditions as an upset case for climate change.

- Sections at critical locations through the existing pile indicated composite slopes with overall slope angles ranging from 3H:1V to near 8H:1V.

- 4 sections through the existing configuration of the NCRP were analysed for 2D slope stability for block and circular failure modes under static and pseudo-static loading. Existing pile configurations met design criteria for slope stability.

Stability analysis indicated that the existing pile configurations meet or exceed design criteria FoS for slope stability for the range of foundation conditions assumed, under both static and pseudo-static loading conditions.

Review of the NCRP stability against the creep rate criteria indicated that higher than design creep rates would be expected at selected locations where the pile is founded on ice rich permafrost soil, including the Northwest Section. However, the result is based on the assumption of continuous ice rich permafrost soils and represents the worst-case scenario. The distribution of ice rich soils is not continuous and actual creep rates may not be greater than design criteria. Creep rates will be controlled locally by foundation temperature and ice content.

The maximum stable rock pile slope angle depends on foundation soils and temperature, but is generally near 1.3H:1V for slope stability criteria and less steep for ice rich foundations based on the creep rate criteria.

- Sensitivity analyses indicated that a rock pile at the maximum slope angle of 1.3H:1V, or angle of repose of rockfill, achieved the design criteria FoS of 1.3 on ice poor foundation conditions.
- For ice rich soil foundation conditions, the maximum slope angle was 2H:1V for conditions near -4^{o}C; and 3H:1V for conditions near -2^{o}C, due to creep rate criteria.

Seasonal variation in creep rate of the pile related to distribution and temperature of ice rich foundation soil conditions is expected. Approximate extents of ice rich soils near the toe of the NCRP with the potential to creep include: 175 m at the north western edge, 400 m of the northern edge of the pile, and approximately 200 m at the southeast edge. Some movement related to ice rich soil foundations is expected at these locations, however, the pile is expected to remain stable against catastrophic run-out failure. Settlement and lateral movement are expected for all rock piles over the long term, and some additional settlement and lateral movement due to ice rich foundation conditions are expected.

Recommendations include:

- Visual inspection and documentation of the NCRP toe and outer slopes on a yearly basis.
- Berms along the pile toe to control roll out of boulders ravelling down the face.
- Monitoring of foundation temperatures in areas of ice rich soils on a quarterly basis to establish the pattern of ground temperature beyond the yearly cycle, *i.e.,* either warming or cooling. (This will require installation of new thermistors strings).
- In areas where the NCRP is at final configuration, establish monitoring of crest and toe displacements to demonstrate movement rates of the pile.
- Review of the PKC East Dam slope inclinometer and temperature data to confirm the current creep rate of this angle of repose rockfill dam shell (pile). Depending on results, it may be further recommended to confirm creep rates at the NCRP for areas of the pile toe founded ice rich soils, *i.e.,* these areas would be monitored for movement and creep rate using inclinometer casings.

We trust that this memorandum meets your requirements at this time. Please do not hesitate to contact the undersigned should you have any questions.

GOLDER ASSOCIATES LTD.

ORIGINAL SIGNED

Ben Wickland, Ph.D., P.Eng.
Geotechnical Engineer

ORIGINAL SIGNED AND SEALED

John Cunning, M.Sc., P.Eng.
Associate, Geotechnical Engineer

BEW/JFC/JCC/rs

Attachments: Figures 1 to 6
Appendix I: Thermistor Data

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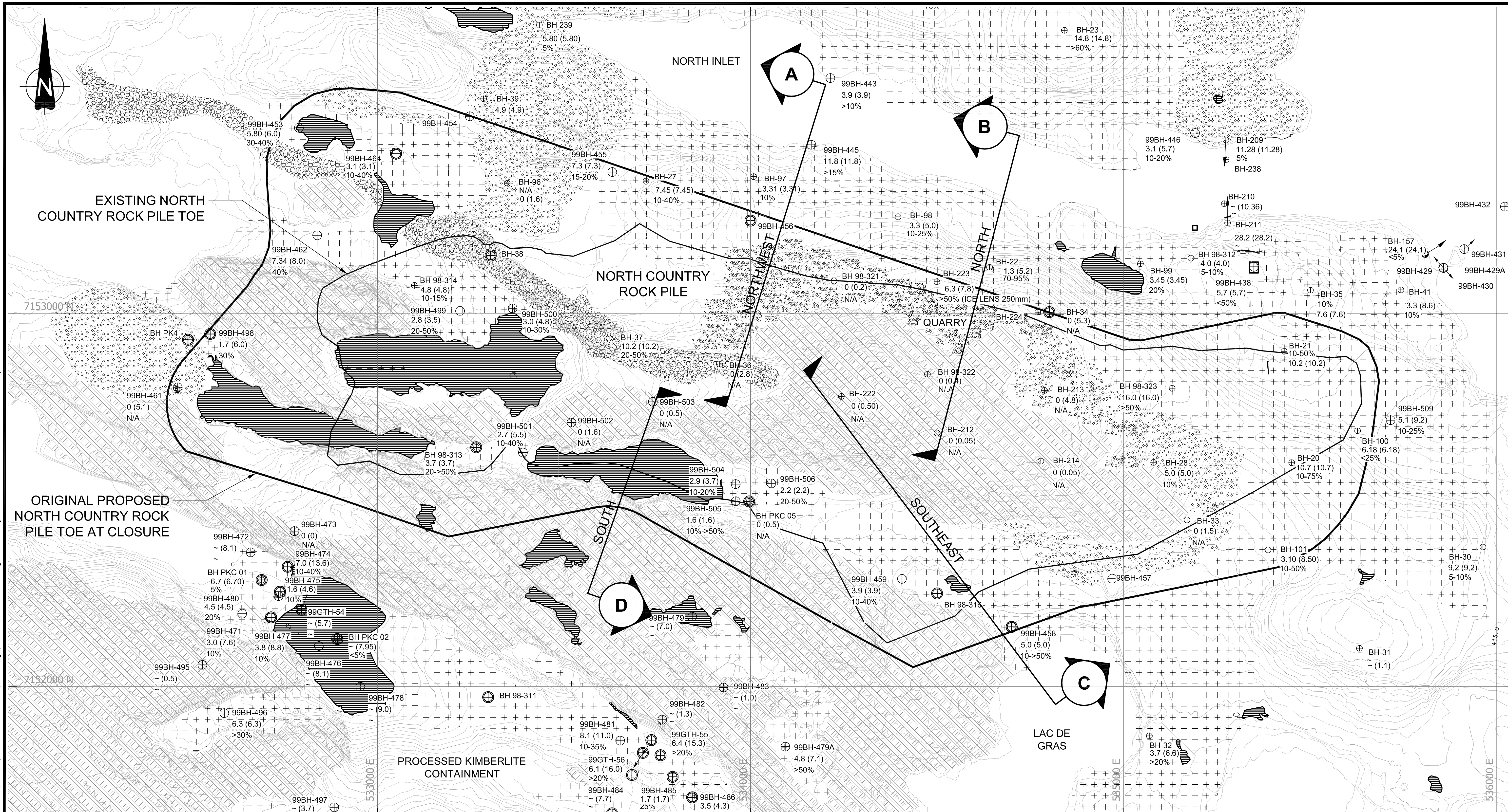
NKSL 1999. Country Rock and Till Storage Design Report. NISHI-KHON/SNC-LAVALIN Limited, August, 1999.

NKSL 2001. Country Rock and Till Storage Updated Design Report. NISHI-KHON/SNC-LAVALIN Limited, August, 2001.



LEGEND

- ICE POOR GLACIAL TILL (<10%)
- ROCK
- LAKE BED SEDIMENTS
- ESKER
- ICE RICH GLACIAL TILL (>10%)
- CAN NOT BE IDENTIFIED PRESENTLY
- BOREHOLE
- BOREHOLE WITH THERMISTOR

98-320 — BOREHOLE NUMBER
0.5 (1.75) — DEPTH TO ROCK
— DEPTH OF SOIL CONTAINING GROUND ICE
5% — ICE CONTENT (%)

0 100 200 300 400 500

SCALE: 1: 10,000

REFERENCES

1. BASE PLAN PROVIDED BY SNC.
   CAD FILE: 420041d93102rB.dwg
   DATED: JUNE 28, 2001

PROJECT

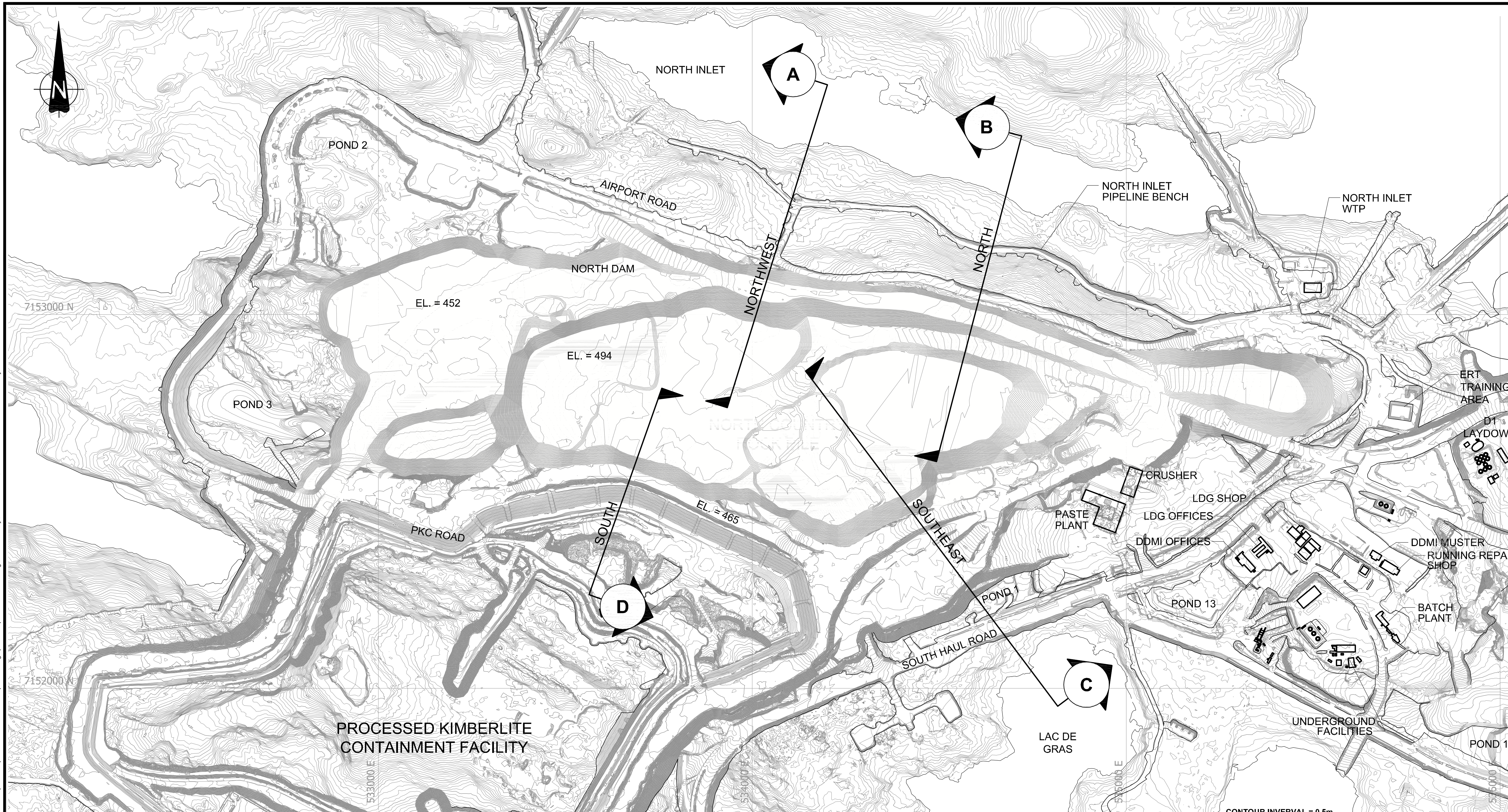
DIAVIK DIAMOND MINES INC.
NORTH COUNTRY ROCK PILE
NWT

TITLE

SITE PLAN SHOWING ORIGINAL GROUND

Golder Associates
Greater Vancouver Office, BC

PROJECT No. 09-1328-0006			PHASE No.		4000
DESIGN	FA	05JUNE09	SCALE	AS SHOWN	REV. –
CADD	AS	05JUNE09	FIGURE 1		
CHECK					
REVIEW					



