BGC ENGINEERING INC.
AN APPLIED EARTH SCIENCES COMPANY

DIAVIK DIAMOND MINES INC.

A21 DIKE

2014 DESIGN REPORT UPDATE

FINAL

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BGC ENGINEERING INC.
AN APPLIED EARTH SCIENCES COMPANY
Suite 800 - 1045 Howe Street
Vancouver, BC Canada V6Z 2A9
Telephone (604) 684-5900
Fax (604) 684-5909

November 28, 2014
Project No.: 1207002-600-01

Gord Macdonald, Project Manager
Diavik Diamond Mines Inc.
Box 2498, 5007 - 50th Avenue
Yellowknife NT X1A 2P8

Dear Mr. Macdonald,

Re: A21 Dike - 2014 Design Report Update (FINAL)

Please find attached a copy of the above referenced FINAL report dated November 28, 2014, presenting the updated design for the A21 dike. Should you have any questions or comments, please do not hesitate to contact the undersigned.

We appreciate the continued opportunity to take part in the development of the A21 Project.

Yours sincerely,

BGC ENGINEERING INC.
per:

Ken Halisheff, P.Eng.
Project Manager

EXECUTIVE SUMMARY

This report presents the updated final design undertaken for the proposed A21 water-retaining dike at the Diavik Diamond Mine, and updates and supersedes previous design reports issued by AMEC in 2007 and 2012. This update is necessitated by a modification to the method of cut-off wall construction, the gradation specification for the central core zone of the dike, design optimizations in the shallow water and abutment sections of the dike, and the assumption of the role of Engineer-of-Record for the A21 dike by BGC Engineering Inc. (BGC).

The A21 dike will be the third water-retaining dike constructed within Lac de Gras to facilitate open pit mining of a diamondiferous orebody, in this case the A21 kimberlite pipe. The A154 and A418 kimberlite pipes are located within the waters of Lac de Gras, just offshore of East Island. Open pit mining of each pipe required the construction of water-retaining dikes to isolate the pipes and allow dewatering of the areas enclosed by the dikes to facilitate mining. Open pit mining of both pipes is complete since mid-2012, and all mine production is currently via underground mining. Development of the A21 kimberlite pipe, which is planned to be via open pit methods only, will supplement the ore from the underground mining of A154 and A418.

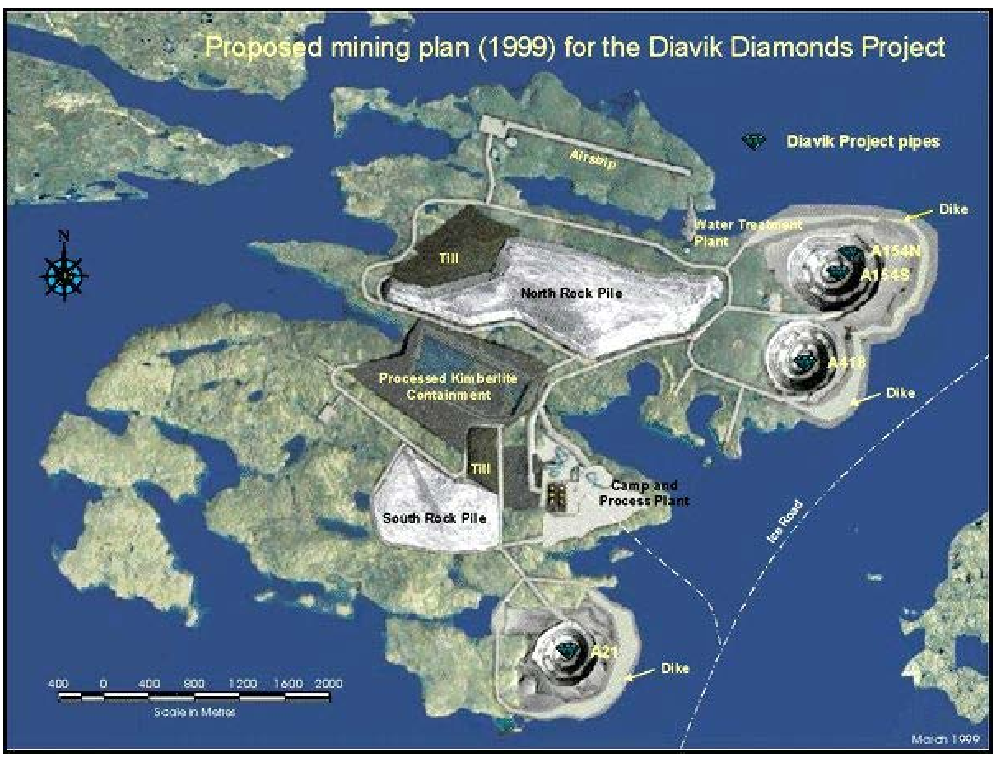

Figure 1. Overall plan of the Diavik site.

The A21 pipe is located to the south of East Island, within Lac de Gras. Open pit mining of this pipe will require construction of the A21 dike. The original (1999) concept for the A21 dike alignment is illustrated above; it included construction of the dike in water depths 10 m greater than the maximum depth encountered at the previous two dikes. A 2006-07 feasibility study of the A21 project resulted in selection of an alignment in shallower water, tighter to the A21 pipe. That feasibility study was updated by AMEC (2012), to incorporate a modified pit design, which required a minor adjustment to the alignment of the A21 dike. A further update to the A21 feasibility study is being undertaken in 2014, which incorporates some design and construction methodology changes for the A21 dike that are documented herein.

The design of the A21 dike builds upon the designs and experiences of the A154 and A418 dikes. The performance of those dikes, designed by Nishi-Khon/SNC Lavalin (NKSL), and constructed by Lac de Gras Constructors (LDG) and their various specialty sub-contractors, has met design expectations. Those successes, and the lessons learned, provide a solid precedent for the design of the A21 dike as documented herein, particularly given the shallow water prevailing for most of the A21 dike alignment. The design, construction and monitoring of the A154 and A418 dikes were undertaken with the benefit of third party review provided by the Diavik Geotechnical Review Board (DGRB). The DGRB was re-formed into the Diavik A21 Geotechnical Review Panel (DGRP) for support of the A21 project studies, and has been engaged in the A21 dike design studies since the 2006-07 A21 feasibility study.

The design of the A21 dike is briefly summarized as follows:

- Foundation preparation comprising:
    - Stripping of ice-rich soils below the on-land portion of the dike
    - Dredging of soft lakebed sediments within the deeper water portions of the dike alignment with sufficient draught to accommodate dredging
    - Removal of lakebed sediments and boulders in the shallows where the draught is insufficient for dredge operation
    - Removal of boulders along the cut-off alignment and below the filter blanket area.
- A zoned rockfill embankment constructed to enclose the A21 kimberlite pipe, that will include:
    - Upstream and downstream shells of mine-run (900 mm minus) and 200 mm minus crushed mine rock, respectively
    - A central core of crushed 50 mm minus rock, most of which will be subsequently vibro-densified so that the core fill will provide adequate support for the cut-off wall
    - A filter blanket of crushed 56 mm minus rock placed via clamshell below the downstream shell of the dike to provide protection against seepage-induced internal erosion of the lakebed till as a result of seepage through and below the cut-off wall.

- Installation of the cut-off wall element of the dike, comprising:
  - Drilling through the vibro-densified core fill with large diameter, over-lapping boreholes extending a nominal 3 m into the underlying lakebed till, and backfilling of these holes with a 12.5 mm minus graded crush
  - Construction of a plastic concrete diaphragm wall within the pre-drilled zone via the cutter soil mixing (CSM) method
  - A single row grout curtain extending a minimum 15 m into bedrock
  - Jet grouting overlapping with the bottom 1 m portion of the plastic concrete CSM wall and the upper 1.5 m of the bedrock grout curtain, to complete the cut-off.
- Abutment and on-land details, comprising:
  - Thermosyphon groups at the talik-permafrost contacts to extend permafrost and cool marginal permafrost to effectively key the cut-off wall into frozen ground
  - Cut-off trenches and freeboard dike on land.

The figure below illustrates the basic dike design section as constructed in the lake for the A154 and A418 dikes, and as planned for A21. This figure does not include the downstream toe berm, which will be constructed in the dry subsequent to pool dewatering, nor is it illustrative of the on-land dike sections.

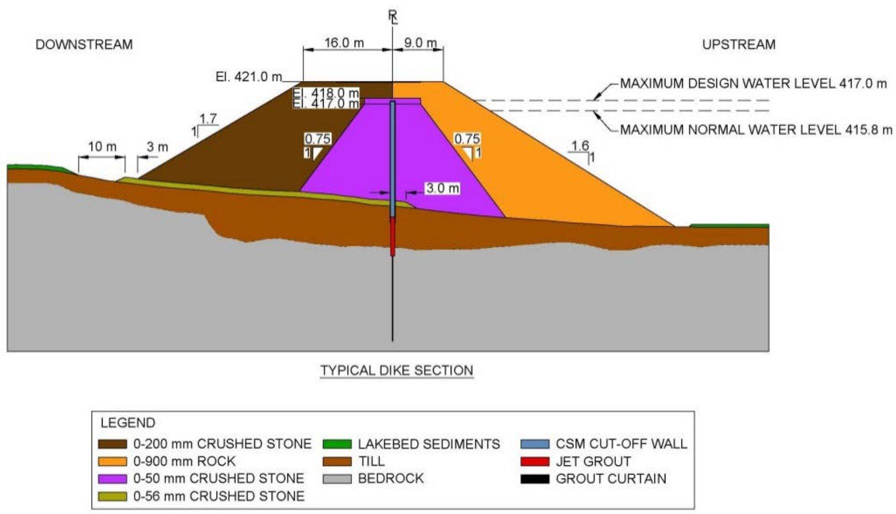

Figure 2. Typical dike embankment section.

Major dike construction is scheduled to commence in 2016; crushing, stockpiling, and development of supporting infrastructure, and initiation of dike embankment construction at the abutments, will commence in 2015. With a two year construction schedule, dewatering of the A21 pool is scheduled to be carried out in the winter months of 2017-2018, allowing for pit pre-stripping to commence in 2018.

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LIMITATIONS

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1.0 INTRODUCTION

1.1. General

Diavik Diamond Mines Inc. (DDMI) is currently mining the A154N, A154S, and A418 kimberlite pipes via underground methods. The Diavik kimberlite pipes, including the A21 kimberlite pipe, were discovered in 1994/5, with all four being located adjacent to East Island in Lac de Gras. Project approval was received in 1999 and construction of the A154 dike began in 2001 with first diamond production from A154S and A154N kimberlite pipes towards the latter part of 2002. Construction of a second dike for the A418 kimberlite pipe was approved in 2004, with construction completed late in 2006 and A418 ore production commencing early in 2008. Underground production commenced in March 2010. As of mid-2012, open pit mining of the A154 and A418 pipes had been completed, and production will continue by solely underground methods until the A21 open pit comes on stream, scheduled for 2018.

The A154 and 418 dikes were designed by Nishi Khon-SNC Lavalin (NKSL, 1999, 2004). Both the dikes have performed in accordance with design expectations.

The 1999 feasibility study and development plan for the Diavik project included the open pit mining of the fourth pipe, designated A21. This pipe, due to project economics, was subsequently reduced from reserve to resource status. Specifically, the size of the pipe was smaller than the others, but required, per the 1999 feasibility study, the largest of the in-lake dikes to exploit it. The cost associated with that dike rendered the project uneconomic.

DDMI undertook a feasibility study in 2006-2007 that had, as its objective, upgrading the status of A21 from resource to reserve, and developing an economic project such that the A21 pipe could be mined via both open pit and then underground methods. In that study, the size of the open pit was reduced, allowing a smaller, less costly dike constructed in shallower water. DDMI updated that feasibility study in 2012, incorporating a revised open pit design, minor adjustments to the A21 dike alignment to suit, and elimination of plans for underground mining of A21 once the pit was completed.

The design of the A21 dike as developed in 2007 is documented by AMEC (2007). That report was submitted to the regulatory agencies, and received the requisite regulatory approvals and permits for construction. The design report was updated by AMEC (2012), and again DDMI obtained the requisite regulatory approvals and permits for construction.

In 2013, DDMI initiated additional studies to optimize the A21 project relative to the 2012 feasibility study configuration, culminating in an update of the feasibility study in mid-2014. The project optimizations included modifications to the design and construction methodology for the A21 dike, which are presented herein. This report presents the updated design for the proposed A21 water retention dike, and updates and supersedes the design as presented in the AMEC (2007, 2012) reports. With this updated design report, BGC Engineering Inc. (BGC) assumes the role of Engineer-of-Record for the A21 dike. However, most of the analytical work undertaken in 2007 remains valid for, and suitably representative of, the updated dike design presented herein, and is therefore included in appendices for ease of reference.

1.2. Project Background

1.2.1. 1999 Feasibility Study

The A21 pipe development as originally envisioned in the 1999 feasibility study (and as illustrated in Figure 1-1) involved an open pit to an approximate depth of 200 m below the lake level (see Figure 1-2). This pit shell, in conjunction with the pit crest to dike toe setback of 100 m (the nominal design criteria incorporated for the A154 and A418 dikes) resulted in the dike alignment as shown on the two figures.