Drawing File: N:\Bur-Graphics\Projects\2009\1328\09-1328-0006\Drafting\4000\FIGURE 2.dwg Tuesday, June 23, 2009 9:34:35 AM By: MHeal

LEGEND

0 100 200 300 400 500

SCALE: 1: 10,000

REFERENCES

1. BASE PLAN PROVIDED BY DIAVIK DIAMOND MINES INC.
   CAD FILE: airconts+abs-jan-17.dwg
   DATED: JANUARY 17, 2009
   UPDATE BASED ON: current_dump_topo.dwg

PROJECT

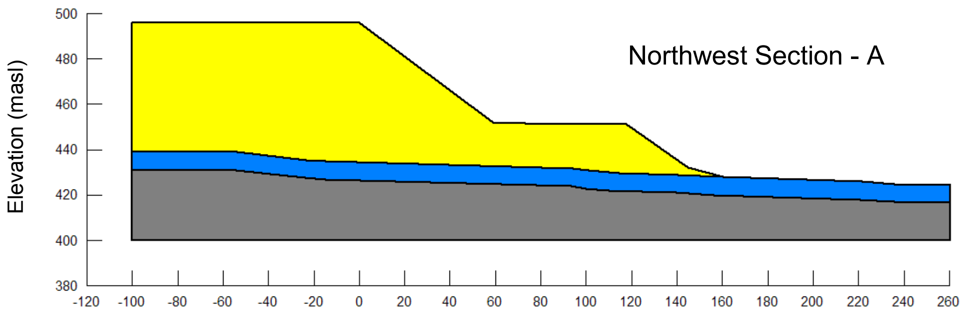
DIAVIK DIAMOND MINES INC.
NORTH COUNTRY ROCK PILE
NWT

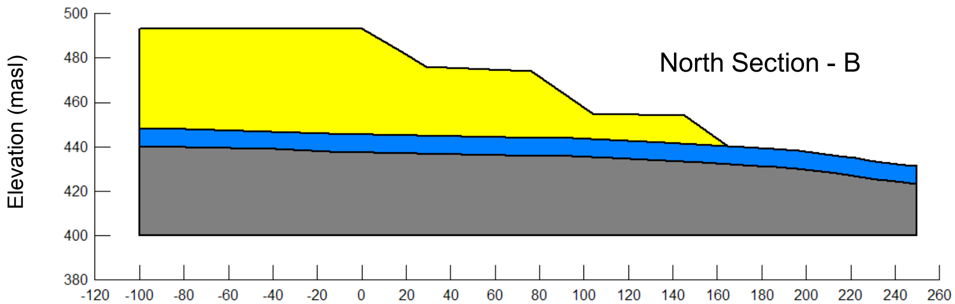
TITLE

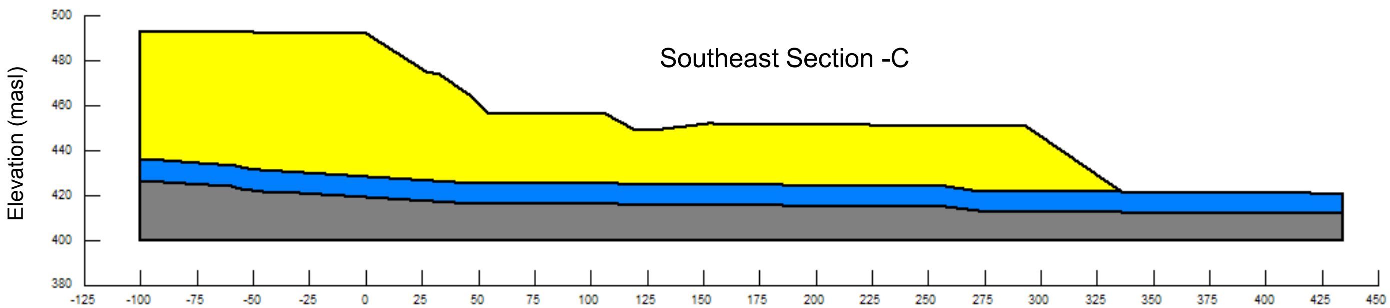
SITE PLAN
SHOWING EXISTING PILE CONFIGURATION

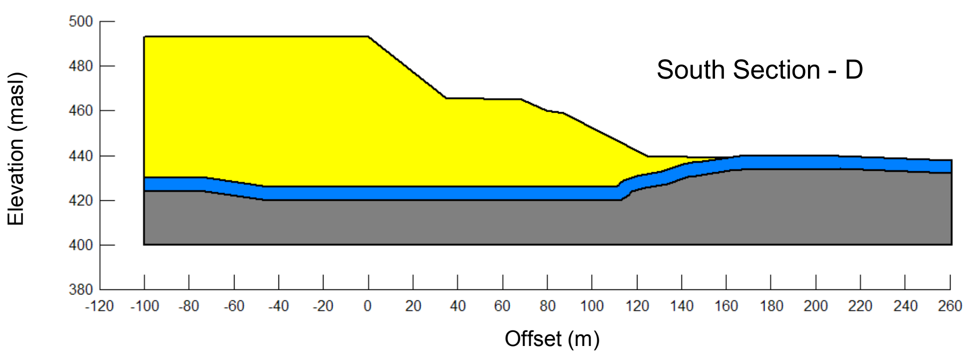
Golder Associates
Greater Vancouver Office, BC

PROJECT No. 09-1328-0006			PHASE No. 4000	
DESIGN	FA	05JUN09	SCALE AS SHOWN	REV. –
CADD	MSH	16JUN09	**FIGURE 2**	
CHECK				
REVIEW				









Material	Unit Weight (kN/m³)	Friction Angle, ϕ' (deg.)	Cohesion, *c* (kPa)
Mine rock	19.6	45	0
Ice-rich soil			
Thawed	16	29	5
Partly thawed	16	24	0
Frozen T = -2°C	14	0	110
Frozen T = -4°C	14	0	150
Ice-poor soil			
Thawed	18	32	0
Frozen	18	0	400
Bedrock	Impenetrable		

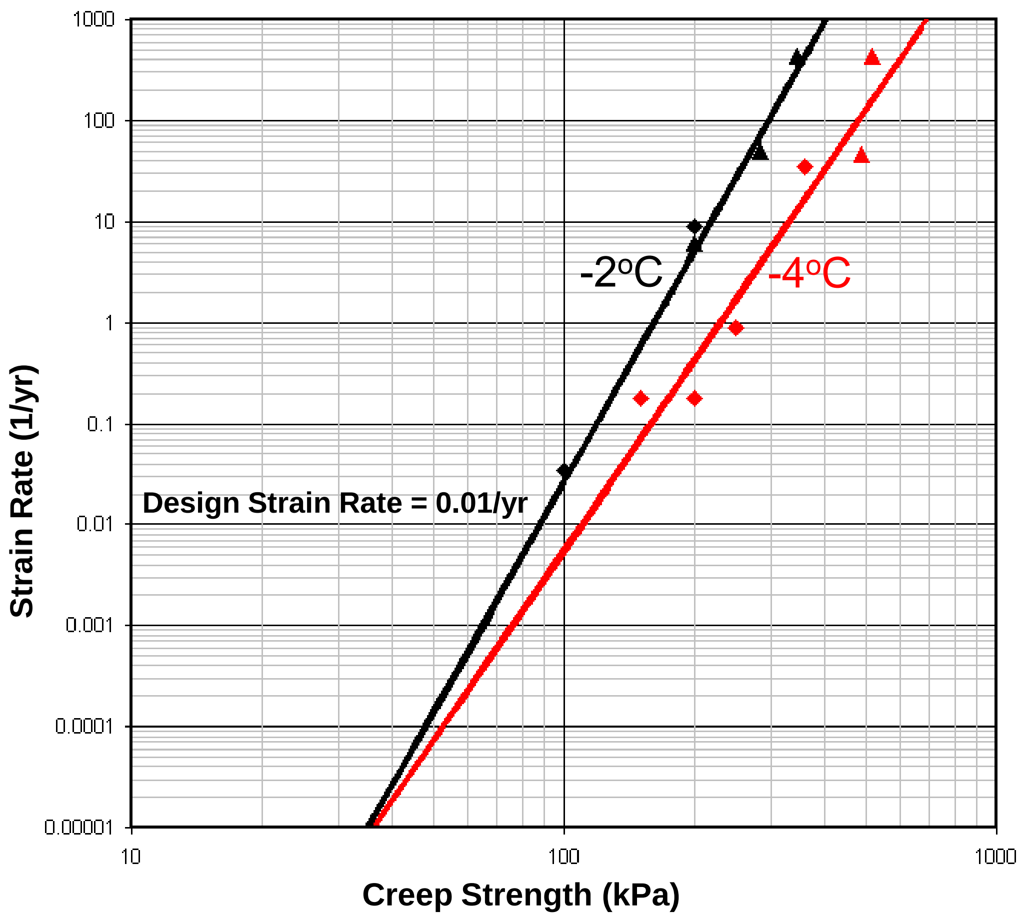
Material	
Mine rock	
Soil	
Bedrock	

PROJECT
DIAVIK DIAMOND MINES INC.
NORTH COUNTRY ROCK PILE
NWT

TITLE
STABILITY ANALYSIS MODEL SECTIONS
NCRP EXISTING CONFIGURATION

Golder Associates

PROJECT No. 09-1328-0006			FILE No.	
DESIGN	BEW	16JUN09	SCALE NTS	REV.
CADD	BEW	16JUN09	**FIGURE 3**	
CHECK	--	--		
REVIEW				



▲	Constant Stress (- 2°C)
◆	Constant Strain (- 2°C)
▲ (red)	Constant Stress (- 4°C)
◆ (red)	Constant Strain (- 4°C)

creep strength laboratory test data from NKSL (2001)

PROJECT

DIAVIK DIAMOND MINES INC.
NORTH COUNTRY ROCK PILE
NWT

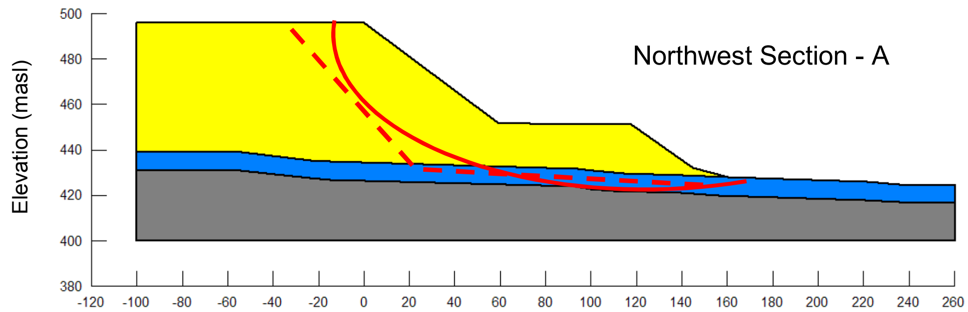
TITLE

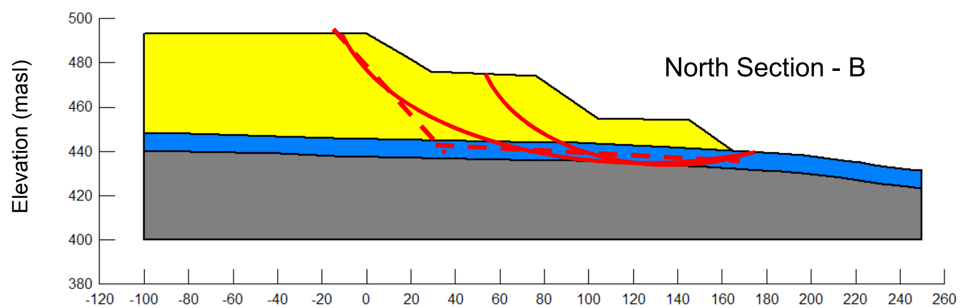
ICE-RICH SOIL CREEP STRENGTH

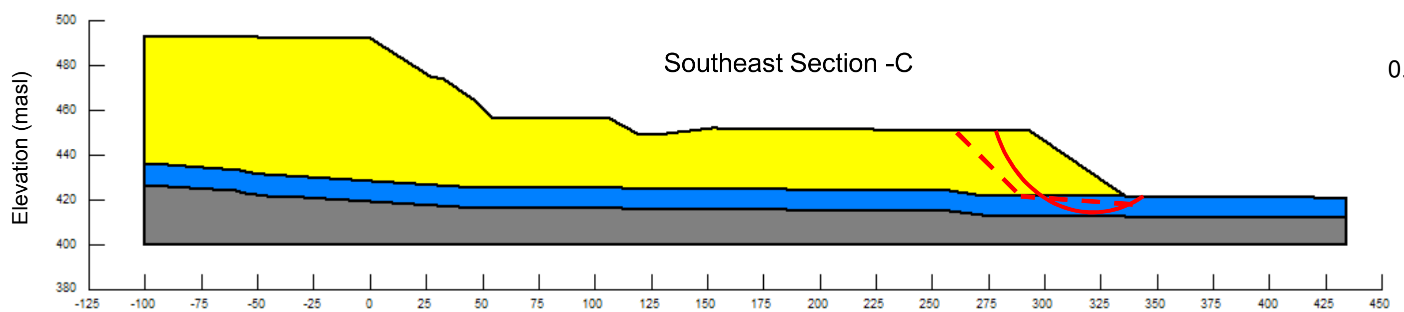
Golder Associates

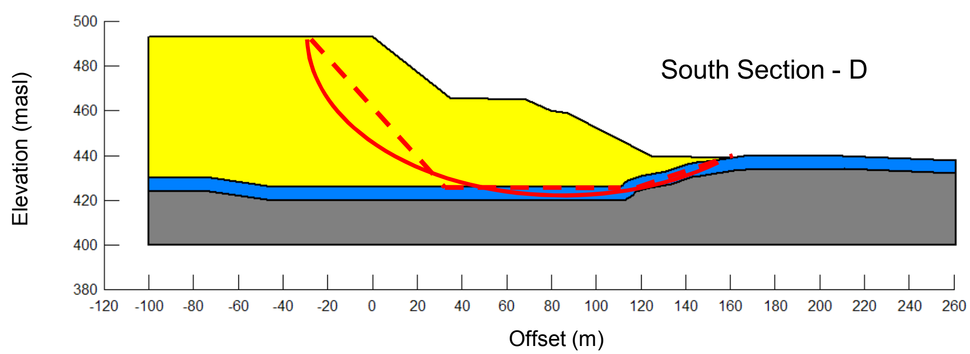
PROJECT No.	09-1328-0006		FILE No.
DESIGN	JFC	24APR09	SCALE NTS REV.
CADD	BEW	17SEP09	**FIGURE 4**
CHECK	--	--	
REVIEW			

REVISION DATE: Sept. 17, 2009 BY: BEW FILE: O:\Active_2009\1328\09-1328-0006\4000 North Country Rock Pile-Slope Stability Assessment\NCRP Stability Analysis\Memo\Figures









Section Location		North		Northwest		South		Southeast	
		FoS for Static and (Pseudo-Static) Loading Conditions							
Foundation Condition		Circ.	Block	Circ.	Block	Circ.	Block	Circ.	Block
Ice rich	Partly Thawed	1.6 (1.4)	2.0 (1.9)	1.5 (1.5)	1.8 (1.7)	2.6 (2.4)	2.5 (2.4)	1.4 (1.4)	1.5 (1.4)
	Thawed	1.5 (1.4)	2.1 (2.0)	1.5 (1.5)	2.0 (1.8)	2.7 (2.5)	2.9 (2.7)	1.4 (1.4)	1.5 (1.4)
Ice poor	Frozen	3.6 (3.3)	3.7 (3.4)	2.7 (2.6)	2.7 (2.5)	3.0 (2.8)	3.0 (2.8)	3.1 (3.0)	3.4 (3.3)
	Thawed	1.7 (1.6)	2.2 (2.1)	1.7 (1.6)	2.1 (2.0)	3.1 (2.9)	3.1 (2.9)	1.6 (1.5)	1.6 (1.5)

typical circular type failure surface

typical block type failure surface

0.02g horizontal seismic load for pseudo-static cases

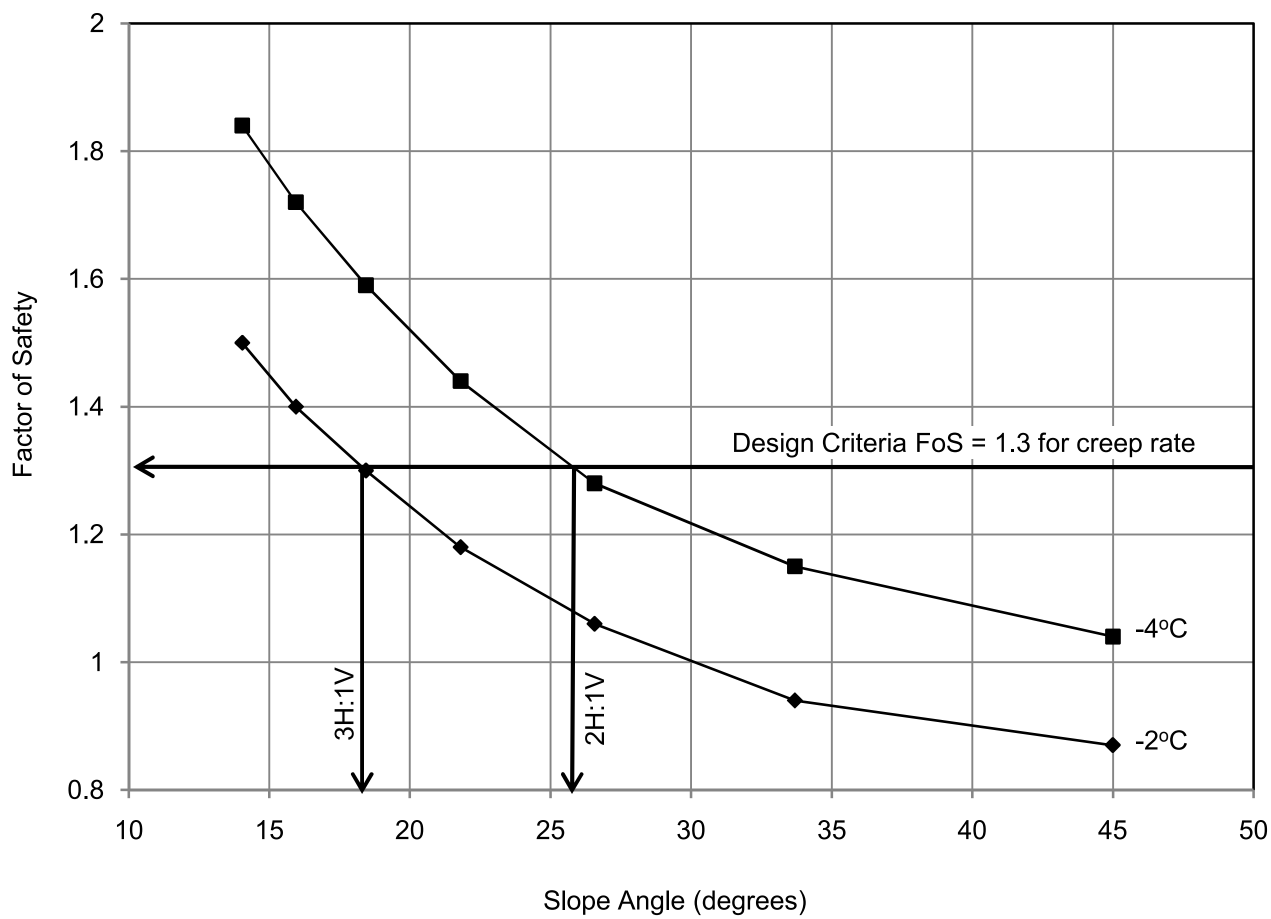
Material	
Mine rock	
Soil	
Bedrock	

PROJECT
DIAVIK DIAMOND MINES INC.
NORTH COUNTRY ROCK PILE
NWT

TITLE
STABILITY ANALYSIS TYPICAL RESULT EXISTING CONFIGURATION

Golder Associates

PROJECT No. 09-1328-0006			FILE No.	
DESIGN	BEW	17JUN09	SCALE NTS	REV.
CADD	BEW	17SEP09	**FIGURE 5**	
CHECK	--	--		
REVIEW				



Calculated values of FoS are for rock pile on ice rich permafrost soils.

PROJECT

DIAVIK DIAMOND MINES INC.
NORTH COUNTRY ROCK PILE
NWT

TITLE

STABILITY ANALYSIS SENSITIVITY
SLOPE ANGLE

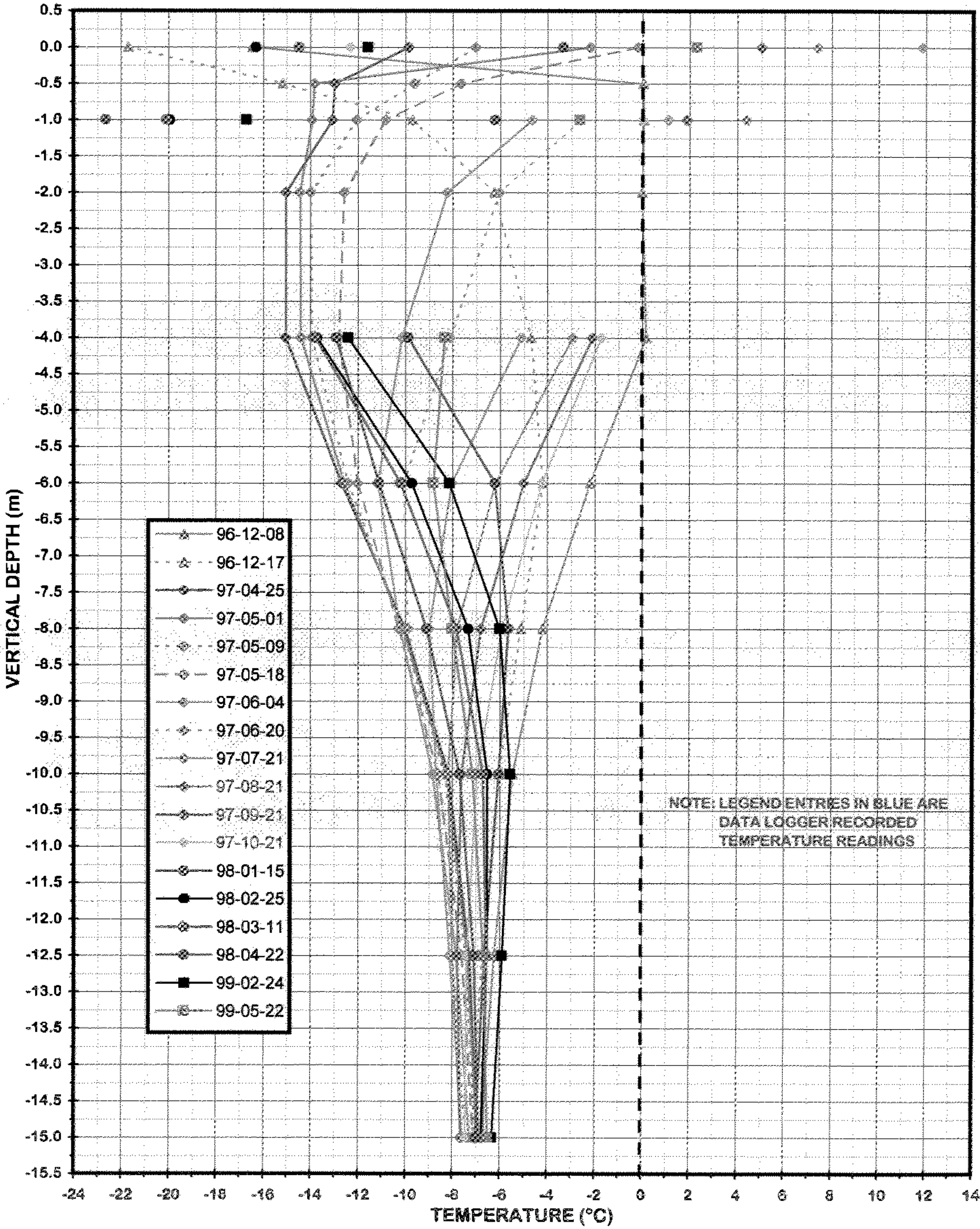
Golder Associates

PROJECT No.	09-1328-0006		FILE No.	
DESIGN	BEW	16JUN09	SCALE NTS	REV.
CADD	BEW	17SEP09	**FIGURE 6**	
CHECK	--	--		
REVIEW				