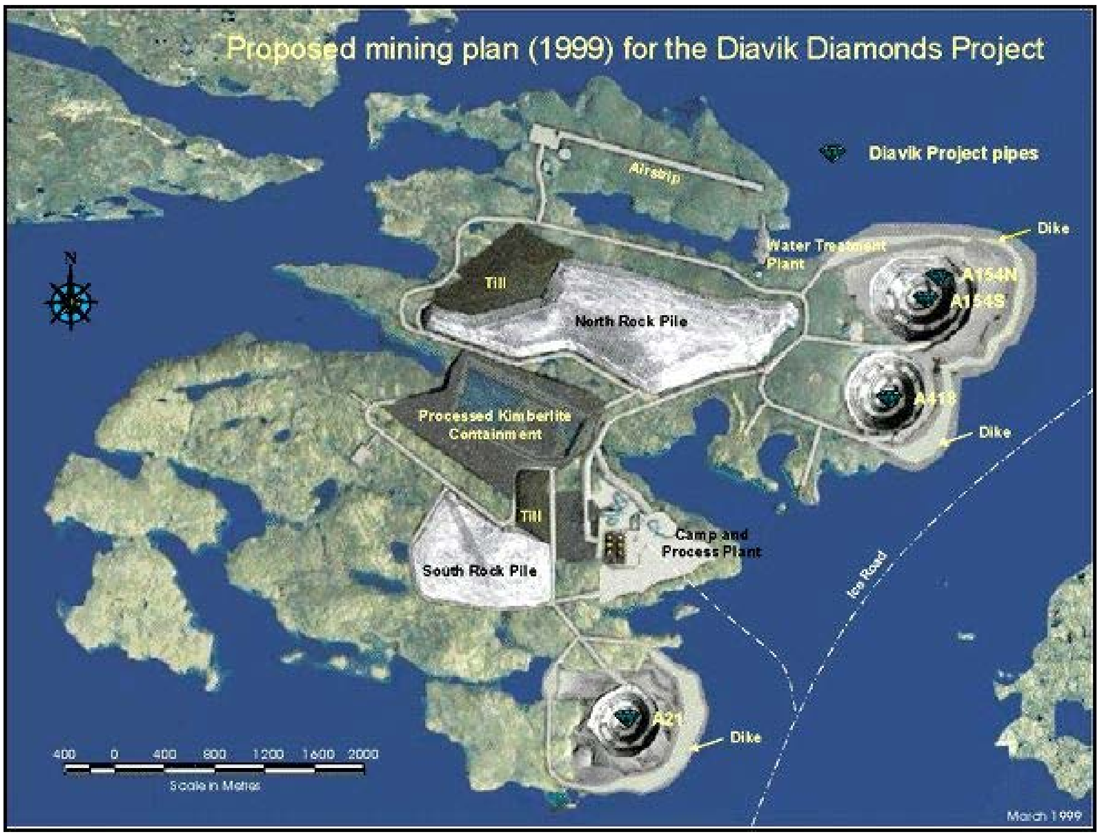


Figure 1-1. General Diavik site layout, showing A21 pit and dike alignment as envisioned in the 1999 feasibility study.

Figure 1-2. 1999 feasibility study A21 dike alignment (isometric).

1.2.2. 2006-2007 Feasibility Study

The deep water alignment as illustrated in Figure 1-1 and Figure 1-2 rendered the A21 project uneconomic. The dike design presented in AMEC (2007), represented an optimization of:

- The dike alignment
- The open pit geotechnical design
- The setback between the dike and the ultimate projected open pit rim.

The A21 dike alignment as laid out in AMEC (2007) is presented on Figure 1-3. The A21 dike was to be approximately 2,200 m in length - nearly 600 m shorter than the 1999 alignment. The basis of the alignment was to minimize the size and cost of the dike by adhering to a shallower water alignment to the maximum extent possible. The dike alignment provided a constraint for the open pit development, effectively limiting the maximum achievable depth of the pit to about El. 240 m, about 176 m below the typical level of Lac de Gras.

The open pit geotechnical design for A21 for the 2007 feasibility study was undertaken by Golder Associates Ltd. (Golder, 2006a). The optimization of the dike alignment and pit design, along with a reduction of the dike toe to pit rim setback from a nominal 100 m (A154 and A418 criteria) to 50 m (A21 criteria) resulted in an A21 dike alignment in shallower water than its A154 and A418 predecessors. This shallower water alignment, together with the experience gained from the construction of the A154 and A418 dikes, and the monitored performance of the A154 dike, permitted some minor optimizations of the dike design relative to the A154 and A418 dikes, as documented in AMEC (2007), and carried forward in this updated design report.

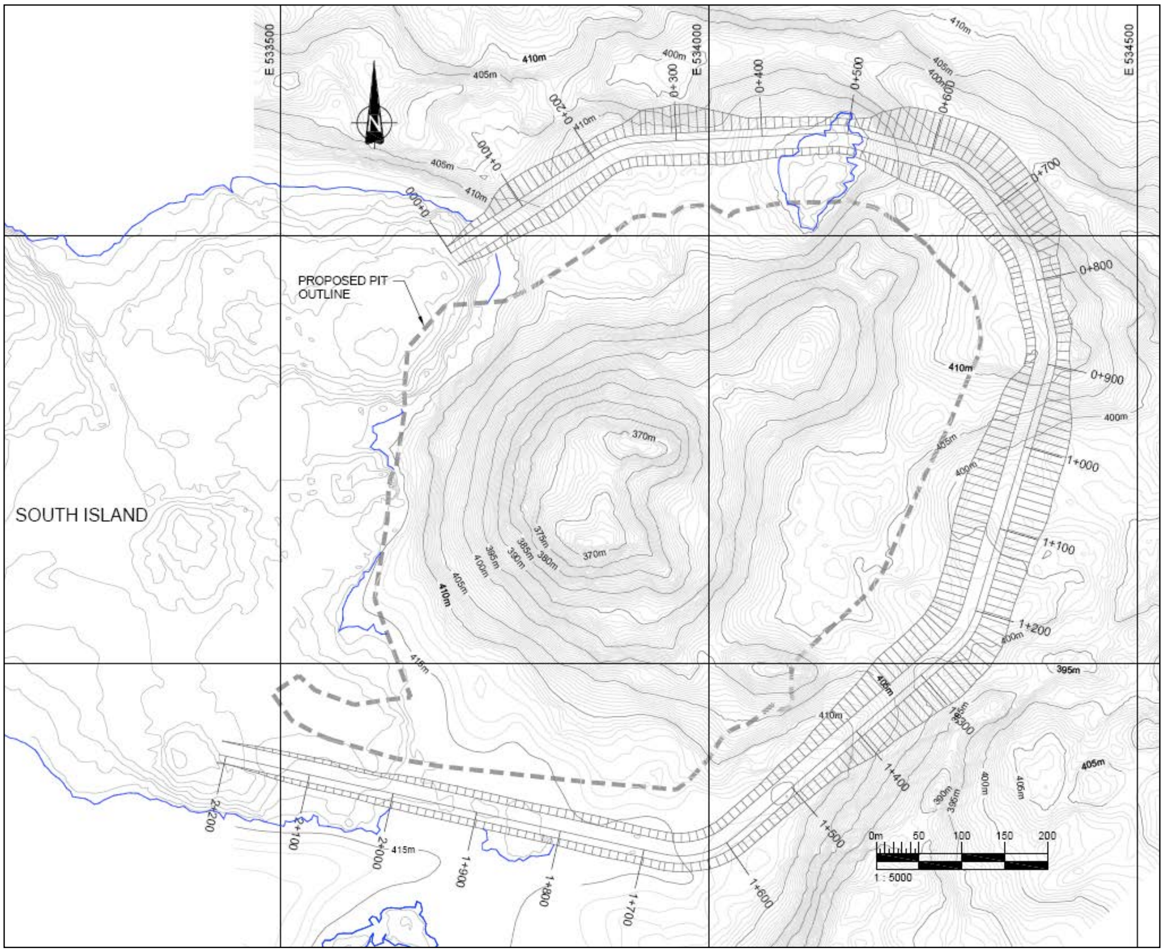


Figure 1-3. 2007 A21 dike alignment and open pit outline.

1.2.3. Project Suspension and Studies 2008-2011

Subsequent to the completion of the 2007 feasibility study, and the completion of the regulatory submittals and approvals for construction of the A21 dike as per the AMEC (2007) design, the project was suspended. The suspension was the result of ore values being considered uneconomic relative to the project's capital cost expenditures (dominated by the A21 dike) on the basis of the underground bulk sample extracted from the A21 pipe.

DDMI thereupon undertook studies of alternative mining methods for exploitation of the A21 pipe, along with large diameter drilling and sampling of the A21 pipe to better characterize the resource. In 2011, a pre-feasibility study for the A21 project was undertaken by DDMI in conjunction with an updated A21 open pit design effort, and completed in the second quarter of 2012, whereupon DDMI immediately launched an updated feasibility study for the A21 project.

1.2.4. 2012 Feasibility Study Update

As part of the pre-feasibility study completed in 2012 DDMI had developed an integrated economic model that allowed for determining the optimal configuration of the A21 pit and dike system. Moreover, with the elimination of future underground mining from the A21 project, the pit design was able to eliminate allowance for access to a portal that had been provided in the 2007 pit design. This allowed for a significant economic improvement for a revised pit and dike configuration. This configuration involved a realignment of the A21 dike, as shown in Figure 1-4 (with the 2007 alignment also shown for comparison, and arrows indicating areas of significant alignment shifts). As shown in Figure 1-4, relative to the 2007 alignment, the 2012 dike alignment extends further out into Lac de Gras along the east-northeast perimeter, the deepest water portion of the dike, while it has been shifted inwards near the south abutment and the north abutment, in more shallow waters. Figure 1-5 shows the range of lake depths along the current dike alignment.

Although the net result of this alignment shift is a slightly larger A21 dike, DDMI determined this to be a net benefit to the overall project owing to increased ore recovery, with the pit bottom now being planned at El. 230 m, which is 10 m lower than the 2007 open pit design. Table 1-1 provides a comparison of key features for the 2007 and the 2012 alignments.

Table 1-1. Comparison of 2007 and 2012 A21 dike alignments.

Alignment Feature	2007 Alignment	2012 Alignment
Length of dike (m)	2,200	2,228
Maximum water depth along cut-off wall alignment (m)	19	20
Maximum water depth to lakeside dike toe (m)	20	22.5
Maximum water depth to pit-side dike toe (m)	17	18
Embankment volume, not including toe berm (Mm^3)	1.14	1.37
Pool volume to be dewatered inside completed dike (Mm^3)	6.9 *	6.7 *
Minimum horizontal setback between the dike and the pit rim, as derived from 2H:1V projection of pit-side dike toe to bedrock, and pit rim in bedrock	50	50

* Pool volumes inclusive of precipitation during dewatering, entrapped water in dike rockfills, seepage, and sediments removal.

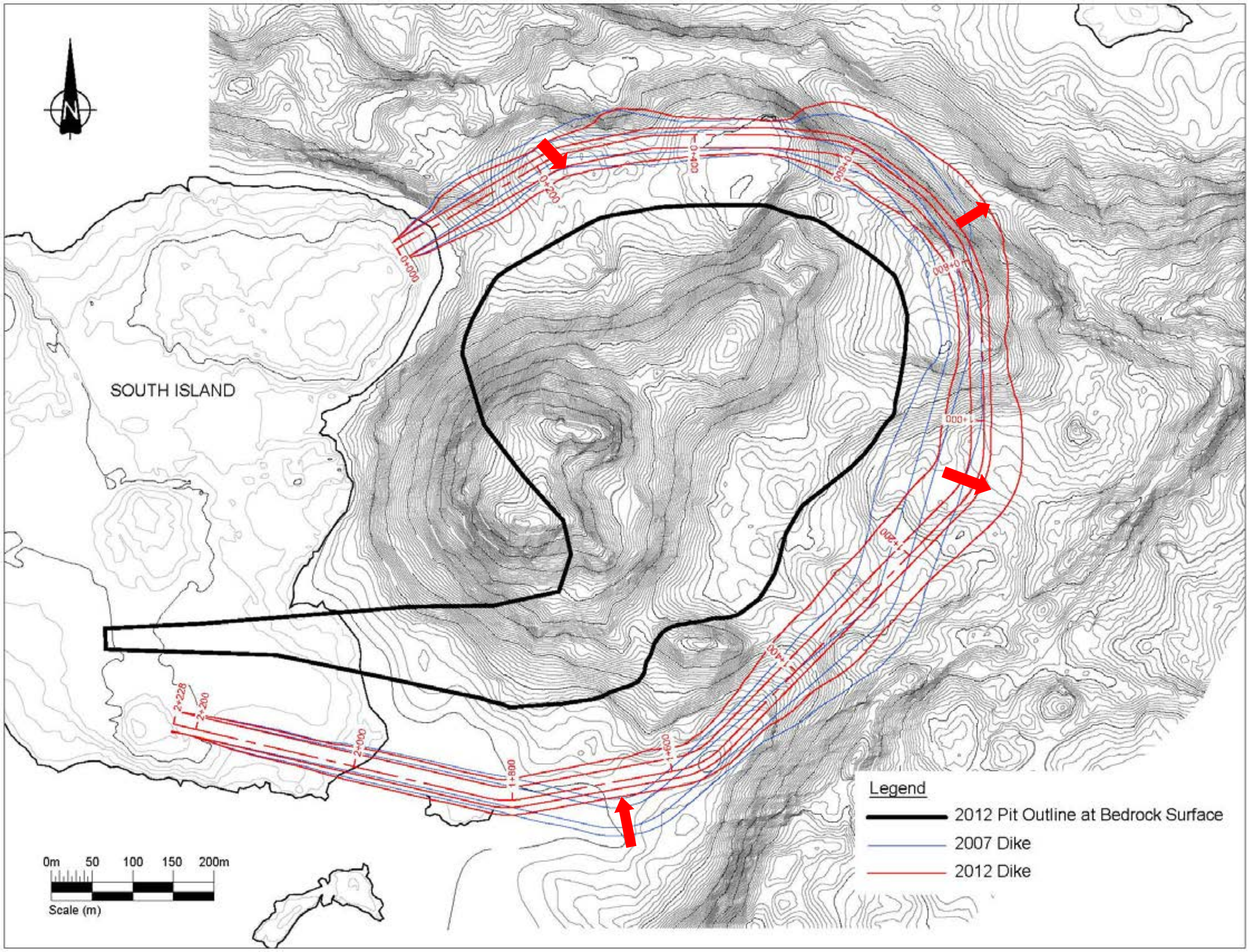


Figure 1-4. Comparison of 2007 and 2012 (current) A21 dike alignments.

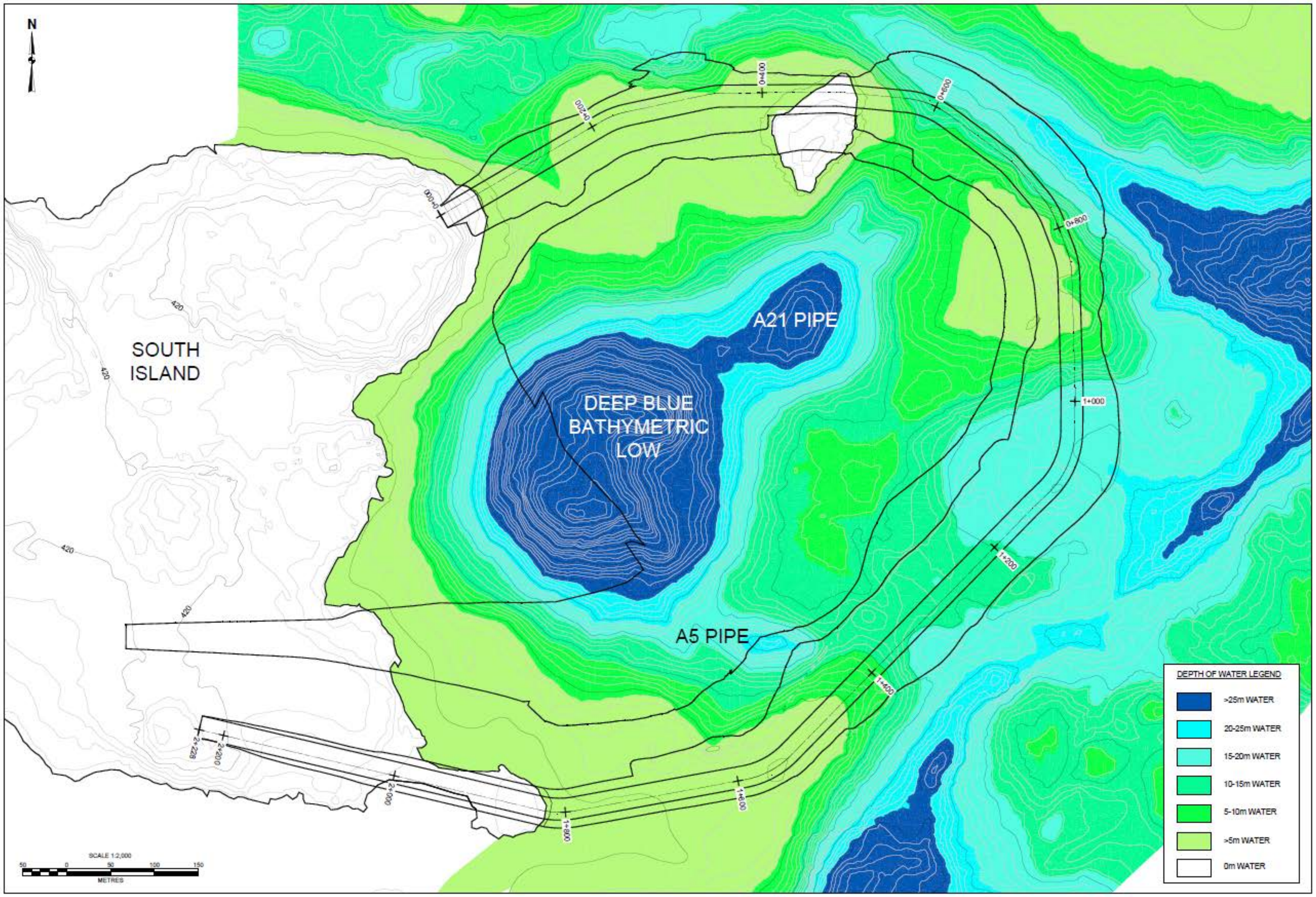


Figure 1-5. Current A21 dike alignment and lake depths.

The A154 dike was constructed within lake depths of up to 25 m, and the A418 dike within lake depths of up to 35 m, greater than the water depths applicable for the A21 dike and cited in Table 1-1.

The 2012 feasibility update indicated the economics of the A21 project still to be marginal. As a result, towards the end of 2012, DDMI undertook value engineering efforts to reduce costs for various elements of the overall A21 project, including the A21 dike. For the A21 dike, the cost reduction opportunities identified for further evaluation were:

- Use of cutter soil mixing (CSM) in combination with large diameter drilling for construction of the upper portion of the cut-off wall, representing a change in methodology but retaining the same performance criteria as established for the previous Diavik dikes
- Modification of the gradation specification for the central core zone of the dike to allow production using the Diavik crusher
- Potential alternatives to dredging as a means of lakebed sediments removal along the dike alignment.

1.2.5. 2013 Studies

In 2013, DDMI undertook additional value engineering studies and refinements for the overall A21 project. BGC supported this effort via further evaluation of the three design optimizations listed above. A meeting with the DGRP was held in Edmonton in December 2013 to review the work then completed, and to discuss future work requirements.

1.2.6. 2014 Feasibility Study and Dike Design Update

At the beginning of 2014, DDMI began the process of updating the overall A21 project feasibility study, which was completed in July 2014. In support of this effort, BGC updated the design for the A21 dike, which is documented herein. This report updates and supersedes the AMEC (2007, 2012) A21 dike design reports.

1.3. Organization of Report

This report is organized as follows:

- Section 2: Describes site conditions
- Section 3: Presents A21 dike design criteria and parameters
- Section 4: Describes the A21 dike embankment design and construction
- Section 5: Provides the design and construction methodology for the A21 dike cut-off wall, the abutment dike sections and transitions, and thermal design aspects
- Section 6: Describes the dike construction sequence and overall schedule
- Section 7: Outlines the plan for the dewatering of the pool once the A21 dike is completed

- Section 8: Summarizes the geotechnical, thermal, hydrogeologic, and hydrologic analyses undertaken in support of the A21 dike design. The analyses themselves are provided in the appendices
- Section 9: Presents material quantity estimates for the A21 dike construction
- Section 10: Provides the design for instrumentation and monitoring of the dike
- Section 11: Describes in general terms the required monitoring of A21 dike-pit interaction
- Section 12: Presents the earthworks and hydrologic aspects for the runoff and seepage collection system to be constructed between the A21 dike and the A21 pit rim
- Section 13: Presents an overview of the construction quality assurance and quality control (QA/QC) plan for the dike's construction. The detailed plan is provided in Appendix N
- Section 14: Presents recommendations for additional bathymetric and geotechnical investigations to support the construction of the dike.
- Section 15: Outlines the concept for dike closure and reclamation.

The report includes 73 drawings, completed to an issued-for-tender (IFT) status.

This report has 16 appendices, which provide:

- Reports documenting the geotechnical, hydrogeologic, and geophysical investigations for the A21 dike (Appendices A and B).
- Analyses supporting the A21 dike design (Appendices C through L). Supporting design analyses for the A21 dike design were undertaken initially by AMEC (2007). Despite the 2012 dike alignment shift (see Figure 1-4), and the design modifications undertaken subsequent to that report, for the most part those design analyses remain representative and were not repeated by BGC. For completeness, rather than referencing the specific appendices in AMEC (2007) presenting those analyses, they have been included as appendices of this updated design report. Certain aspects of the supporting design analyses have been updated and are documented herein.
- Technical Specifications for construction of the A21 dike (Appendix M).
- Quality Control and Quality Assurance (QA/QC) plan for A21 dike construction (Appendix N).
- Material Quantity Estimates for dike construction (Appendix O).
- An example Trigger Action Response Plan (TARP) for monitoring of the A21 dike.

2.0 SITE CONDITIONS

2.1. Site Description

The Diavik site is located approximately 300 km northeast of Yellowknife. As was the case for the A154 and A418 kimberlite pipes, the A21 pipe is located under Lac de Gras. The northern extent of the A21 pipe is approximately 400 m due south of East Island (where all site facilities are located), as shown on Drawing 14300-41D2-1002. There is a small island, approximately 0.7 ha in area, located about 65 m north of the A21 pipe, and South Island, approximately 800 ha, located approximately 400 m to the west of the A21 pipe. East Island and the South Island are separated by a channel that ranges between 50 m to 285 m in width. A rockfill causeway bridges that channel.

2.2. Climate

Diavik is located within the Arctic Climate Region, where daylight reaches a minimum of four hours per day in winter, and a maximum of 20 hours per day during summer. Climate at the site comprises long, cold winters and very short, cool summers. Average monthly temperatures measured on site, between 1998 and 2013, range from +15°C in July 2001 to -32°C in February 2003. The mean annual air temperature (MAAT) at the site measured between 1998 and 2013 is -8.4°C. Estimated minimum, maximum and average monthly temperatures based on data recorded on site are given in Table 2-1. As noted in the table, several data gaps exist in the data collected on site. Nearby Environment Canada weather stations at Ekati and Lupin, which are located approximately 25 km and 140 km north of Diavik, respectively, were examined but they also have incomplete data sets. Nevertheless, these stations confirm the observations from the Diavik site that July 2000 was the warmest month recorded and February 2003 was one of the coldest. The more complete data sets from Ekati and Diavik indicated January 2004 to be about 1.5°C colder than February 2003. Colder months had only been recorded before 1994, i.e. pre Diavik records. Similarly, the maximum values recorded at Ekati and Lupin were also in the month of July 2000. This comparison suggests that, however incomplete, the weather data recorded at the Diavik site are representative and provide an acceptable basis for thermal analyses, summarized in Section 8.6.

Table 2-1. Mean monthly air temperatures at Diavik. The minimum and maximum in the record are highlighted in bold.

Month	Mean Monthly Air Temperature (°C)					Years missing
	Minimum	Maximum	Average			
			All	10 yr	5 yr	
January	-31.1 (2013)	-23.5 (2001)	-27.2	-27.8	-28.0	1998, 1999, 2004, 2006
February	**-31.9** (2003)	-21.1 (2010)	-26.6	-25.9	-24.3	1999, 2005, 2006
March	-28.0 (2008)	-15.9 (2010)	-23.4	-24.4	-22.7	1998, 2005, 2006
April	-18.6 (2002)	-6.5 (2010)	-14.1	-14.0	-13.5	1998, 2005, 2006
May	-10.4 (2004)	1.5 (2012)	-4.0	-4.3	-3.6	1998, 2000, 2002, 2005, 2006
June	5.5 (2009)	11.8 (2013)	7.4	7.6	8.5	1998, 2002, 2005, 2006, 2011
July	9.7 (1999)	**15.3** (2000)	12.9	12.5	12.0	1998, 2002, 2004 - 2006, 2008, 2011, 2012
August	8.2 (2007)	12.8 (1998)	10.6	10.6	11.4	2004 - 2006, 2008, 2011, 2012
September	2.6 (2000)	9.6 (2002)	5.2	5.7	5.4	2005, 2008, 2011, 2012
October	-8.7 (2004)	-2.2 (1998)	-5.2	-5.2	-4.3	2000, 2005, 2008, 2011, 2012
November	-20.3 (2006)	-15.2 (2010)	-17.9	-18.5	-17.4	1998, 2005, 2008, 2011
December	-29.7 (2004)	-18.0 (2006)	-24.6	-25.1	-25.5	1998, 1999, 2005, 2008
Annual	-9.7 (2009)	-6.8 (2010)	-8.4	-9.0	-8.8	1998 - 2000, 2002, 2004-2006, 2011, 2012

Notes: Data provided by DDMI, March 2014
10 year averages were determined from 2004 - 2013 and 5 year averages from 2009 - 2013, respectively.
Several years had incomplete data sets.

No significant trends were noted on site during the last ten years in annual or monthly air temperatures (Appendix L, and Figure 2-1). Because of this observation, together with the short design life of the A21 dike of seven years, average climate conditions are judged suitable as input parameters for the thermal design (Section 8.6) of the dike. Extreme values, such as exceptionally warm or cold years, were not considered in thermal design aspects. First, the data set is too short and incomplete to allow for an accurate statistical analysis of the extremes and secondly, the probability of such an event occurring during the design life of the A21 dike is very low. Most importantly, however, the consequences associated with an extreme event are minimal, eliminating the need to evaluate extreme scenarios, and supporting analyses and design is based on average conditions.