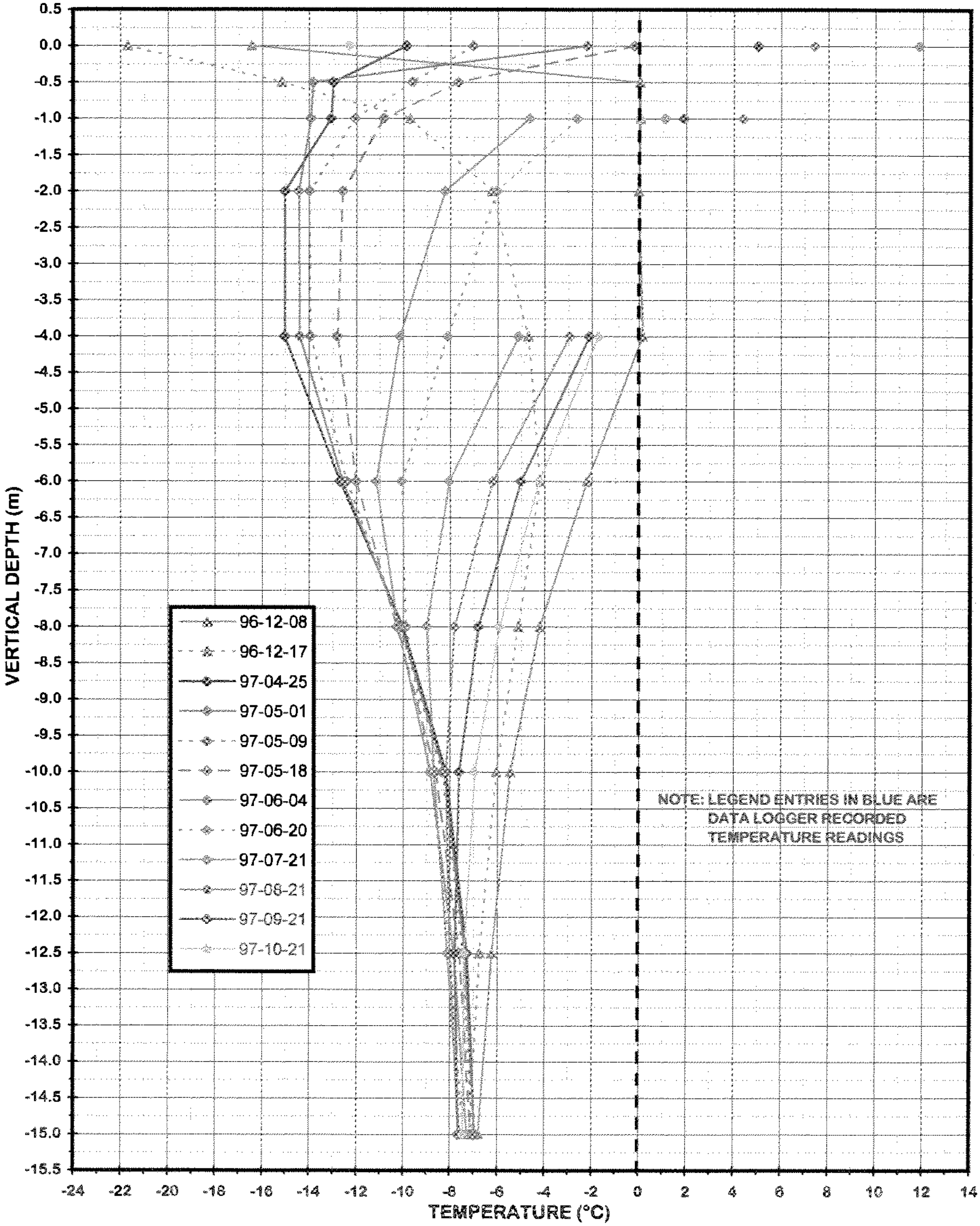
APPENDIX I

THERMISTOR DATA

96BOR-34 GROUND TEMPERATURE PROFILE (ALL DATA)

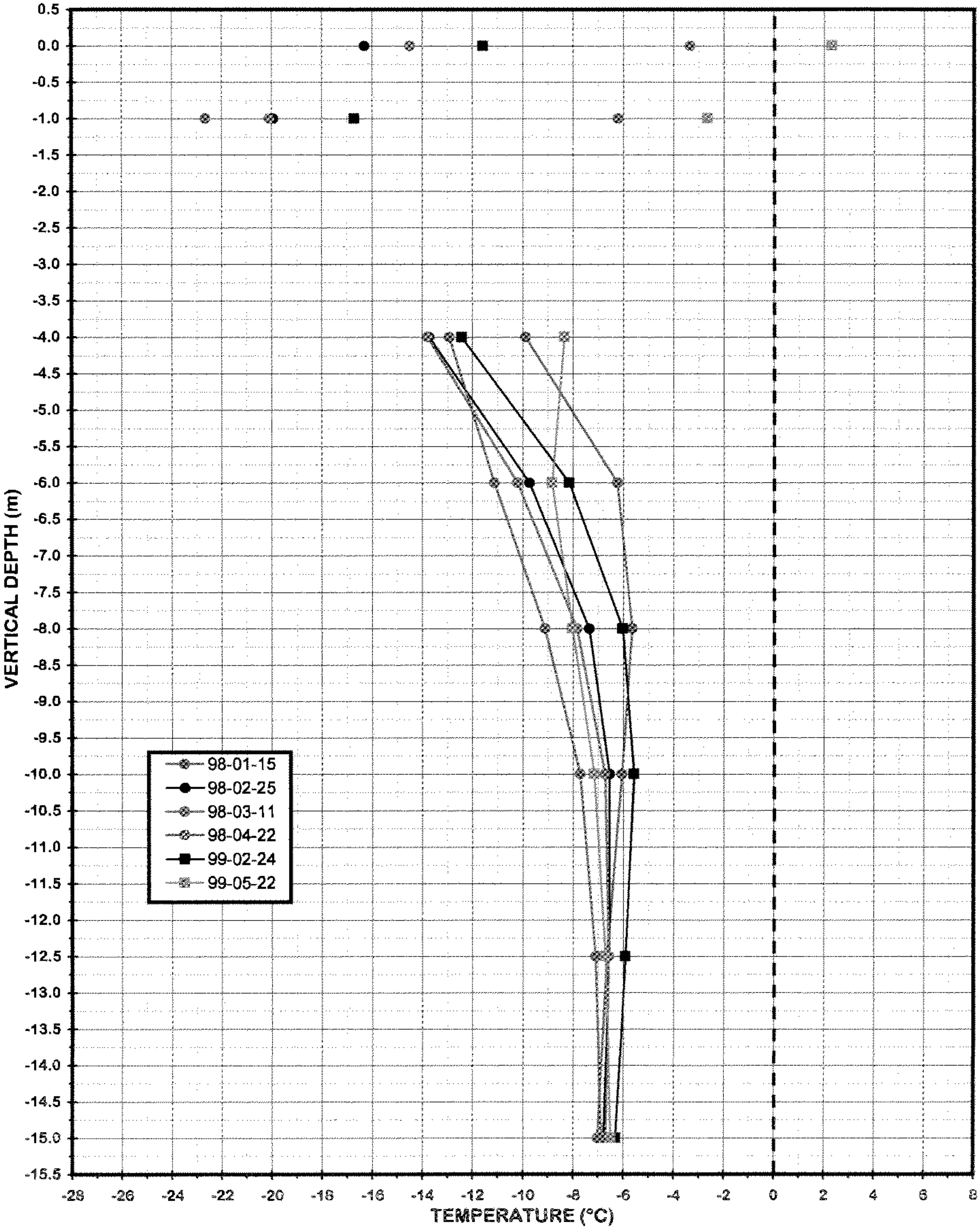


96BOR-34 GROUND TEMPERATURE PROFILE (1996 - 1997 DATA)



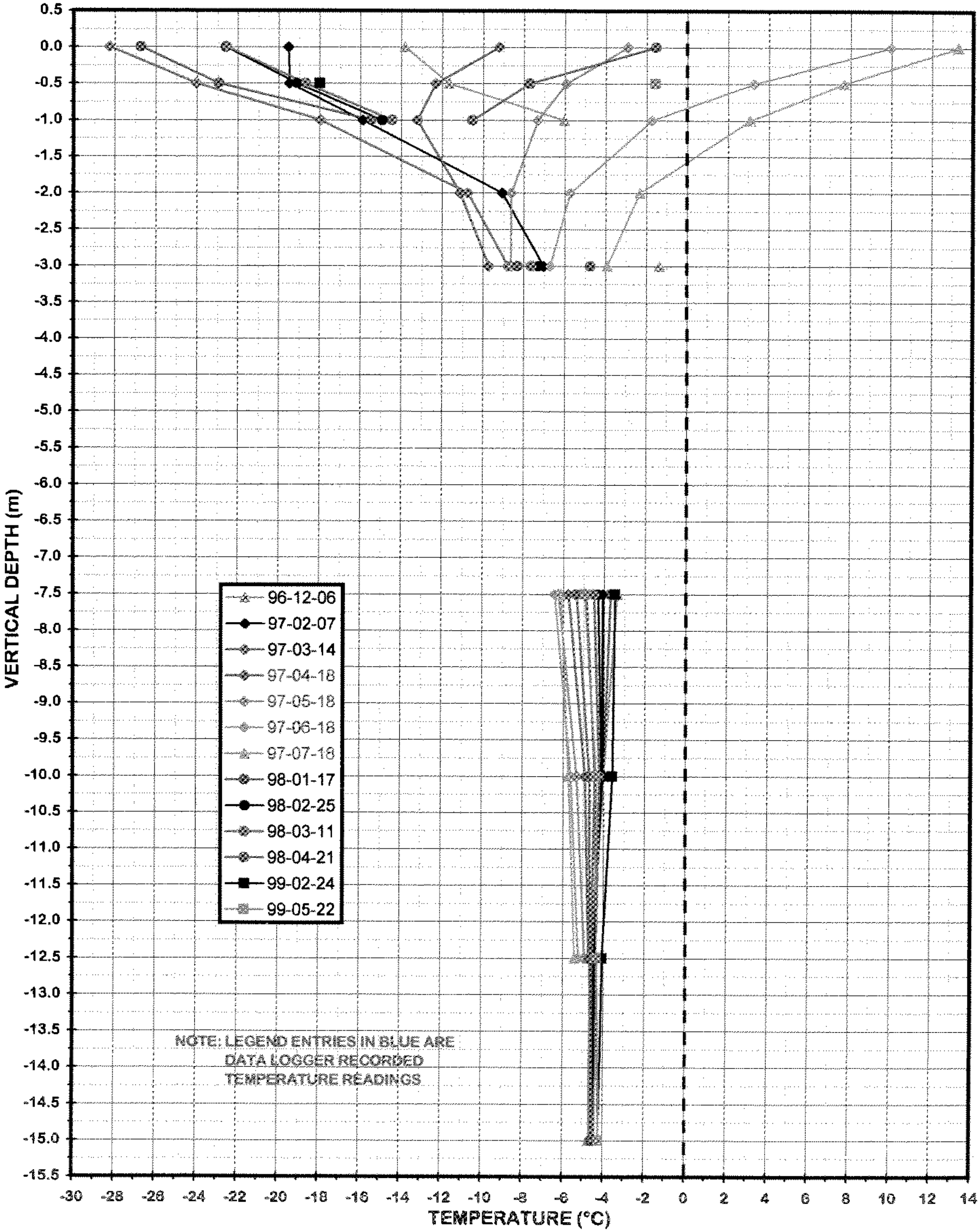
DIAVIK DIAMOND MINES THERMISTOR DATA

96BOR-34 GROUND TEMPERATURE PROFILE (1998 - 1999 DATA)