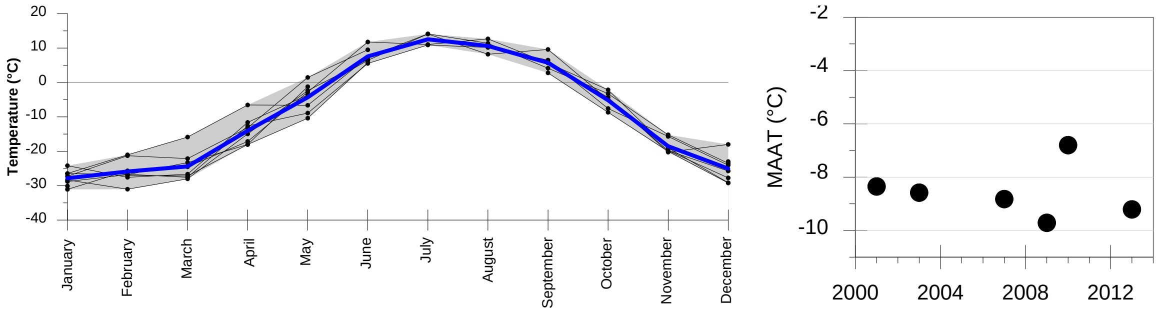


Figure 2-1. Range in monthly air temperatures between 2004 and 2013 (left) and mean annual air temperatures available since 1998 (right) recorded on site.

Winds are moderate to strong and show a slight north-west/south-east tendency, with average wind speeds of 16 to 22 km/h. No significant seasonality was noted during the 1998 to 2013 records from the site (Figure 2-2).

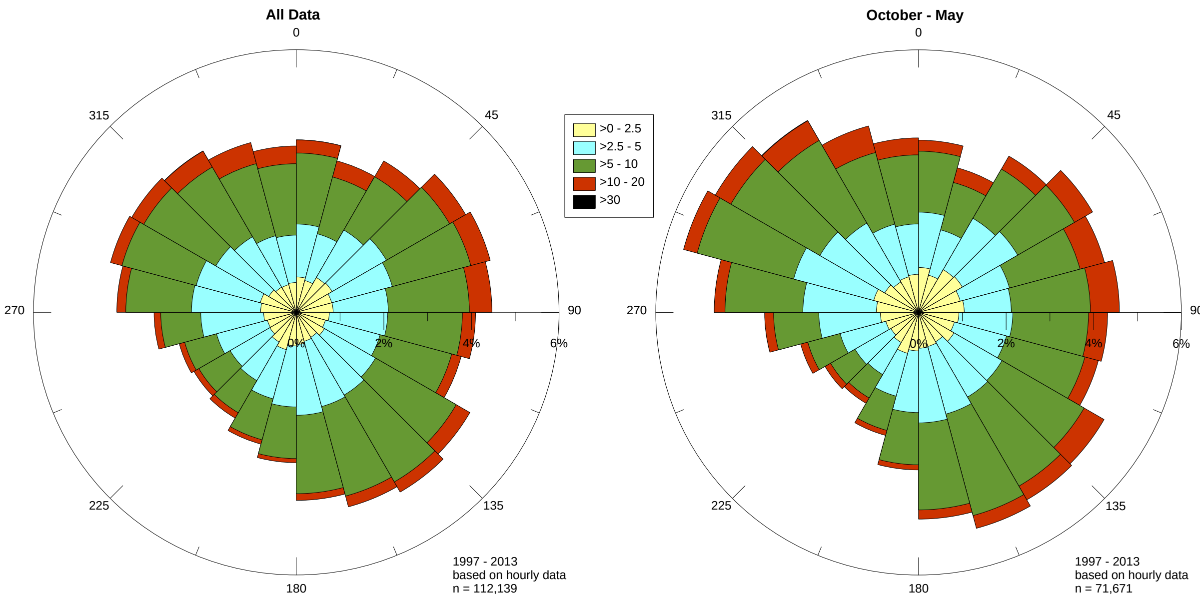


Figure 2-2. Rose diagrams for the full year (left) and winter months only (right) based on data reported by DDMI between 1997 and 2013.

Site date on precipitation and snow depth are incomplete and therefore, information presented in previous reports and from Ekati was used. Average annual precipitation for the site is estimated to be 373 mm, approximately 40% of which (144 mm) occurs as rainfall, and the remainder as snowfall (229 mm snow water equivalent). Snow can fall every month of the year and rain generally falls only between May and October. Net annual average runoff (precipitation minus evapotranspiration and sublimation) is 171 mm. Snow depth records by Environment Canada from nearby Ekati show, from 1999 through 2008, that the maximum monthly snow depth (not cumulative snow fall) occurs in March with an average of 34.4 cm. Between November and April, the average natural snow depth varies between 11 cm and 37 cm, and the absolute maximum snow depth was 53 cm, recorded in April 2001.

Solar radiation measured by DDMI is highest during June with values of about 250 W/m^2 and lowest in December with only 2 W/m^2.

Summer is the driest season with an average relative humidity of 73% ± 7%, followed by winter (77% ± 6%) spring (82% ± 5%) and fall (87% ± 5%). These values are based on the on-site measurements collected between 1998 and 2013.

2.3. Permafrost

Diavik is located approximately 100 kilometers north of the diffuse boundary between extensive discontinuous and continuous permafrost (Heginbottom et al., 1995). Based on deep thermistor data, permafrost has been confirmed to a depth of 150 m below East Island (~Elev. 270 m), although temperature data suggests the permafrost could extend to a depth of 240 m (~Elev. 180 m) or even deeper inland. The active layer is typically 1.5 to 2 m in till deposits, 2 to 3 m deep in glaciofluvial deposits (eskers) and may be more than 5 m deep in exposed bedrock. Thermistor data indicate a range of mean annual ground temperatures from approximately -3°C to -6°C.

The depth of permafrost decreases, and ground temperatures increase, towards Lac de Gras, with a talik below the lake itself. Thermistor data show that permafrost exists where water depths are less than about 2 m, i.e., the depth to which the lake freezes to the bottom during the winter months. The shallow lake conditions on the south abutment of the A21 dike indicate permafrost extends approximately 450 m into the lake, following the proposed dike alignment from the shoreline at about El. 415.8 m.

Details on the ground temperature data from thermistors installed at the abutments of the Dike A21 alignment are discussed in Appendix L.

2.4. Site Physiography and Regional Geology

The Lac de Gras area is in the central part of the Slave geological province of the Canadian Shield, the largest physiographic region of Canada. The Canadian Shield hosts the largest area of Archean rocks (more than 2.5 billion years old) in the world. The rock units at the Diavik site are primarily Archean, Proterozoic, or Eocene in age.

The northern portion of the Diavik site is underlain by granite and pegmatite country rock intruded into meta-sedimentary rock originally deposited as sandstone and shale. The meta-sedimentary rock generally represents only a small proportion of the country rock, occurring as meta-sedimentary rafts within the intrusives. Diabase dikes have been encountered throughout the site. Kimberlite pipes occur as volcanic cores injected into the much older country rock. The kimberlite pipes are generally aligned along a northeast trending axis that represents a zone of structural weakness exploited by the intrusions. Golder (2012) identified the northeast structural trend during geotechnical drilling in support of the A21 open pit geotechnical design. The trend is about 100 m in width, with boundaries characterized by narrow, discrete faults with high angle dips to the northwest.

Moving south in the mine area, the geology transitions from being mainly pink granite with meta-sedimentary lenses, in the area of the A154/A418 pits, to metaturbidites, to tonalite to quartz diorite at A21. Around the A21 kimberlite pipe, and below the A21 dike alignment, the country rock comprises tonalite to quartz diorite. Stubley (1998) describes the tonalite as medium grained with biotite and hornblende to 35%. Pegmatite dikes intrude the tonalite. The tonalite is intruded by the A21 kimberlite pipe. Metasediment units common to the A154 and A418 mine areas are absent at A21. The A21 kimberlite pipe comprises weak volcaniclastic kimberlite and kimberlitic mudstone with occasional hypabyssal kimberlite intrusions, particularly at the pipe margins.

Lac De Gras lies in an area that was subjected to multiple glacial events with divergent ice-flow events (Dredge et al. 1994, Ward et al. 1995, 1997, Kerr et al. 1998, and Rampton, 2000). Surficial geology in the project area comprises veneers of organic material, till and/or weathered parent material overlying undulating to hummocky bedrock. Observed periglacial features are typical of continuous permafrost areas. Physical weathering (frost wedging and frost shattering) typically occurs on exposed bedrock and in boulder fields. The principal soil unit in the project area is till (ablation in origin), which was deposited during glacial retreat. This unit is typically heterogeneous, unsorted, and unstratified, having been laid down by glacier ice, and contains particles ranging from clay-size (< 2 µm) to boulder-size.

Within Lac de Gras in the area of the Diavik site, the lakebed till unit is overlain by relatively recent soft deposits, which generally cover most of the Lac de Gras lakebed.

During the late stages of deglaciation it has been suggested (Dredge et al., 1994) that subglacial meltwater eroded the till resulting in exposed bedrock, boulder fields and ubiquitous evidence of landforms resulting from meltwater flow. Unweathered till in the region is described as a stoney diamicton with silty sandy matrix (Dredge et al. 1994, Ward et al. 1995); locally its upper part may be less compact and devoid of silt.

The land around the site consists of low, rolling hills with numerous lakes. Topography in the immediate vicinity of the A21 kimberlite pipe comprises low relief terrain, with a range in elevation of approximately 15 m to 20 m. Elevations range from about 415 m to 416 m along the shore of Lac de Gras (the level of which varies seasonally) to up to 430 m on land.

2.5. Lakebed Overburden

The A21 area lakebed overburden stratigraphy has been interpreted from numerous site investigation programs at the A21 site and at the sites of the A154 and A418 dikes. The typical stratigraphic sequence consists of soft, primarily silty lakebed sediments overlying ablation till, in turn overlying bedrock. For a detailed description of the geotechnical conditions of the A21 area the reader is referred to Golder (2006a, 2007). The geotechnical test holes undertaken in 2006 and 2007 that form the basis of these two reports, including sonic boreholes, diamond drill holes, piezocone soundings, thermistor installations, and Ground Penetrating Radar (GPR) and seismic refraction lines, are illustrated on Drawings 14300-41D2-1010.1 through 14300-41D2-1010.6.

The lakebed soil units are summarized below. A detailed description and characterization of each of these units is provided in the Golder reports referenced above, and are supported by extensive work undertaken previously by Nishi-Kohn/SNC Lavalin (NKSL) in support of the designs of the A154 and A418 dikes (NKSL, 1999, 2004).

2.5.1. Lakebed Sediments

Soft lakebed sediments exist over much of the A21 dike alignment. The thickness of this layer within the areas investigated to date in Lac de Gras near East and South Islands varies between 0 and 5 m (within the bathymetric low to the southwest of the A21 pipe), with an average thickness of about 1.2 m. Per Golder (2006a), the thickness of the lakebed sediments varied from 0.0 m to 4.6 m, with an average thickness of 1.2 m for 111 diamond drill borehole locations. In comparison, the average thickness of sediments was 1.7 m from 19 Sonic Drill boreholes, and 1.2 m from 92 CPT soundings. Figure 2-3 presents a histogram of lakebed sediment thicknesses as derived from piezocone soundings undertaken in the area of the existing dikes (A154 and A418), the proposed A21 dike, and the North Inlet Pond.

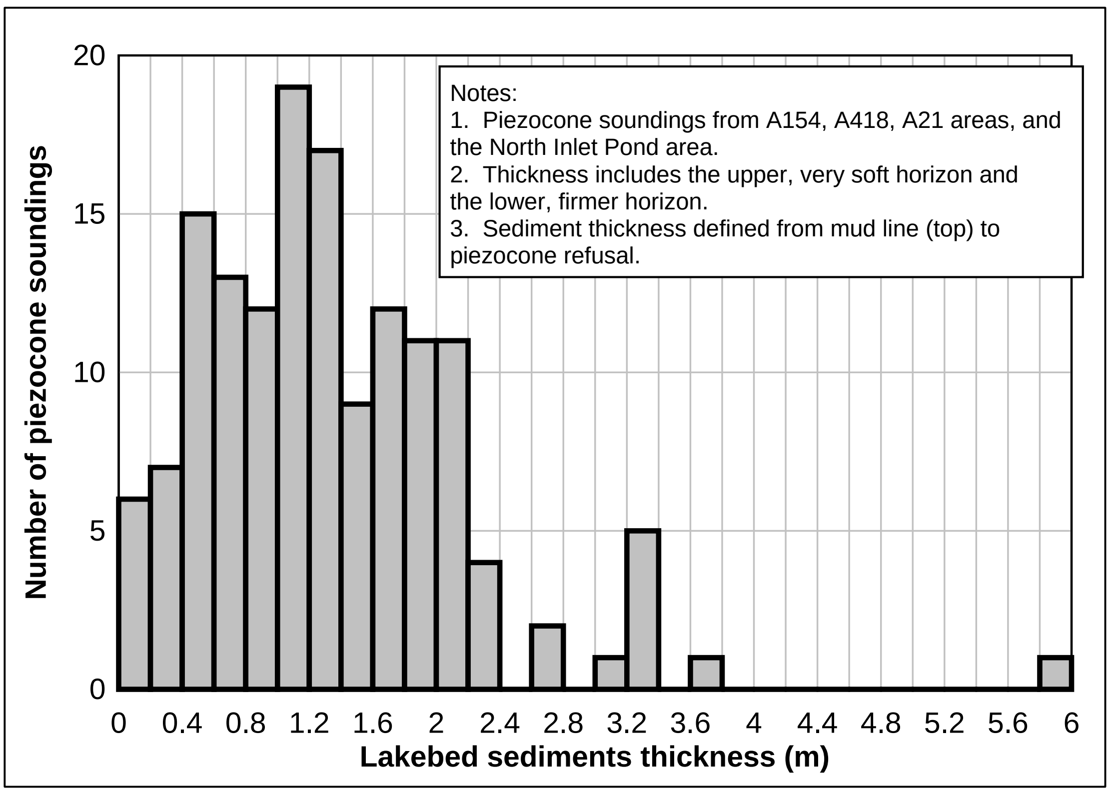


Figure 2-3. Histogram of lakebed sediment thicknesses derived from piezocone soundings in the area of the A154, A418, and A21 dike alignments, and from the North Inlet Pond.

Golder (2006a) describes a piezocone investigation of the lakebed sediments along the original (1999) deep water alignment envisioned for the A21 dike. Figure 2-4 provides a histogram of the lakebed sediment thicknesses (from mud-line to piezocone refusal, which was interpreted to be at the top surface of the till) derived from the 2006 piezocone soundings (see Drawing 14300-41D2-1010.4).

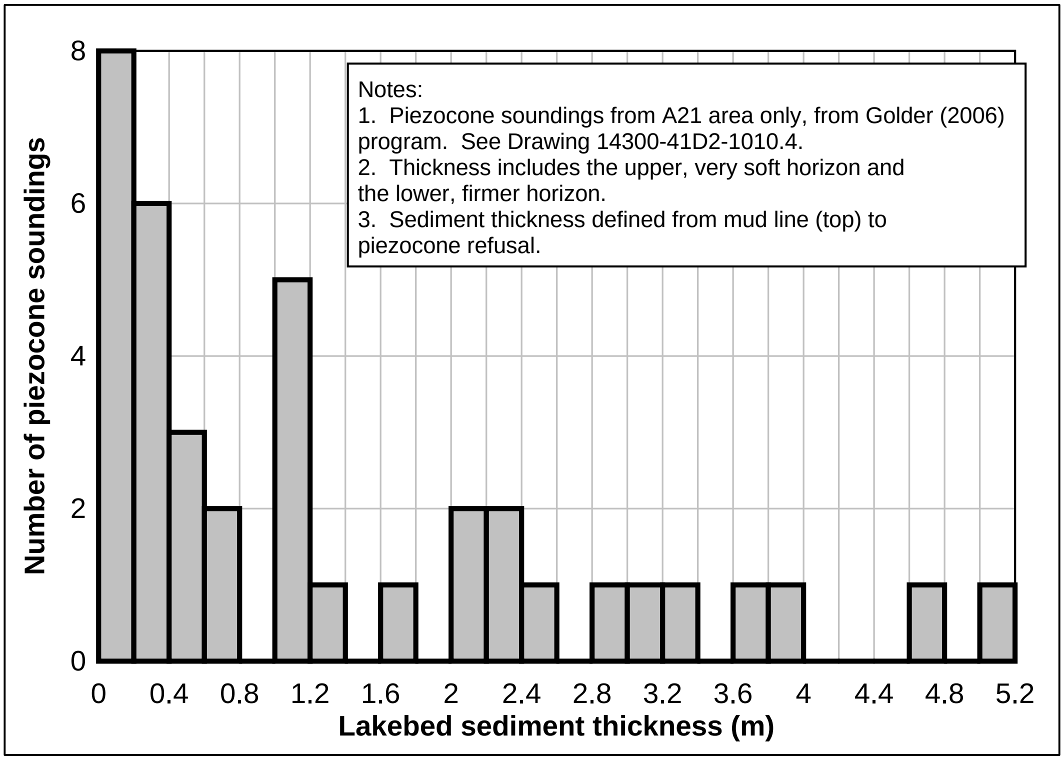


Figure 2-4. Histogram of lakebed sediment thicknesses derived from piezocone soundings in the area of the A21 dike alignment.

The lakebed sediments are judged to have been largely derived from the erosion of fine particles present in the till around the lake and in shallow waters. Previous investigations of the lakebed sediments for the A154 and A418 dikes included piezocone soundings, sonic drilling and sampling, vane shear testing, and laboratory geotechnical testing.

The lakebed sediments have been classified as highly compressible, soft, non-plastic to low-plasticity silty clays and clayey silts, with the following characteristics:

- Hydrometer testing indicated that the clay-sized particle content is typically less than 10% with occasional values in the 20-30% range. The silt size fraction typically ranges between 44% and 71%, approximately.
- The correlation between plasticity index and clay content confirmed that sediments do not contain appreciable amounts of plastic clay materials.
- The plasticity index ranges between 0 and 30%, averaging around 11%, and the liquid limit averages 29%. These values are typical of inorganic silts and rock flour material.

- X-ray diffraction testing confirmed fine particles in sediments to be predominantly rock flour rather than true clay minerals with mica representing between 50% and 70% of the mineralogical composition of the clay size fraction, chlorite 20 to 30% and kaolinite 10 to 30%.
- The hydraulic conductivity of sediments measured on undisturbed and reconstituted samples ranges from 10^{-3} to 10^{-10} m/sec with an average of about 10^{-5} m/sec, which is indicative of low clay content
- Shear strength parameters measured in consolidated-undrained triaxial (CIU) testing on relatively undisturbed (but consolidated within the laboratory) samples and direct shear testing on reconstituted samples range from an effective friction angle (ϕ') of 32 to 38.7 degrees, with an average of 34 degrees. These strength values are generally consistent with low clay contents in the sediments.
- Organic contents in the lakebed sediments vary between 0.8% and 6.0% by dry mass, with an average of 1.7%.

The lakebed sediments are underlain by a lakebed till with a matrix similar to the sandy silt sediments. The main characteristics distinguishing the sediments from the lakebed till, described in Section 2.5.2, are:

- Till is broadly graded (with silt, sand, gravel, cobbles and boulders) versus narrowly graded sediments
- Till has lower void ratio and lower water content
- The till is compact to dense whereas the sediments are soft to stiff (excepting the upper 0.5 to 1 m which is very soft).

Subsequent to completion of dewatering of the A154 pool, the lakebed sediments were exposed for the first time for detailed inspection. Eight Shelby tube samples were taken by NKSL, with three of the samples subjected to drained direct shear testing. The results reported (NKSL, 2003) for those tests are given in Table 2-2. The tests indicated modest (up to about 20%) post-peak reduction in shear strength. They also suggested a cohesion component for the residual strength, which is more likely a cohesion intercept for a best fit line between data points rather than actual cohesion. NKSL (2003) suggested an alternative interpretation of the residual strength values, with zero cohesion, as follows:

- ϕ' = 30º to 33º at 100 kPa normal stress
- ϕ' = 27º to 31º at 200 kPa normal stress, which is consistent with the value of 30º used in the design.

In contrast to the drained shear strength tests, in situ vane shear testing in the lakebed sediments reported by EBA (1998) indicated the sediments to be sensitive, with low remolded relative to peak strengths, as shown in Figure 2-5.

Table 2-2. NKSL (2003) Direct shear strength test results for lakebed sediments.

Sample #	Peak Drained Shear Strength		Residual Drained Shear Strength	
	c' (kPa)	ϕ' (degrees)	c' (kPa)	ϕ' (degrees)
A1	9	32	9	26
B2	4	35	4	30
C1	12	30	11	29

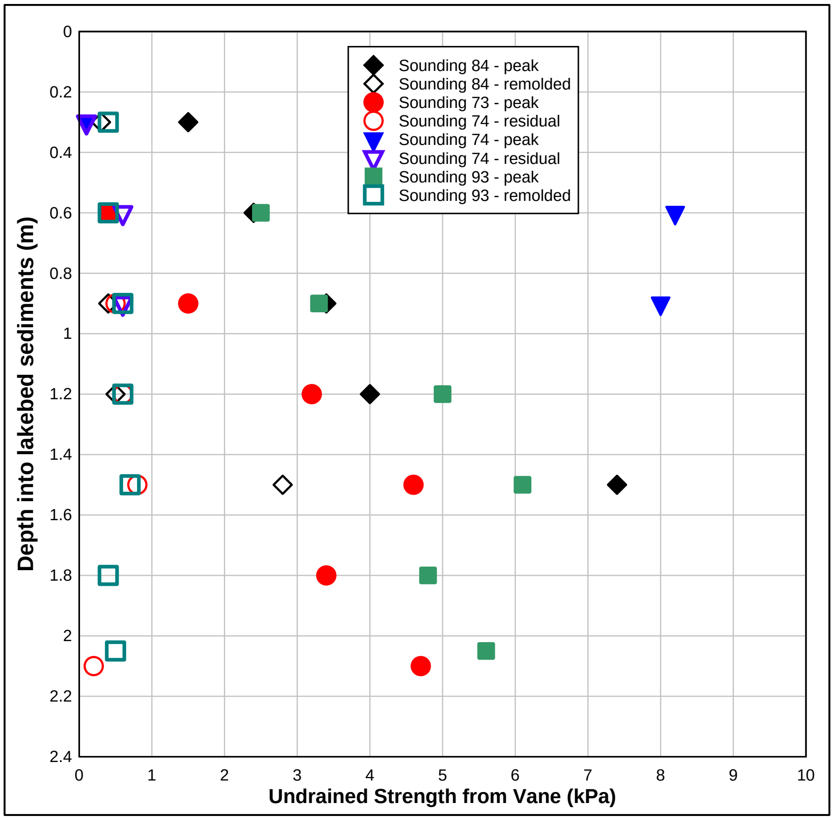


Figure 2-5. Lakebed sediment vane shear strength tests (EBA, 1998).

EBA (1997) undertook a series of isotropically-consolidated, undrained (CIU) triaxial compression strength tests on remolded lakebed till samples (obtained from sonic drill hole cores) and "undisturbed" lakebed sediment samples. The results are provided in Table 2-3.

Table 2-3. EBA (1997) CIU test results for "undisturbed" lakebed sediments samples.

Sample #	Sample Depth (m) Below the Mud Line	Peak Drained Shear Strength	
		c' (kPa)	ϕ' (degrees)
58A	0 to 0.61	1.2	33
91	0.16 to 0.31	2.4	38.7
92	0.61 to 1.22	2.5	34.4

The main objective of the Golder (2006a) piezocone program (see Drawing 14300-41D2-1010.4), and associated analysis, was to geotechnically characterize the lakebed sediments. The piezocone data, and observations of the A154 and A418 in-field areas following dewatering, indicated the following materials:

- An upper, softer portion of the lakebed sediments, generally 0.5 m to 1.5 m thick, with near zero tip resistance and positive dynamic pore pressure response, underlain by
- More firm sediments exhibiting significantly higher tip resistance coupled with negative dynamic pore pressure response (indicative of dilatant response to piezocone penetration).

Table 2-4 summarizes the geotechnical parameters derived from field and laboratory testing of the lakebed sediments.

Table 2-4. Lakebed sediments geotechnical parameters (Golder, 2006a).

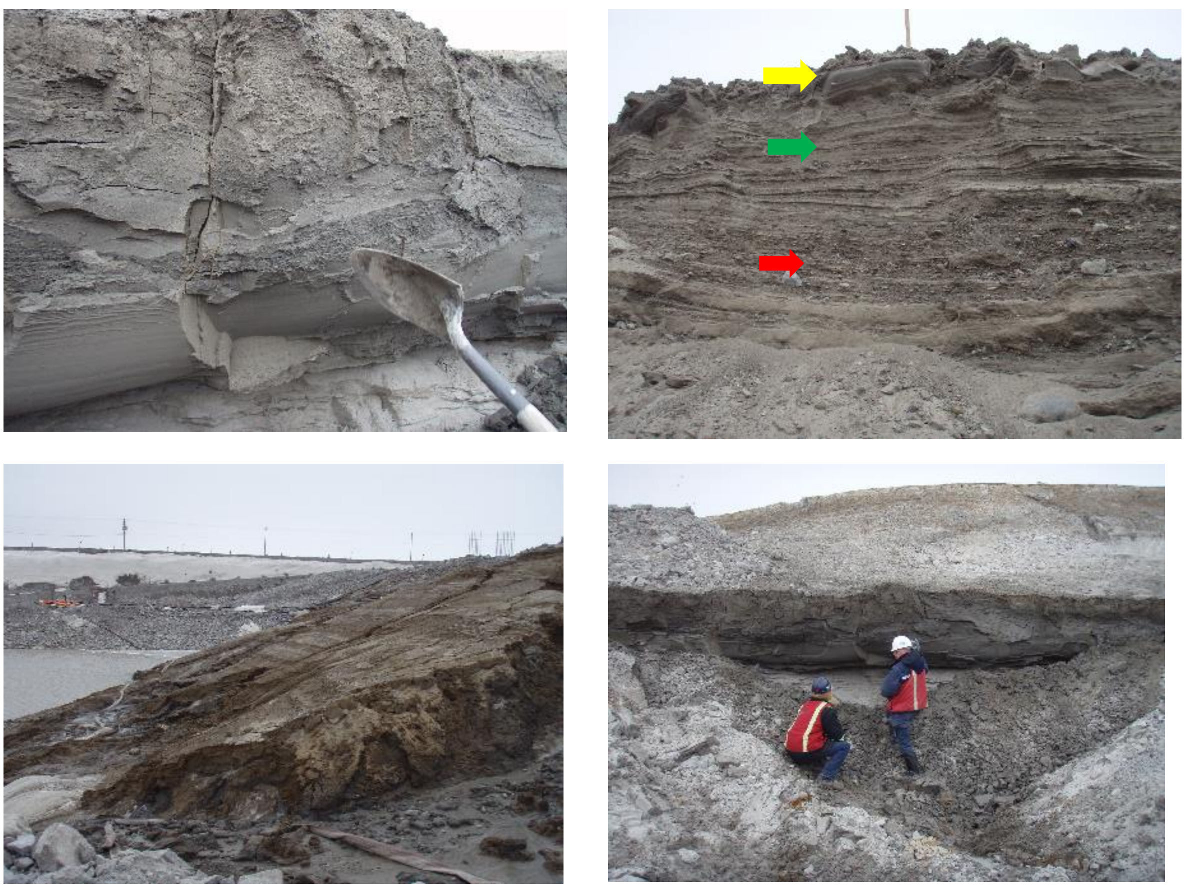
Property	Range	Suggested Design Value
In-Situ Void Ratio	0.5 - 3.2	1.7
Bulk density (T/m^3)	1.34 - 2.11	1.63
Dry Density (T/m^3)	0.31 - 1.80	1.00
Specific Gravity	2.61 - 2.73	2.71
Degree of Saturation (%)	100	100
Plasticity Index	4.5 - 28	11
Friction Angle, ϕ (degrees)		
- Drained	31.3 - 36.2	32
- Undrained	33.0 - 38.7	
Effective Cohesion c' (kPa)	3.1 - 33.6	0
Hydraulic Conductivity, k (m/sec)	8.2×10^{-11} - 2×10^{-3}	4×10^{-5}
Coefficient of Consolidation, c_v (m^2/yr)	0.3 - 447	10
Coefficient of Volume Change, m_v (m^2/MN)	0.002 - 2.75	0.06
Undrained Shear Strength (S_u) of the upper, very soft horizon		≈ 6 kPa, at 1 m below the mud line

The scatter in hydraulic conductivities indicated in Table 2-4 is mainly attributed to two factors: (i) limitations in the testing procedures; (ii) disturbance during sampling as a result of the high-frequency vibration sonic drilling technique; (iii) gradation variations in such a heterogeneous material. Further, the sediments exhibit distinct layering that gives rise to anisotropic hydraulic conductivity parameters.