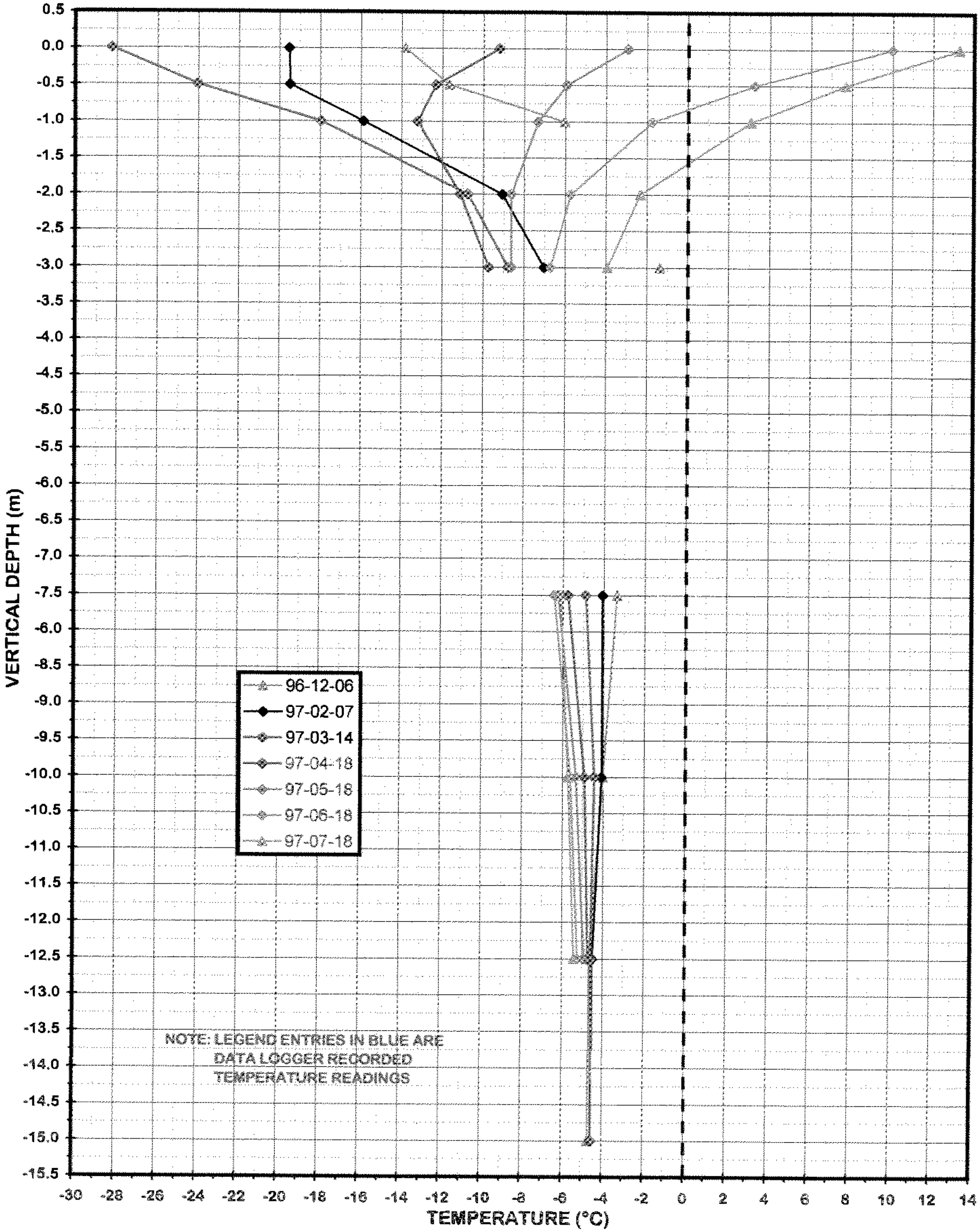


DIAVIK DIAMOND MINES THERMISTOR DATA

96BOR-38 GROUND TEMPERATURE PROFILE (ALL DATA)

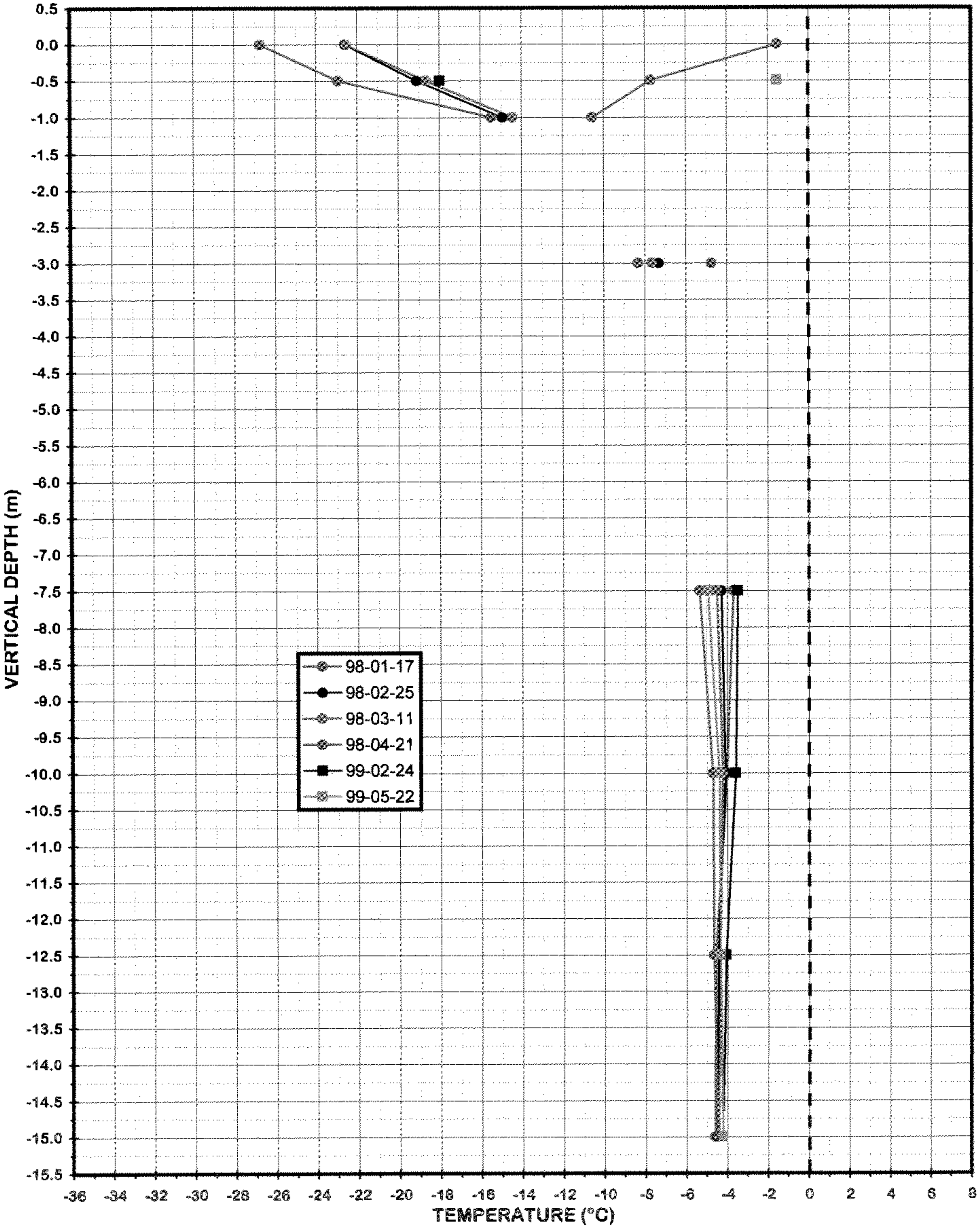


96BOR-38 GROUND TEMPERATURE PROFILE (1996 - 1997 DATA)