In relatively shallow areas of the lake near shorelines, the soft sediments are generally absent. Instead, the shallow areas and shorelines comprise significant concentrations of boulders, termed boulder pavements or boulder lags. In these areas, due to wave action, the sediments appear to have been eroded (and subsequently deposited in deeper water areas), leaving the boulder concentrations in place. In addition, during glacial retreat there appears to have been substantial fluvial reworking of the upper portion of the till which may also explain the frequent occurrence of boulders, and the discontinuous presence of glaciofluvial sands between the lakebed sediments and the underlying lakebed till.

For both the A154 and A418 dikes construction, the design specified the removal of the lakebed sediments from below the dike footprints. This was accomplished via dredging, except in shallows where due to dredge draught limitations, excavation via hoe or clamshell had to be undertaken. An important observation of the dredging campaigns for the A154 and A418 dikes, as discussed in more detail in Section 4.2.2, is that the dredging was, at best, only partially successful in removal of the firmer, lower horizon of lakebed sediments.

Figure 2-6 shows representative photographs of the lower, firmer horizon of lakebed sediments exposed within the pit areas subsequent to A418 pool dewatering.



Top left - sand and silt exposed in a cut in the A418 pit area. Top right - silt (yellow arrow), grading to sand (green arrow), underlain in turn by sand and gravel glaciofluvial deposit (red arrow). Bottom left - exposed lakebed slope downstream of the A418 dike. Bottom right - cut into lakebed sediments.

Figure 2-6. Lakebed sediments exposed in A418 pit area.

2.5.2. Lakebed Till

Till (ablation in origin) underlies the soft sediments over the entire A21 area. This layer is 4 m to 5 m thick on average, with the maximum thickness of the lakebed till being about 27 m (within the bathymetric low to the west of the A21 kimberlite pipe). The till is characterized by significant heterogeneity as it contains particles ranging from clay size to boulders. The lakebed till has been classified as a non-plastic and stony till, and essentially consists of silty sand with some gravel, with sand lenses (associated with glacio-fluvial activity - see top right photo in Figure 2-7) and boulder to cobble-sized particles. Such boulders occur in a disseminated manner, floating in the till matrix, as shown in the photographs in Figure 2-7.

Top left - typical exposure of lakebed till - note content of cobbles and boulders.

Above - glacio-fluvial silt/sand/gravel. Such channel deposits occur frequently within the lakebed till.

Lower left - cobbles and boulders in silt/sand matrix.

Figure 2-7. Lakebed till exposed in A418 pit area.

A summary of the engineering design parameters for the till is presented in Table 2-5. This data is based on Golder's (2006a and 2007) reports, and on data from the A154 and A418 sites from the NKSL studies.

Table 2-5. Lakebed till geotechnical parameters (Golder, 2007).

Property	Range	Suggested Design Value
In-Situ Void Ratio		0.36
Bulk density (T/m^3)	1.56 - 2.30	2.24
Dry Density (T/m^3)	1.03 - 2.13	1.98
Specific Gravity	2.61 - 2.72	2.69
Degree of Saturation (%)	100	100
Plasticity Index	non-plastic	non-plastic
<u>Friction Angle, ϕ (degrees)</u>		
- Drained	33.6 - 46.2	34
- Undrained	34.1 - 41.5	
c' (kPa)	0 - 58.3	0
Hydraulic Conductivity, k (m/sec)	$7x10^{-9}$ - $6x10^{-3}$	$4x10^{-5}$
Coefficient of Consolidation, c_v (m^2/yr)	8 - 106	40
Coefficient of Volume Change, m_v (m^2/MN)	0.032 - 0.065	0.043

EBA (2004) undertook a test trenching program within the lakebed upon the completion of A154 pool dewatering. That program represented the first investigation of the lakebed till not constrained by drilling limitations. Key findings as presented by EBA (2004) were as follows:

- The till matrix comprises relatively well-graded fine to coarse sands with 10-30% fines
- The matrix soils support varying gravel, cobble and boulders, and typically show no evidence of segregation at the bedrock contact
- Fines contained minimal clay and the silt was generally coarse
- The till contained sub-angular clasts with some rounded and some angular clasts
- The tills were dark in color suggesting little oxidation; thus most of the tills were likely either frozen or in a subaqueous condition since deposition. Brownish oxidized soils were typically limited to surficial glacio-fluvial soils and near surface lakebed sands and silts
- Glacio-fluvial deposits were observed in layers just below lakebed sediments and sometimes between till units. Granular pockets and lenses were encountered with reduced frequency with depth
- Deposition anomalies such as pockets of sand or silt and perched seepage zones were encountered in the upper till, which typically included one or more thin horizons of sand and gravel modified by melt-water - sand lenses and pockets existed in the till but appeared more prevalent in the upper 2 m
- Clean glacio-fluvial deposits just below the lakebed sediments were encountered discontinuously in the text excavations on the north side of the A154 pit. On the south side glacio-fluvial deposits occurred more consistently
- Boulder content was visually estimated at 10-20% of the till by volume

- Boulders 1 m and larger in size were uncommon, whereas cobbles and small diameter boulders (up to 300 mm) were pervasive
- Where large diameter boulders were encountered, it was not uncommon to find two or more together.

The field density testing carried out by EBA (2004) on the lakebed till as part of the test trenching program is summarized in Table 2-6.

Table 2-6. EBA (2004) field density testing on lakebed till.

Sample Description	USCS *	Moisture Content (%)	Dry Density (kg/m^3)	Void Ratio
Sand and gravel (till) - fine to coarse grained sand, silty, angular and subrounded clasts, dark olive grey	GM	6.9	2176	0.24
Sand and gravel (till) - fine to coarse grained sand, some silt, some angular and subrounded gravel, dark olive grey	GM	5.3	2042	0.32
Sand (glaciofluvial) - poorly graded, some gravel, trace of silt	SP	1.1	1739	0.55
Sand and gravel (till) - fine to coarse grained sand, silty, some angular and subrounded gravel, dark olive grey	GM	7.9	2335	0.16
Sand (till) - fine to coarse grained sand, silty, gravelly, angular and subrounded clasts, dark olive grey	SM	5.3	2163	0.25
Sand and gravel (till) - fine to coarse grained sand, some silt, some angular and subrounded gravel, dark olive grey	SM	7.2	2175	0.24
Sand (till) - fine to coarse grained, gravelly, silty, rounded and angular-subrounded clasts, dark olive grey	SM	6.5	2207	0.22
Sand (till) - fine to coarse grained, gravelly, silty, rounded and angular-subrounded clasts, dark olive grey	SM	7.4	2186	0.24

** USCS = Unified Soil Classification System*

The lakebed till unit represents a competent foundation for the water retention dikes. In general, the till is of relatively low hydraulic conductivity. However, due to its heterogeneous nature, the frequent presence of glaciofluvial interbeds (see for example, top right photo) and the potential for concentrated boulders at the interface with bedrock, it must be considered in general as a hydraulically conductive unit requiring seepage control measures as part of the dike design. As such, the seepage cut-off element for both the A154 and A418 dikes was required to fully penetrate this unit. This same requirement applies to the proposed A21 dike.

2.6. Bedrock

Bedrock in the A21 area has been investigated for the purposes of the A21 dike design, open pit geotechnical design, and to support hydrogeologic modeling. The results of these investigations are reported by Golder (2006a, 2007).

Average core recovery from the winter 2006 diamond drill program was 98% (Golder, 2006a). The average value of the Rock Quality Designation (RQD) was 86%. In general, the rock mass was found to be of good quality.

Detailed descriptions of the bedrock, structure and major features in the A21 area are provided in Golder (2006a, 2007). Design values adopted for the bedrock at the A21 site are presented in Table 2-7.

The bedrock is generally very competent and poses no concerns in terms of the stability of the dikes. It is, however, fractured, particularly at relative shallow depths owing to the effects of post-glacial rebound and stress relief, imparting a fracture-controlled secondary hydraulic conductivity that in places is significant, and if not remediated would represent a potential concern in terms of dike seepage. As such, the seepage cut-offs for both the A154 and A418 dikes were required to key into bedrock via a single row grout curtain. This same requirement applies for the proposed A21 dike. Work to date at the A21 site indicates dominant joint sets that are both sub-horizontal and sub-vertical, mandating angled holes for any curtain.

Experience at the site suggests that the faults and contacts are likely to be continuous, while systematic joint structures are more limited in continuity (Golder, 2006b and 2012). It is believed that the continuity of the systematic joint structures may be in the order of approximately 30 m.

Table 2-7. Engineering design parameters adopted for bedrock at A21 (Golder, 2006).

Property	Range	Suggested Design Value
	Unconfined Compressive Strength (MPa)	
<u>Pit</u>		
Granite (Tonalite)	116 - 144	130
Pegmatite	79.9 - 101	90
<u>Dike</u>		
Granite (Tonalite)	144 - 206	130
Pegmatite	208.3	90
	Elastic Modulus (GPa)	
<u>Pit</u>		
Granite (Tonalite)	64 - 77	57
Pegmatite	32 - 42	36
<u>Dike</u>		
Granite (Tonalite)	81 - 85	57
Pegmatite	76	36
	Joint Shear Strength	
Peak Friction Angle (degrees)	47 -61	47
Residual friction Angle (degrees)	29 - 52	29
	Hydraulic Conductivity, k (m/sec)	
<u>Bedrock</u>		
Weathered Rock (0 - 15 m)		4×10^{-7}
Competent Bedrock (25 m - 300 m)		2×10^{-7}
Competent Bedrock (300 m - 500 m)		5×10^{-8}
Competent Bedrock (500 m - 1,500 m)		1×10^{-8}
	Hydraulic Conductivity, k (m/sec)	
<u>Kimberlite Pipe</u>		
25 m - 500 m (1)		6×10^{-8}
500 m - 1,500 m (1)		5×10^{-9}
<u>Deep Blue</u>		
25 m - 400 m		1×10^{-5}
400 m - 1,500 m		1×10^{-6}

1. Data based solely on A154 and A418 areas.

2.7. Deep Blue Bathymetric Low

A bathymetric low designated “deep blue” exists approximately 200 m southwest of the A21 pipe (see Figure 1-5). The origins of the Deep Blue bathymetric low are uncertain. At one point DDMI geologists hypothesized the presence of an additional kimberlite pipe associated with this feature. Kimberlite was not encountered, however, and the feature may represent a “failed” kimberlite pipe. There is a small, uneconomic, kimberlite pipe in a smaller bathymetric low to the southeast of the Deep Blue feature (and north of the A21 dike alignment); this is designated as the A5 pipe (see Figure 1-5). This bathymetric low was transected by seismic refraction survey line SL-1 (see Drawing 14300-41D2-1010.1).

Investigations completed to date indicate the Deep Blue feature comprises a considerable thickness of till and sediment sequences overlying very poor quality bedrock. These areas of greater fracturing have greater hydraulic conductivity than the surrounding less disturbed bedrock. The geotechnical characteristics within this bathymetric low, which represents a significant consideration in terms of the design of the proposed A21 open pit that will encompass it, are provided in the Golder (2006a) report (Appendix A), and in Golder (2012).

The Deep Blue feature affects the mining conditions in the open pit, but has no effect on the A21 dike. A geotechnical hole was drilled from the A21 decline (hole A21-U11, see Drawing 41D2-1010.1), to the south to evaluate the potential presence of deep hydraulically conductive structure, transecting the dike alignment, connecting Deep Blue and the A5 pipe to Lac de Gras. No such feature was detected.

3.0 DESIGN CRITERIA AND PARAMETERS

3.1. CDA Consequence Classification

The A154 and A418 dikes were assigned a "very high" consequence classification according to the Canadian Dam Association (CDA) guidelines (CDA, 1999) in place at the time of their design and construction, based on the potential for loss of life in the event of dike breach with workers in the A21 pit. CDA has subsequently updated the dam safety guidelines (CDA, 2007). Although there is no permanent population at risk, with mine workers in the A21 open pit and infield area, the A21 dike is assigned a "very high" classification under the updated CDA (2007) guidelines for the purpose of selecting earthquake and flood design criteria.

3.2. Earthquake Design Criteria

Given the "very high" consequence classification, the maximum design earthquake is the 10,000-year return period ground motion.

Diavik is not located within a seismic source zone. There are no identifiable active faults, and the site is far from other more active seismic zones. The peak horizontal ground acceleration (PGA), for a 10,000-year return period, is 0.023g (NKSL, 1999, 2004). The 2010 National Build Code of Canada seismic model[1] indicates a PGA, for a 1 in 2,475 year return period, of 0.036 g. This still represents a very low level of seismic hazard, and as such seismicity does not affect the dike design.

3.3. Hydrologic Design Criteria and Parameters

Hydrologic design criteria and parameters, which govern the required elevations of the cut-off wall and the dike crest, are as listed below.

- Drainage basin Lac de Gras: 3,980 km^2
- Lake area: 580 km^2
- Average lake depth: 15 m
- Typical lake level fluctuation: El. 415.5 m to 416.0 m
- Maximum Normal Water Level (MNWL): 415.85 m, corresponding to a return period of 2 years
- Maximum Water Level for 10,000-year return period: 416.70 m
- Wind speeds and set-up in Lac de Gras:
  - 10-year return period: 89 km/hr wind speed, set up = 0.26 m
  - 100-year return period: 95 km/hr wind speed, set up = 0.33 m
  - 1,000-year return period: 114 km/hr wind speed, set up = 0.42 m
- Maximum Design Water Level (MDWL): $MWL_{(10,000)}$ = 417.0 m

[1] See http://www.earthquakescanada.nrcan.gc.ca/hazard-alea/interpolat/index-eng.php.

- Maximum Instantaneous Water Level (MIWL): MDWL + wind set up and wave run up for return period of 100 years (417 + 2.3 m wave + 1.2 m run up = 420.5 m)

Based on these criteria and parameters, the top of the cut-off wall, and the dike crest elevation, for the Diavik dikes are as follows:

Top of cut-off wall: El. 417.5 m (MDWL + 0.5 m)

Dike crest: El. 421 m (MIWL + 0.5 m)

3.4. Dike Stability Design Criteria

Design criteria for limit equilibrium factor of safety against slope failure of the dike are as specified by CDA (2007), and listed in Table 3-1.

Table 3-1. Minimum required factors of safety

<table>
<tr><th>Loading Case</th><th>Type of loading</th><th>Minimum required Factor of Safety</th></tr>
<tr><td>Embankment construction</td><td rowspan="2">Short-term static loading (e.g. end of construction), with excess pore pressures induced by construction or during dewatering (i.e. rapid drawdown)</td><td rowspan="2">1.3</td></tr>
<tr><td>Dewatering</td></tr>
<tr><td>Long term, static conditions</td><td>Long term, steady state conditions at normal reservoir level</td><td>1.5</td></tr>
<tr><td>Earthquake Loading</td><td>Seismic loading (pseudo-static analysis)</td><td>1.0</td></tr>
</table>

Considering dike-pit interaction, any potential slip surfaces involving the A21 pit walls that could affect the A21 dike are required to have a minimum factor of safety of 1.5.

3.5. Filters Design Criteria

Criteria for filter design are as outlined in CDA (2007) and ICOLD (2013). The filters design for the A154 dike (applicable also for A418 and A21) is described by Rattue et al. (2004).

3.6. Cut-off Wall

The dike cut-off wall comprises the seepage reduction element within the dike, and extends from El. 417.5 m to 15 m to 25 m below the bedrock surface (where permafrost is absent). There are three phases of the cut-off wall as outlined below.

3.6.1. Plastic Concrete Cut-off Wall

The plastic concrete cut-off wall, to be developed via the CSM method, extends from El. 417.5 m to a nominal 3 m into lakebed till, except in abutment areas where its base limit is either El. 412 m or bedrock if encountered above El. 412 m. The CSM cut-off wall may be extended deeper in areas where the lakebed soils comprise glaciofluvial material (as opposed to lakebed till) - this can only be evaluated during pre-drilling in advance of CSM construction.

Design criteria for the plastic concrete cut-off are as follows:

- Low hydraulic conductivity - to function effectively as the seepage barrier within the dike.
- Resistance to erosion in the event of seepage though a through-going crack in the cut-off. The plastic concrete must not be susceptible to a progressive internal erosion type failure.
- Structural performance aspects to avoid cracking and potential hydraulic fracture:
    - Sufficient resistance against compressive failure considering maximum extreme fibre stresses
    - No tensile stresses developed within the cut-off considering minimum extreme fibre stresses.
    - Compressive stresses within the cut-off greater than hydrostatic pressure, to avoid potential hydraulic fracture.

The acceptance criteria, constituting the specified parameters to achieve the design criteria listed above, are as presented below.

3.6.1.1. Width

The specified width of the cut-off wall is 0.8 m.

3.6.1.2. Hydraulic Conductivity

The specified maximum hydraulic conductivity for the CSM component of the cut-off wall is 10^{-8} m/sec, as tested by ASTM D5084 (28-days after casting the test cylinder at 23°C) with a hydraulic gradient of 10. This remains unchanged from the A154 and A418 dike plastic concrete requirement.

3.6.1.3. Erosion Resistance

Although the design of the CSM cut-off wall is intended to prevent, or at least minimize cracking, the design must provide for the integrity of the cut-off wall, and the safety of the dike, if cracking does occur. This requires that the CSM cut-off wall be protected against internal erosion due to concentrated seepage through any potential cracks. Such protection is provided via the following means:

- 1st line of defense - erosion resistance of the plastic concrete CSM cut-off wall itself
- 2nd line of defense - filter capability of the vibro-densified Zone 1 fill against the downstream face of the cut-off wall.

As noted by Zhou et al. (2004) in specific reference to the A154 dike cut-off wall, erosion resistance can usually be obtained if the unconfined compressive strength is at least 35 kPa and the hydraulic gradient is 30 or less (ICOLD, 1985). Beier and Strobl (1985) determined that the allowable hydraulic gradient for double phase systems (e.g. cut-off wall formed by CSM) is about 150.

The erosion resistance of the CSM cut-off wall is to be tested and confirmed as non-dispersive by laboratory testing adapted from the pinhole dispersion test for soils (ASTM D4647).

The erosion resistance specifications are unchanged from those established for the A154 and A418 dikes.

3.6.1.4. Structural Performance Aspects

The CSM cut-off wall will be subject to axial and bending stresses, particularly within the dike core zone in response to deformations and rotation in the downstream direction induced by pool dewatering. Acceptance criteria[2] established for the structural design aspects of the plastic concrete cut-offs constructed for the A154 and A418 dikes were based on:

- Stress-deformation analyses to predict stresses and strains imposed upon the cut-off
- Laboratory mix trials, incorporating unconfined compressive strength (UCS) and triaxial unconsolidated undrained compressive (UU) strength testing to determine the stress-strain properties of various plastic concrete mixes at varying water-cement ratios, bentonite contents, and curing times
- A quality assurance/quality control (QA/QC) program tailored to the production of the plastic concrete in a controlled, batch plant process

The CSM process, which yields a mixed-in-place rather than batched product, is inherently more variable than the excavate-and-replace with tremied plastic concrete method of construction used for the A154 and A418 dikes. This increased variability was apparent by comparing UCS data from various CSM case histories against the lesser variability within the A154 and A418 dikes' QA/QC data. A certain degree of variability can be accommodated while still achieving the requisite design criteria. This was evaluated on the basis of stress-deformation analyses (see Appendices D and I), calibrated to the monitored cut-off wall deflections within the A154 and A418, and CSM lab mix trials (Tetra Tech EBA, 2014). The approach taken was to develop a relationship between the unconsolidated undrained compressive (UU) strength and initial tangent modulus (also referred to as Young's modulus, E). The development of this approach to date, and the path forward to finalize the acceptance criteria prior to CSM construction, is as outlined in Appendix D.

[2] The stiffness limitation was coupled with a range of triaxial strength, and strain to failure, as determined in an unconsolidated, undrained triaxial compression (UU) test, with a confining pressure of 150 kPa. The acceptance criteria specified for the A154 and A418 plastic concrete cut-off walls were as follows:

- Samples cured, with zero confining pressure, at 2-5°C, for 28 days
- UU strength to be in the range of 1.2 to 1.8 MPa
- Maximum initial tangent modulus (E) of 150 MPa
- Minimum axial strain to failure (peak deviator stress) of 5%

3.6.2. Jet Grout Cut-Off

The jet grout component of the cut-off is required to provide continuity between the plastic concrete portion of the cut-off wall and the bedrock grout curtain, and to treat the upper fractured, joint filled part of the rock. The required overlaps are 1.5 m below the lakebed till to bedrock contact, and 1 m above the base of the CSM plastic concrete cut-off.

During design of the A154 dike, the double fluid system for jet grouting was proposed; however experience indicated the triple fluid system to be more appropriate for till treatment (NKSL, 2003). The triple fluid system was subsequently used for jet grouting for the A418 dike (NKSL, 2007a). Based on these precedents, the triple fluid system has been specified for the A21 dike. Jet grout hole spacing is to be 0.75 m for a triple fluid (water, air and cement) system. The diameter of columns produced in the lakebed till is expected to be 1.25 m for the triple fluid system. The minimum width of the overlapping jet grout columns at their intersections is to be at least 0.8 m.

Jet grout columns are not expected to be produced in the rock, but the closeness of the jet grout holes provides significant filling action of the joints. Jet grouting may also prove useful in treating zones of open fractures and high grout take, such as was encountered on the south abutment of the A418 dike (NKSL, 2007a).

The jet grout mixes are designed primarily for erosion resistance - ductility is less of a consideration given that cut-off wall cracking within the lakebed till is much less consequential than wall cracking within the Zone 1 core fill.

The specified parameters to achieve the objectives of the jet grout portion of the cut-off are as presented below.

3.6.2.1. Hydraulic Conductivity

The specified maximum hydraulic conductivity for the jet-grout component of the cut-off wall (per samples from the soil-crete) is 10^{-8} m/sec, as tested by ASTM D5084 (28-days after the jet grout column is formed) with a hydraulic gradient of 30. This remains unchanged from the A154 and A418 dike requirement.

3.6.2.2. Unconfined Compressive Strength

UCS requirements for the jet grout samples are as follows:

- UCS at 7 days: minimum 1 MPa
- UCS at 28 days: minimum 2 MPa, maximum 6 MPa

These UCS specifications are based on QA/QC results from the A418 Dike, as documented by NKSL (2007a).

3.6.2.3. Erosion Resistance

The jet grout is to be non-dispersive when tested using the cracked erosion test (ASTM D4647) with a hydraulic gradient of 10, unchanged from the A154/A418 requirement.

3.6.3. Bedrock Grout Curtain

The bedrock grout curtain is to be a single row curtain, extending:

- 0.75H into bedrock, where H = the difference between MNWL (El. 415.8 m) and the lakebed foundation (at the cut-off wall) following dredging
- 15 m minimum, with a minimum of 25 m in selected areas as shown on the drawings.

These criteria are unchanged from the A418 dike (NKSL, 2004) with the exception of areas having the grout curtain extend to 25 m.

For most of the A21 dike alignment, the 15 m minimum depth below bedrock governs. However, as discussed in Section 5.2, the grout curtain is to extend to 25 m depth below the bedrock surface on the northern and southeastern areas of the dike.

3.7. Dike Embankment Design Parameters

The A21 dike will be constructed to El. 421 m with a crest width of 25 m. The external downstream slope will be 1.7H:1V and the upstream slope will be 1.6H:1V. The internal core of the dike will be constructed to El. 418 m and have nominal upstream and downstream slopes of 0.75H:1V. Dike geometry criteria are unchanged from those established for the A154 and A418 dikes.

Vibro-densification of the central portion of the core fill will be carried out from the El. 418 m working platform over a 10 m wide zone and will be carried out where the lakebed elevation is less than 414.3 m. The vibro-densification spacing pattern will be 2.5 m (see Drawing 14300-41D2-1019).

The filter blanket is to be 1 m in thickness and placed in areas where the lakebed elevations are less than El. 414 m. The blanket is to be extended 2 m beyond the downstream toe of the dike, to the limits indicated on Drawings 14300-41D2-1005 and -1007.

3.8. Setback from Open Pit Rim to Downstream Toe of Dike

The primary purpose of the setback (defined as the distance from the line created by the open pit limit and the bedrock surface and the projection of the dike downstream toe through the overburden to the bedrock surface) is to position the dike and its foundation outside the zone which may be excessively influenced by the excavation of the open pit (due to stress relief and pit wall dilation) and the zone which may be affected by pit wall rock instability. Excessive influence in this case is defined as straining below the dike due to pit wall dilation of sufficient magnitude to induce potentially damaging deformation of the dike. The dike element of most concern in terms of foundation straining is the cut-off wall. The specific concerns are:

- Sufficiently concentrated straining so as to induce a crack within any or all of the cut-off elements (plastic concrete cut-off, jet grout columns, and bedrock grout curtain)
- Dilation of the rock mass sufficient to open up joints and fractures previously "sealed" via the single row bedrock grout curtain.

The setback criterion is determined based on open pit stability and seepage assessments as well as the cave and crack angles for the ultimate mining depths. For the A154 dike a minimum setback of 100 m was selected as the criterion (Figure 3-1). This represented a guideline, one that was infringed upon along the northern perimeter of the A154 pit.