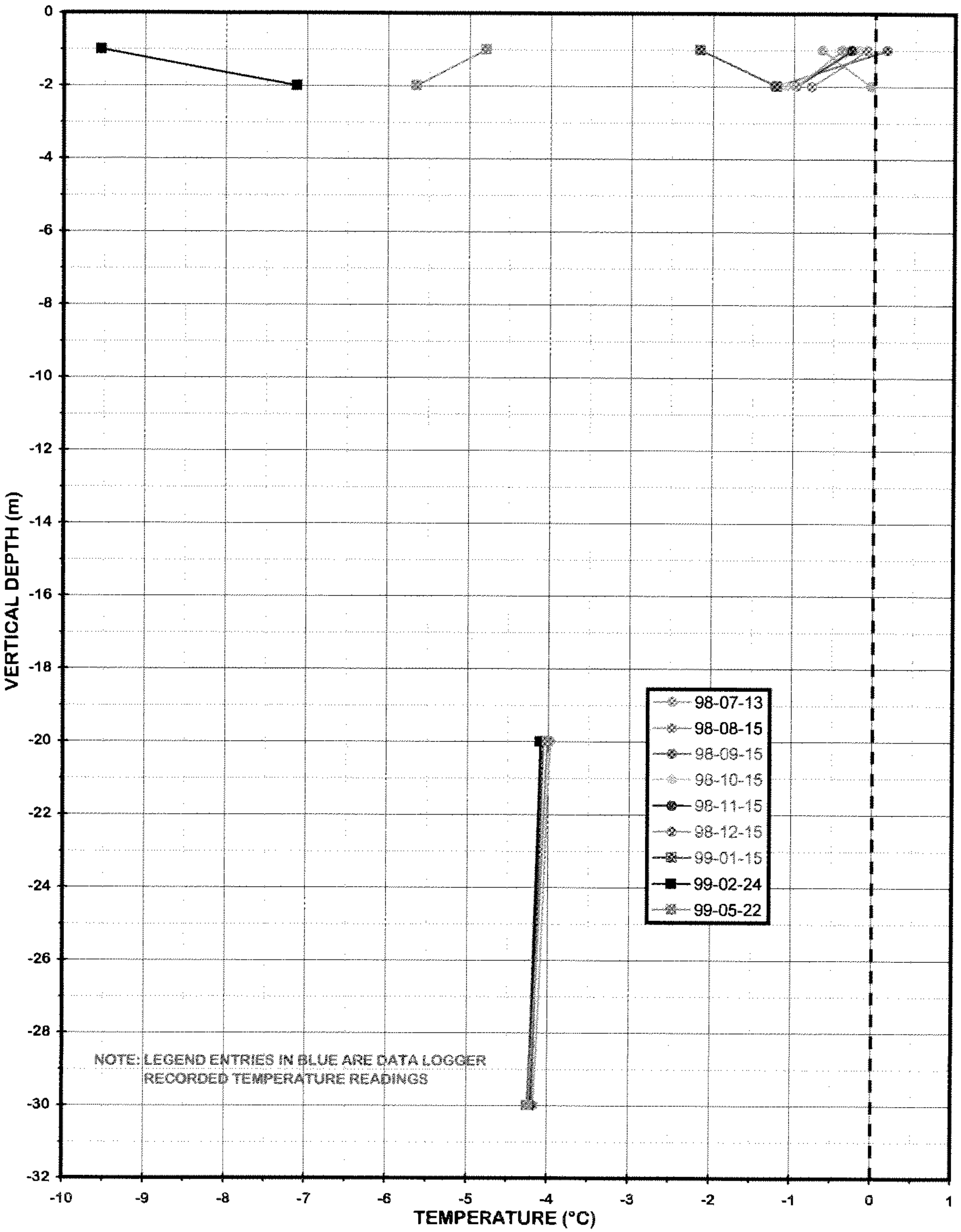


DIAVIK DIAMOND MINES THERMISTOR DATA

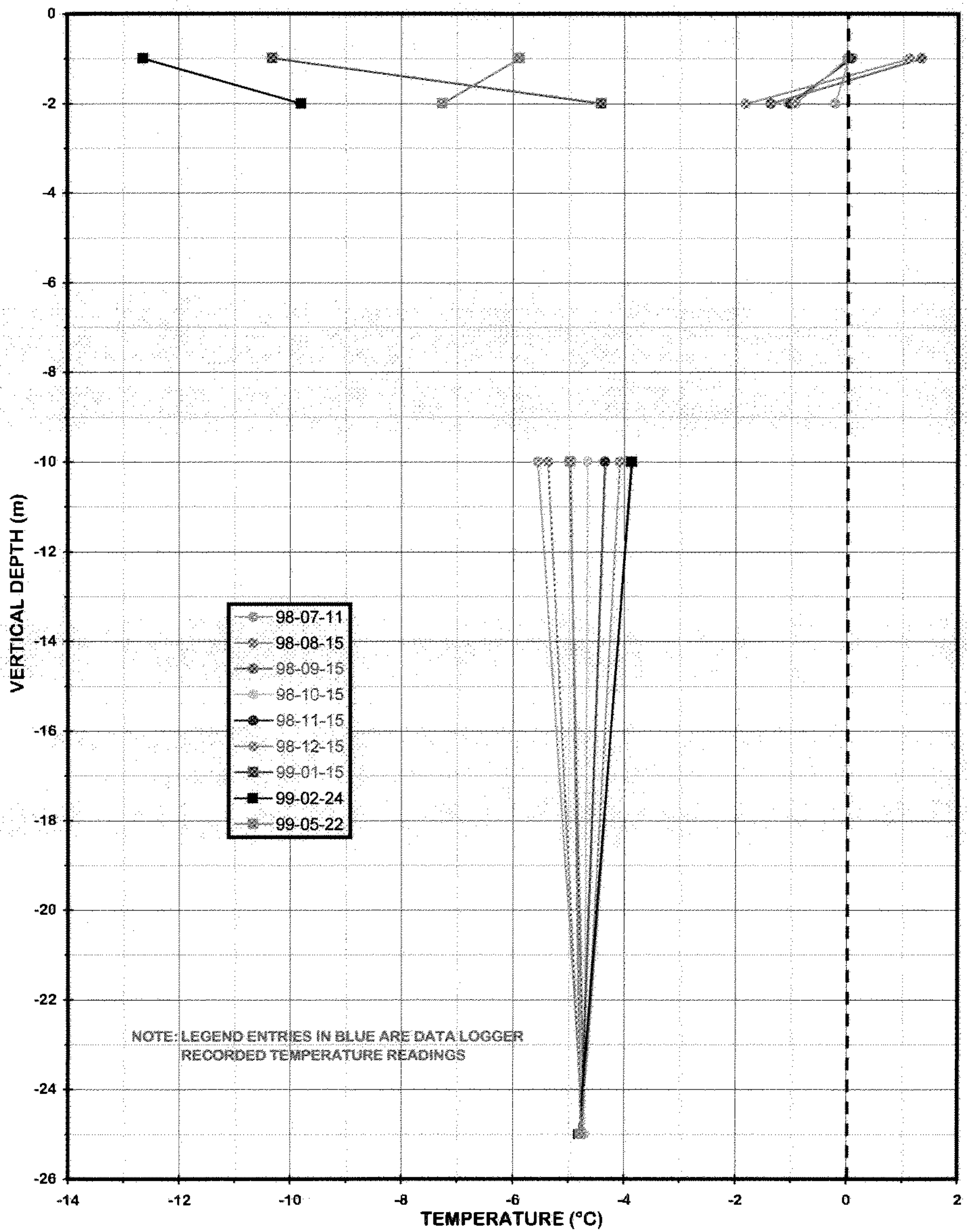
96BOR-38 GROUND TEMPERATURE PROFILE (1998 - 1999 DATA)



98BH-313 GROUND TEMPERATURE PROFILE (ALL DATA)

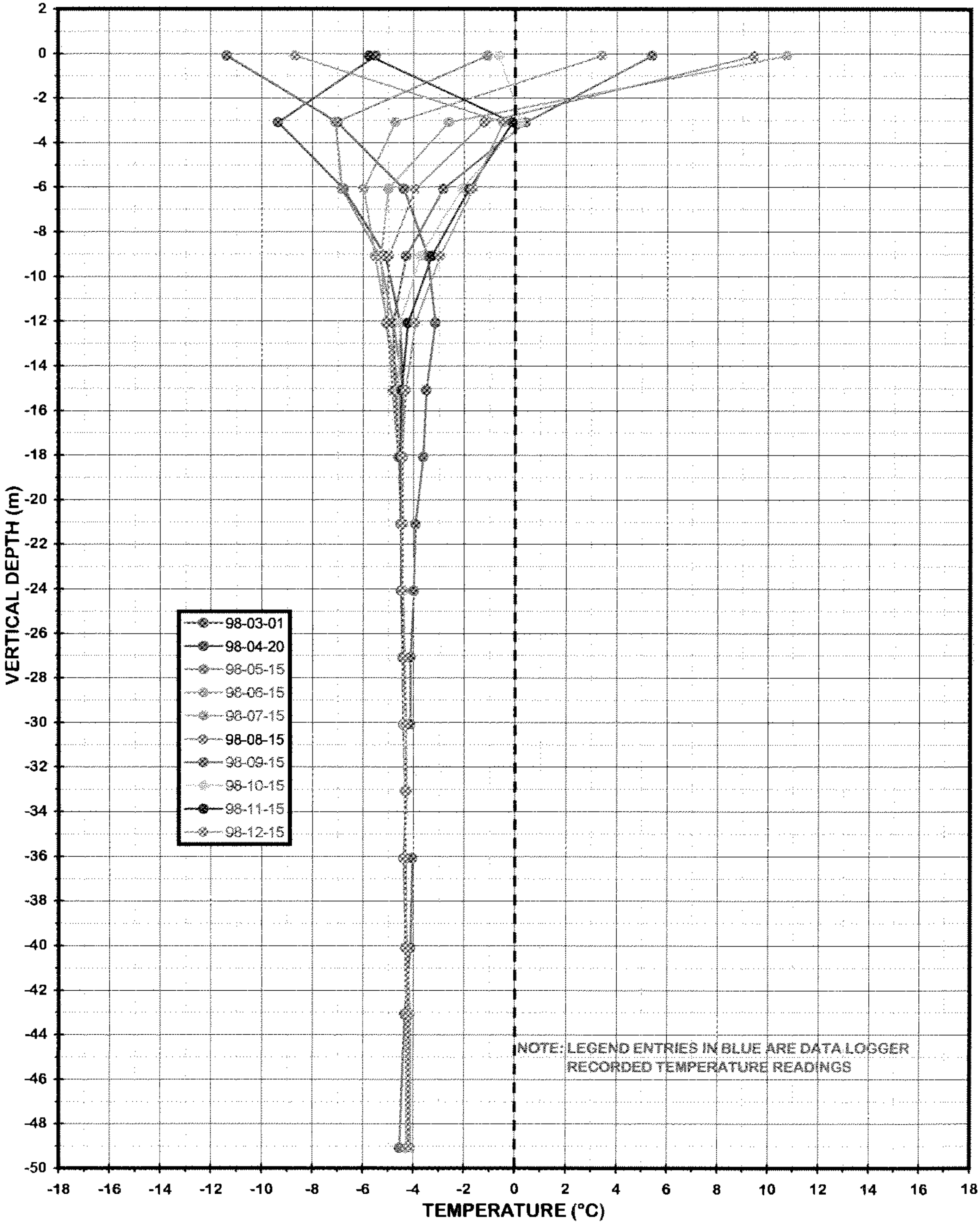


98BH-316 GROUND TEMPERATURE PROFILE (ALL DATA)

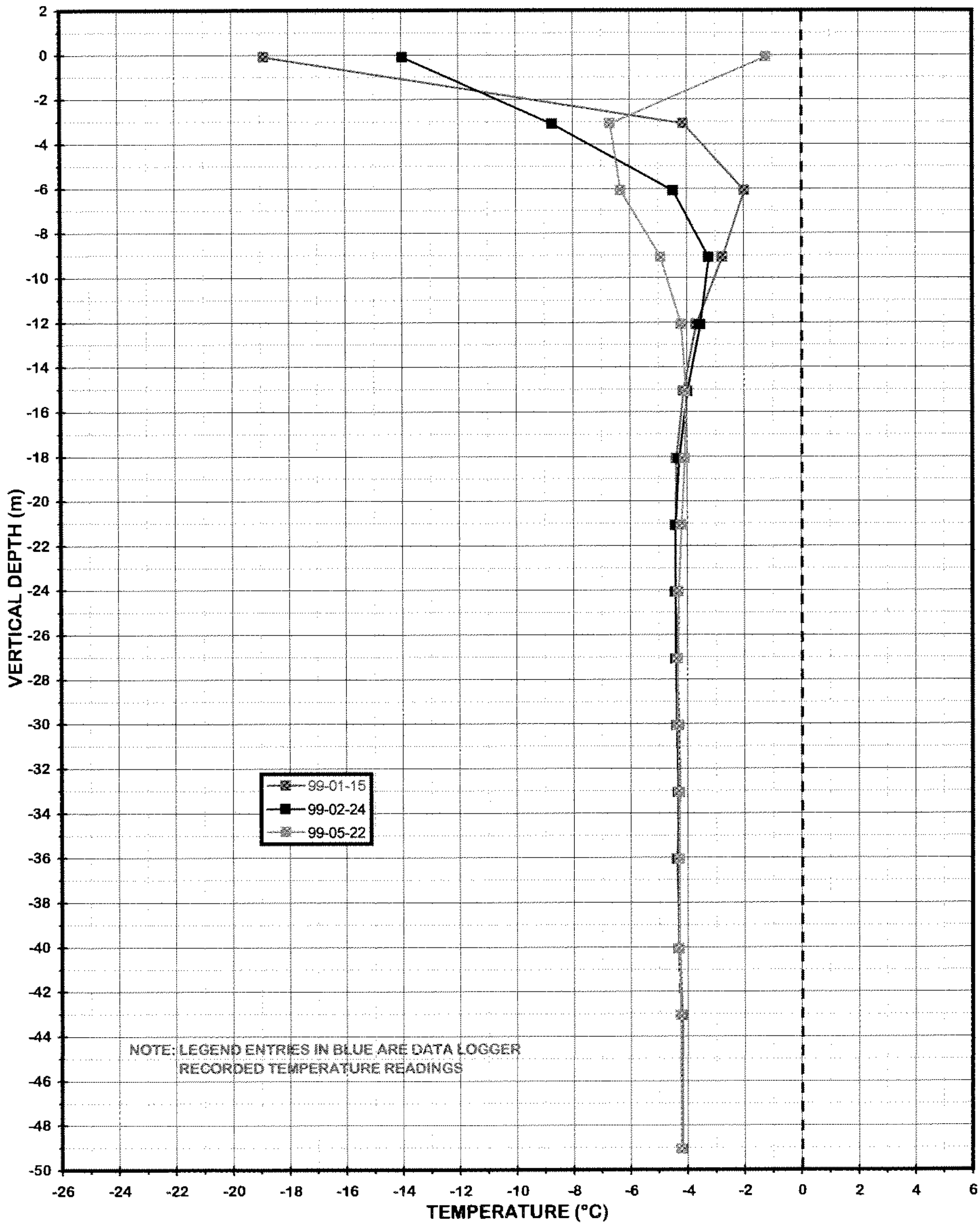


DIAVIK DIAMOND MINES THERMISTOR DATA

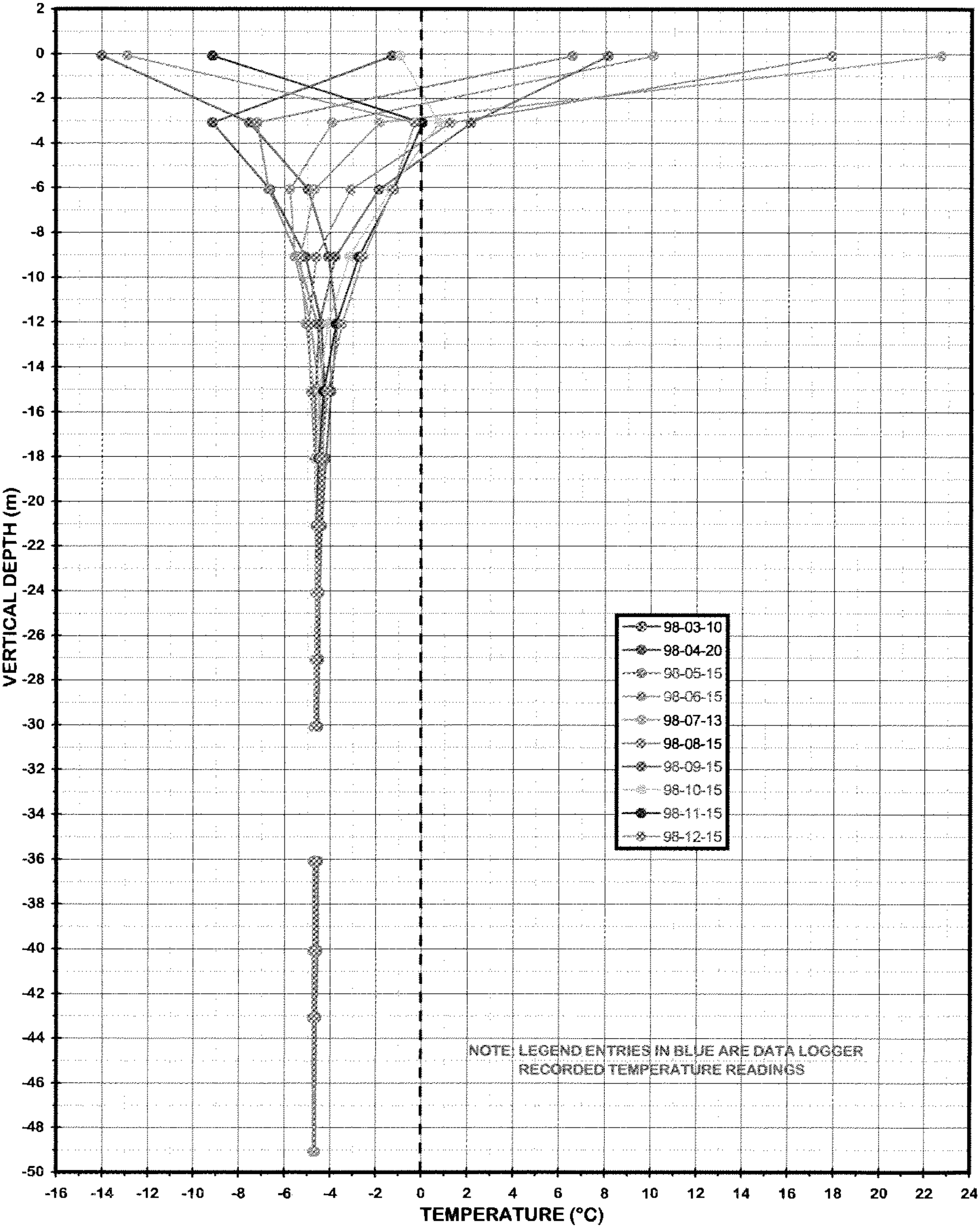
98PKC-04 GROUND TEMPERATURE PROFILE (1998 DATA)



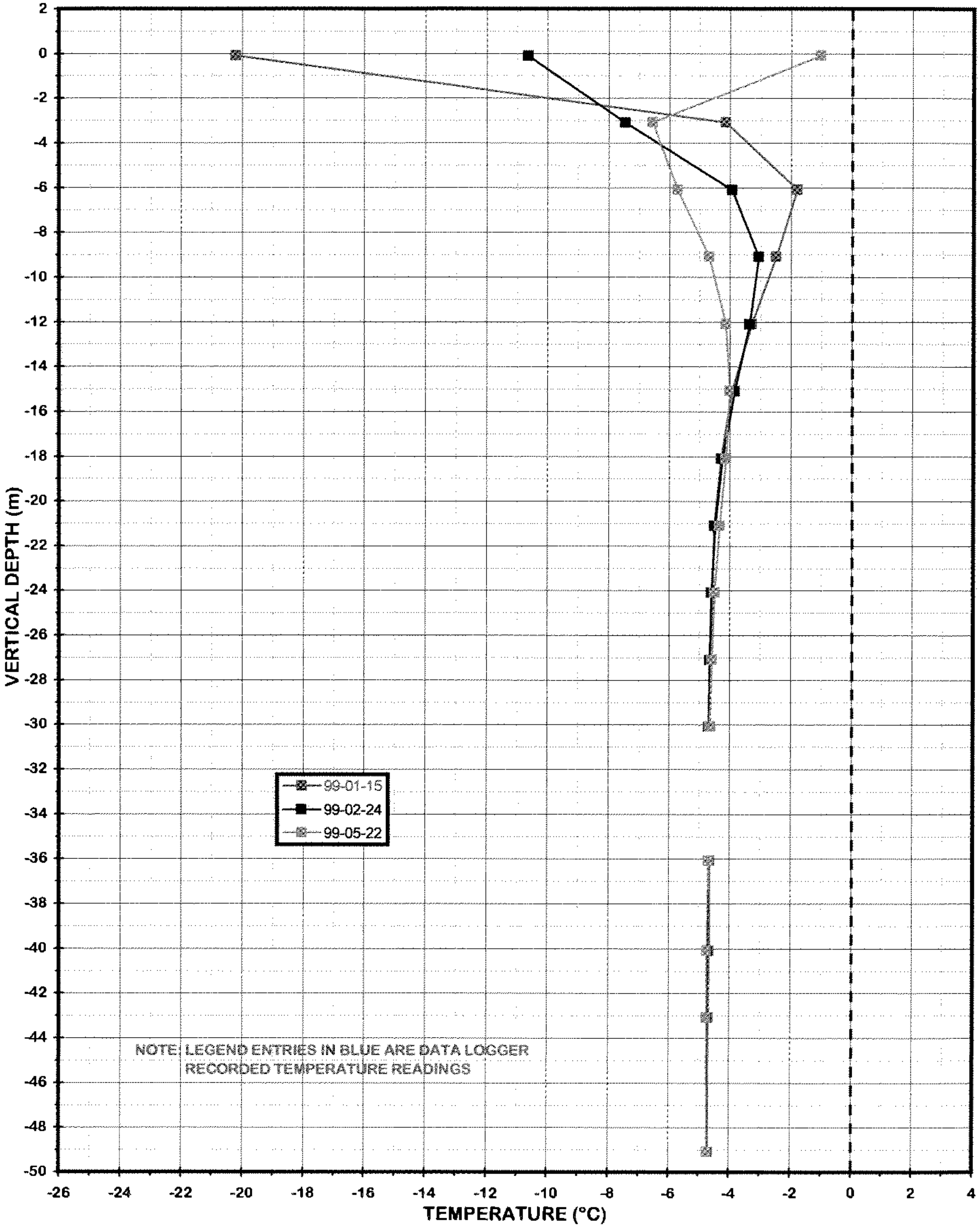
98PKC-04 GROUND TEMPERATURE PROFILE (1999 DATA)



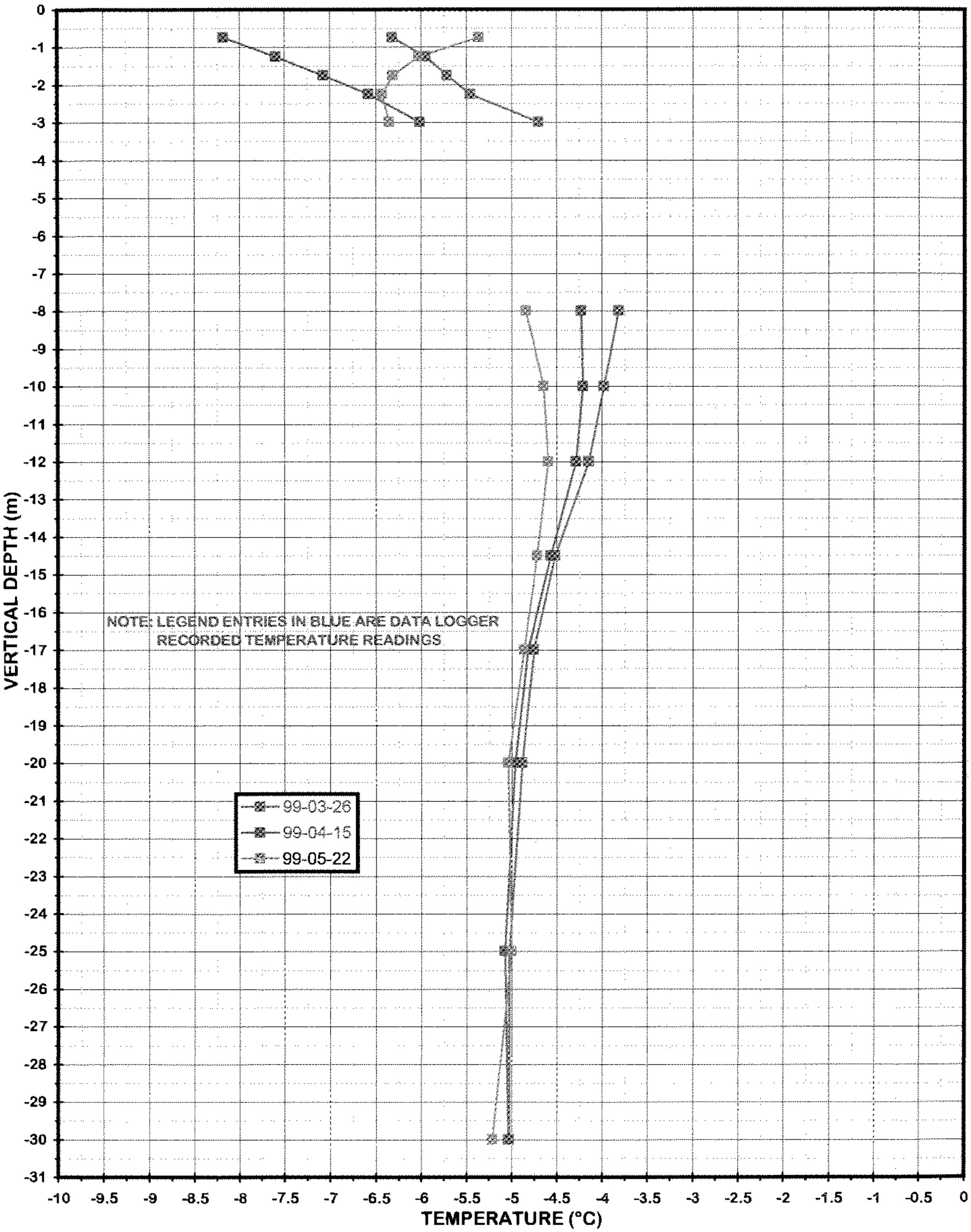
98PKC-05 GROUND TEMPERATURE PROFILE (1998 DATA)



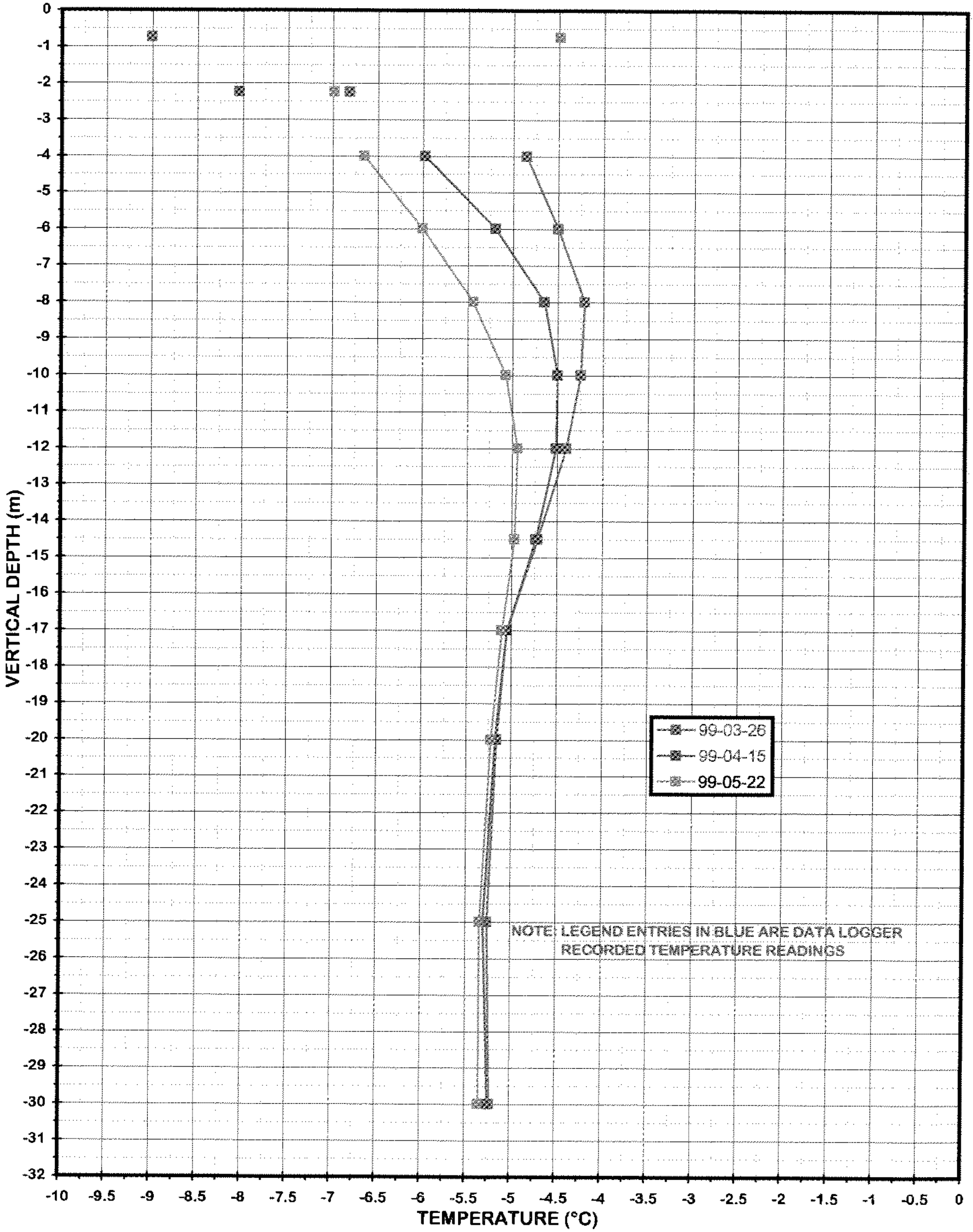
98PKC-05 GROUND TEMPERATURE PROFILE (1999 DATA)



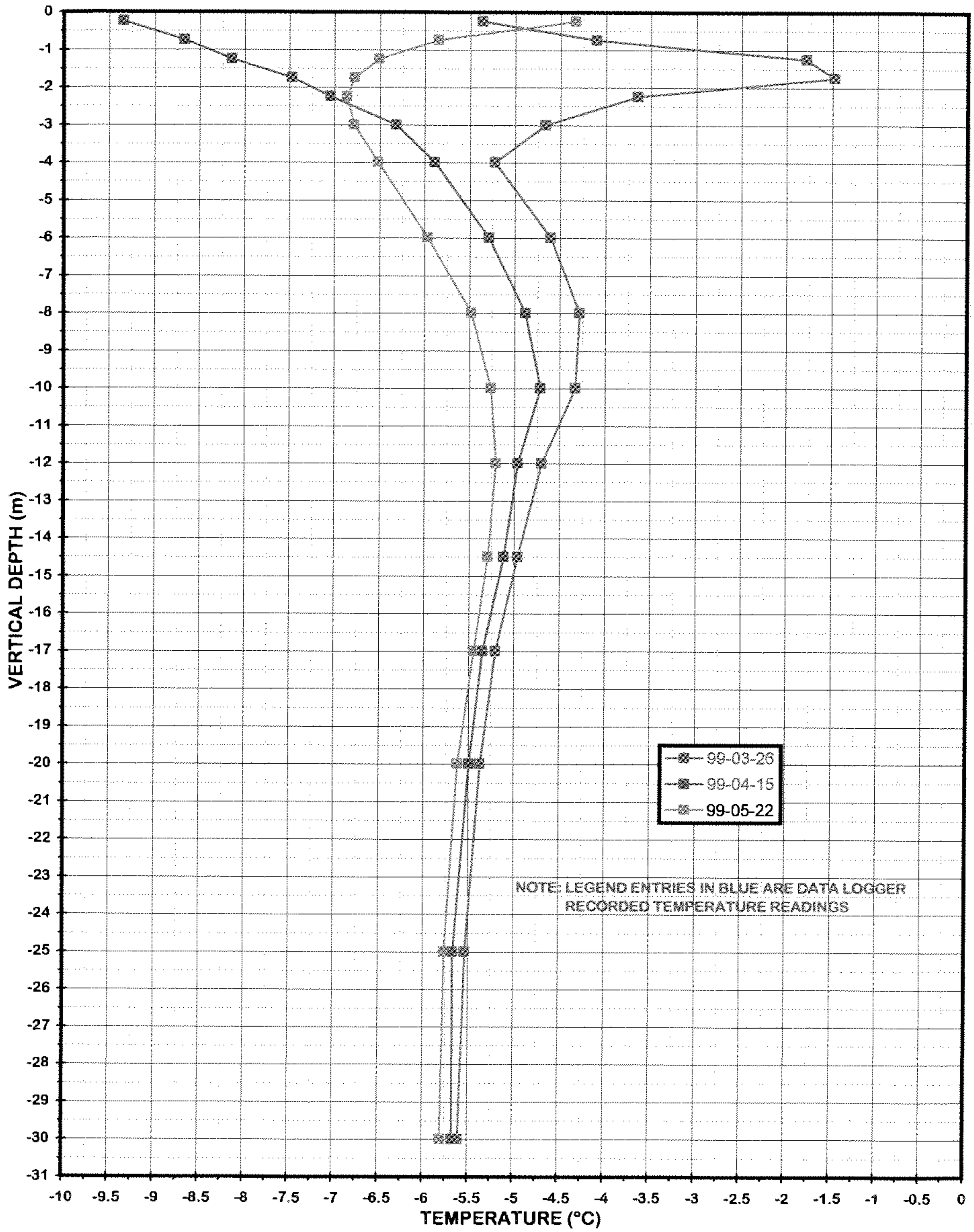
99BH-456 GROUND TEMPERATURE PROFILE (1999 DATA)



99BH-458 GROUND TEMPERATURE PROFILE (1999 DATA)

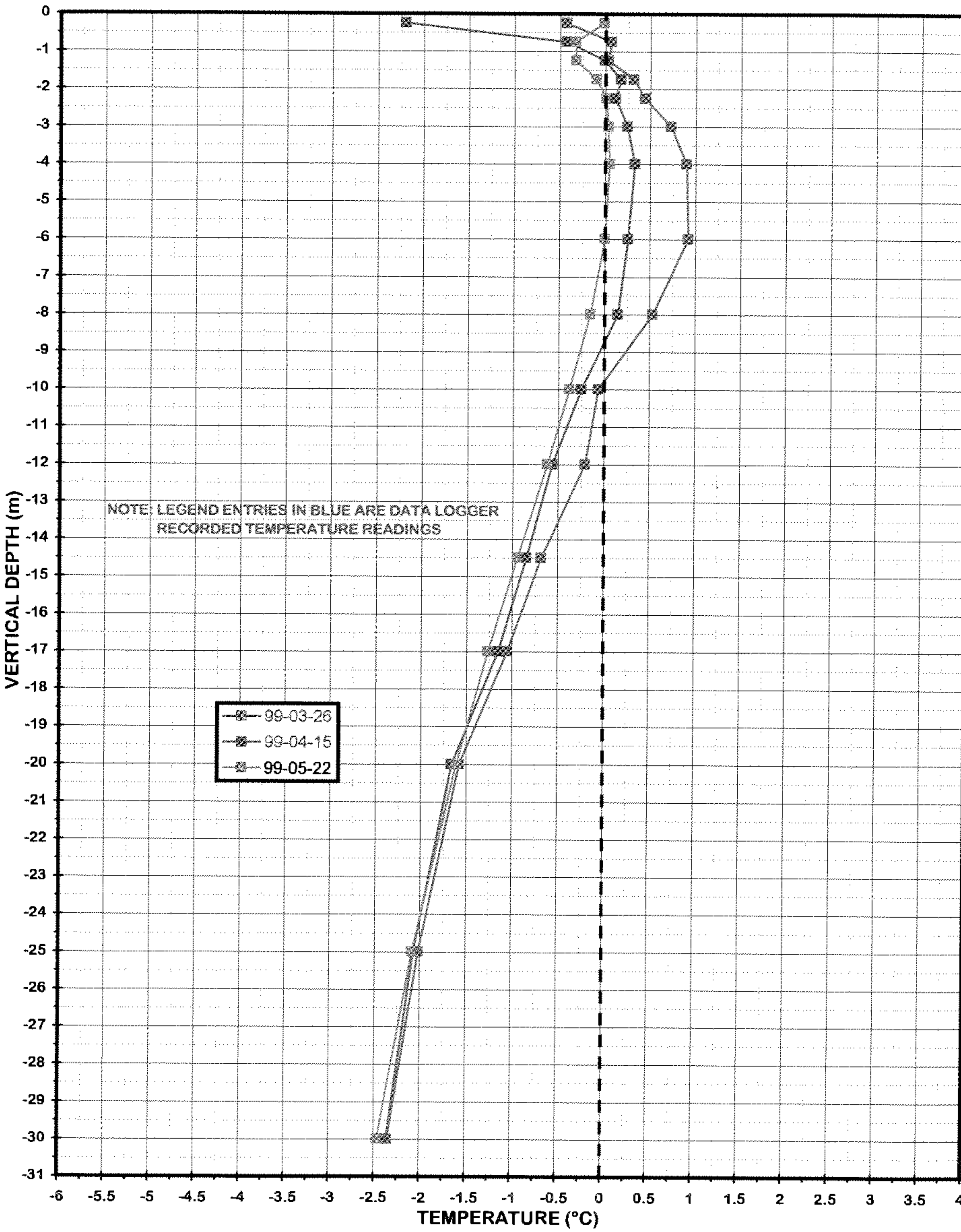


99BH-464 GROUND TEMPERATURE PROFILE (1999 DATA)



DIAVIK DIAMOND MINES THERMISTOR DATA

99BH-498B GROUND TEMPERATURE PROFILE (1999 DATA)



APPENDIX D

2010 National Building Code Seismic Hazard Calculation

2010 National Building Code Seismic Hazard Calculation

INFORMATION: Eastern Canada English (613) 995-5548 français (613) 995-0600 Facsimile (613) 992-8836
Western Canada English (250) 363-6500 Facsimile (250) 363-6565

Requested by: M Wade,

September 21, 2015

Site Coordinates: 64.498 North 110.2892 West

User File Reference: NCRP

National Building Code ground motions:
2% probability of exceedance in 50 years (0.000404 per annum)

Sa(0.2)	Sa(0.5)	Sa(1.0)	Sa(2.0)	PGA (g)
0.095	0.057	0.026	0.008	0.036

Notes. Spectral and peak hazard values are determined for firm ground (NBCC 2010 soil class C - average shear wave velocity 360-750 m/s). Median (50th percentile) values are given in units of g. 5% damped spectral acceleration (Sa(T), where T is the period in seconds) and peak ground acceleration (PGA) values are tabulated. Only 2 significant figures are to be used. ***These values have been interpolated from a 10 km spaced grid of points. Depending on the gradient of the nearby points, values at this location calculated directly from the hazard program may vary. More than 95 percent of interpolated values are within 2 percent of the calculated values.***

Ground motions for other probabilities:

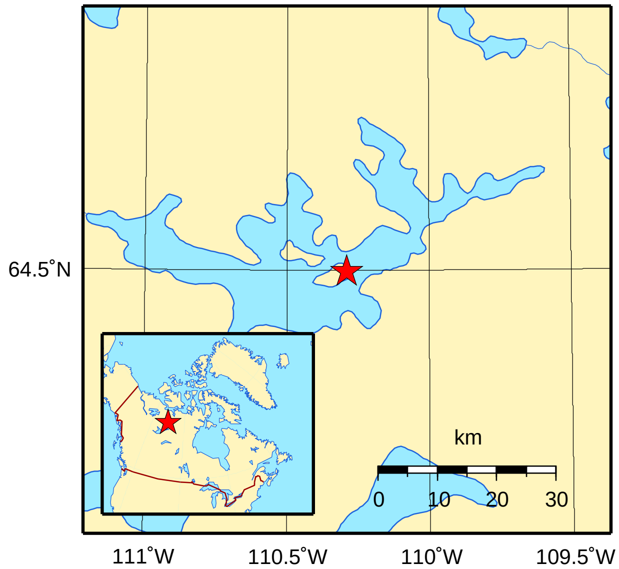
Probability of exceedance per annum	0.010	0.0021	0.001
Probability of exceedance in 50 years	40%	10%	5%
Sa(0.2)	0.011	0.035	0.055
Sa(0.5)	0.007	0.022	0.034
Sa(1.0)	0.003	0.011	0.016
Sa(2.0)	0.001	0.003	0.005
PGA	0.003	0.011	0.019

References

National Building Code of Canada 2010 NRCC no. 53301; sections 4.1.8, 9.20.1.2, 9.23.10.2, 9.31.6.2, and 6.2.1.3
Appendix C: Climatic Information for Building Design in Canada - table in Appendix C starting on page C-11 of Division B, volume 2

User's Guide - NBC 2010, Structural Commentaries NRCC no. 53543 (in preparation)
Commentary J: Design for Seismic Effects

Geological Survey of Canada Open File xxxx
Fourth generation seismic hazard maps of Canada: Maps and grid values to be used with the 2010 National Building Code of Canada (in preparation)



See the websites *www.EarthquakesCanada.ca* and *www.nationalcodes.ca* for more information

Aussi disponible en français

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