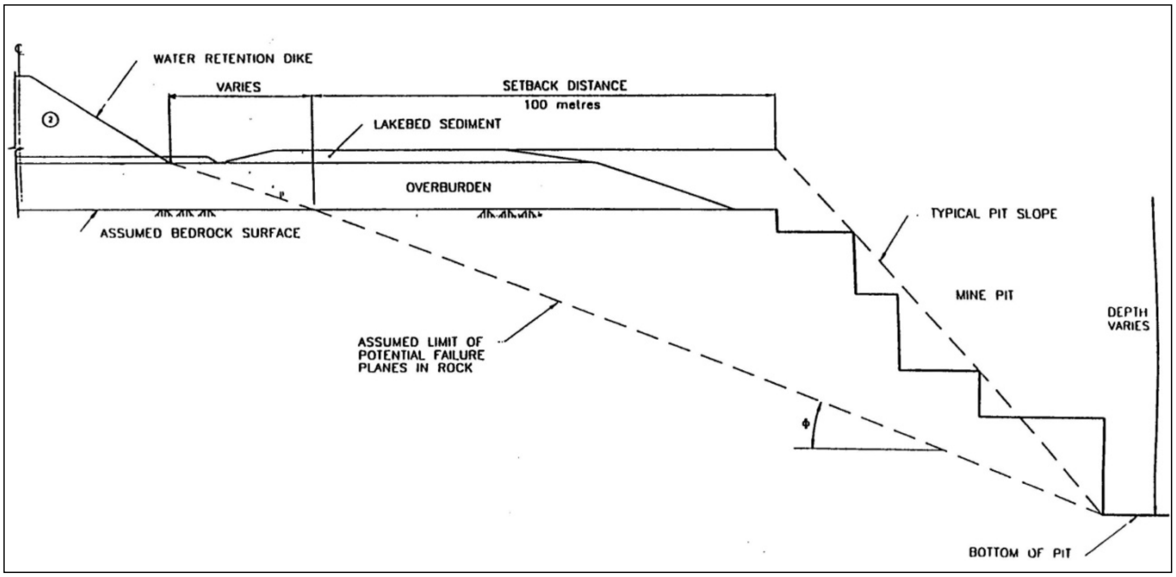


Figure 3-1. Pit setback criterion schematic: A154 dike

For the A418 dike, the dike alignment selected was a relatively economic one with the intention that the A418 pit would be excavated within that alignment with a safe setback as appropriate based on pit wall geotechnics and observed performance.

Golder (2006b) as part of the A21 feasibility study carried out a review of the geotechnical conditions of the A21 proposed pit/dike location and determined that an acceptable limit equilibrium factor of safety of 1.5 exists for pit failure geometries extending back to the A21 dike at its proposed 50 m setback. The setback was determined as follows (and as shown schematically on Figure 3-2):

- From the downstream toe of the main dike (not the toe berm), extend a line at 2H:1V through the lakebed sediments and till (overburden) to the bedrock interface.
- From that intersection, apply a horizontal setback distance of 50 m towards the pit. That defines the crest of the open pit (in bedrock).

Golder (2006b) further determined that the influence of pit wall dilation as far removed from the ultimate pit shell as the proposed dike would have a negligible effect in terms of differential strains induced in the bedrock below the dike. AMEC (2007) determined that the magnitude of these predicted strains in response to pit wall dilation would be well within the tolerance of the dike as governed by acceptable deformations of the cut-off wall element (as per analyses discussed in Section 8.4.4.3). As such, it is the requirement for a pit slope factor of safety of at least 1.5 (the minimum recommended factor of safety in the CDA 2007 guidelines for dam slopes under long term steady-state conditions), for failure geometry extending to the A21 dike that governs the acceptability of the 50 m setback, rather than predicted pit wall dilation.

Golder (2012) undertook additional pit slope stability and deformation analyses for the updated A21 open pit design which updated and validated this conclusion. It is of note that the A21 pit will be excavated to a maximum depth of 186 m below lake level, as compared to the A154 pit which was excavated to a maximum depth of 290 m below lake level.

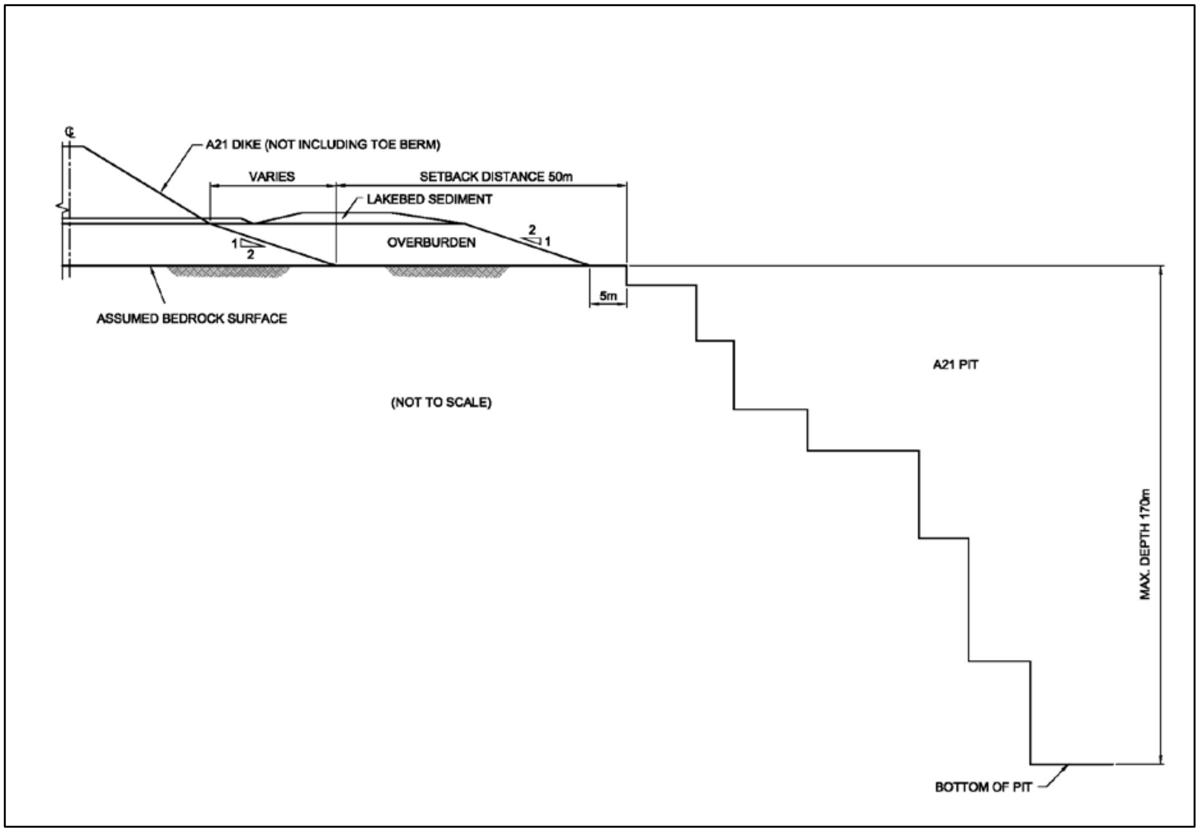


Figure 3-2. Pit setback criterion schematic: A21 dike

3.9. Upstream Erosion Protection

The upper upstream portion of the dike requires riprap protection from potential wave action in Lac de Gras. The riprap is not shown on the drawings. The design includes:

- Design wave governing riprap design: 1,000-year return period
- Riprap extent: Required down to El. 409.9 m (twice the design wave height below normal low lake level, which is El. 415.5 m)
- Riprap design: Stones 1.1 m to 1.6 m, layer thickness 2.8 m.

4.0 A21 DIKE: EMBANKMENT DESIGN AND CONSTRUCTION

4.1. Turbidity Barrier

A silt curtain is required as a turbidity barrier upstream of the A21 dike to control sediment generated from the lakebed foundation preparation and embankment construction activities. The barrier is designed to reduce the presence of suspended solids in the surface layer of water outside the barrier where wind generated waves and currents inhibit sedimentation. The specified turbidity barrier consists of a reinforced thermoplastic material suspended vertically in the water upstream of the dike during construction along the alignment illustrated on Drawing 14300-41D2-1003.1. A profile along the barrier alignment is shown on Drawing 14300-41D2-1003.2. A typical plan and details are shown on Drawing 14300-41D2-1003.3, along with the offset criteria governing the required offset of the barrier from the dredging limits.

The turbidity barrier (NKSL, 1999, 2004) will be fabricated from high visibility yellow 610 g/m^2 (18 oz/yd^2) weight polyester reinforced vinyl. Connectors will consist of shackled and bolted load lines with reinforced PVC pipe for fabric closure. Flotation will be provided by 0.2 m of expanded polystyrene (28.3 kg/m or 19 lbs/ft minimum buoyancy). Ballast will be provided by an 8 mm galvanized chain. The top load line will be an 8 mm galvanized wire rope enclosed in heavy tubing. The support cable will be fixed on South Island, and intermediate anchors will be placed on the lakebed. The anchors will be marked by buoys. The barrier will extend to 1.0 m above the lakebed level, except where the water depth exceeds 20 m. In general, the barrier will be assembled on shore and towed into position by boat and anchored in place.

The panels are subject to significant tension imposed by the vertical motion of the upper part attempting to follow wave movements and the lower part being held by hydrodynamic drag. The A154 turbidity barrier experienced damage in several areas and subsequent repair following severe storms. The A21 dike is relatively more exposed within Lac de Gras and as such similar occurrences may be experienced, despite the more robust curtain adopted for A418 (and A21).

Daily monitoring and maintenance of the barrier will be required.

4.2. Foundation Preparation

4.2.1. On-Land Portions of the Dike

On-land portions of the dike are to be founded on ice-poor till or bedrock. Unacceptable materials such as organic materials and ice-rich soils and massive ice in the active zone of the foundation will be excavated. Boulder lags, which are extensive along the shorelines, will be removed, except below the Zone 3 upstream shell limits. The foundation excavations will be advanced to a competent foundation capable of supporting the dikes. The cut-off trench excavations on the A21 dike abutments will be excavated as per the details given on Drawing 14300-41D2-1008, Section D.

4.2.2. In-Lake Portions of the Dike

Specifications for the A154 and A418 dikes called for the removal (via excavator in shallows, and dredging in deeper water) of lakebed sediments, such that the dike would be founded on lakebed till or bedrock. For both dikes, however, it was found that the dredging was unable to fully remove the firmer, lower horizon of lakebed sediments, despite which the dikes experienced no instabilities during construction, pool dewatering, and post-dewatering steady state seepage conditions.

Pre- and post-dredging probing of the lakebed sediments was undertaken for the A154 dike construction, as summarized in Table 4-1. The pre-dredging thicknesses are based on the piezocone soundings listed. The post dredging thicknesses represent an average of four probe points surrounding the piezocone sounding location (it was not possible to probe the precise piezocone sounding location), hence the designation of the dredged thickness as "average". The results of this pre and post-dredging probing indicate:

- In over half of the cases (18 out of 31), none of the lower (firmer) sediments were removed
- In only 5 of the 31 cases was 100% of the indicated upper and lower sediments removed
- In a third of the cases (11 out of 31) not even all of the upper (very soft) lakebed sediments were removed.

Despite the indicated limited effectiveness of the dredging, the A154 dike, with heights greater than proposed for A21, has performed well, with deformations less than predicted, and no indication of high foundation seepage pressures due to impeded drainage via the remaining lakebed sediments. As observed in the NKSL (2003) as-built report for the A154 dike construction:

> *After dewatering, the lakebed could be examined and it was apparent that the cutter was not able to dig into the fine silty sands and sandy silts due to the suction created on shearing of these soils. Nevertheless, the absence of problems during the embankment construction compared to the flow slides observed in other non-dredged areas indicated that the softer material was successfully removed.*

For the A418 dike, NKSL specified the use of a dredge with a more robust toothed cutter head (see Figure 4-1), specifically to remove the firmer sediments that the cutter suction dredge employed for A154 was unable to remove. DDMI (2007) noted that the depth of lakebed sediments removed during the A418 dike dredging was about 1 m, indicating a slight improvement relative to the dredged depths for the A154 dike noted in Table 4-1. However, given the combined thickness of the upper and lower sediments, it is unlikely that all of the lower horizon sediments were removed.

Table 4-1. Effectiveness of dredging for A154 dike.

A154 CPT Number	Sediment Thickness Prior to Dredging (m)			Average Dredged Thickness (m)	Soft Sediment Dredged (m)	Firmer Sediment Dredged (m)	% Soft Sediment Dredged	% Firmer Sediment Dredged
	Soft	Firmer	Total					
1	0.24	0.20	0.44	0.26	0.24	0.02	100	10
2	1.25	0.59	1.84	0.48	0.48	0.00	38	0
3	0.50	0.60	1.10	0.23	0.23	0.00	46	0
4	1.34	0.66	2.00	0.29	0.29	0.00	22	0
5A	0.64	0.00	0.64	1.09	0.64	0.45	100	NA
6	1.50	0.10	1.60	0.80	0.80	0.00	53	0
7	1.10	0.10	1.20	1.00	1.00	0.00	91	0
9	0.40	0.10	0.50	0.65	0.40	0.28	100	100
10	0.85	0.07	0.92	0.78	0.78	0.00	92	0
13	0.36	0.02	0.38	0.34	0.34	0.00	94	0
14	0.50	0.00	0.50	0.24	0.24	0.00	48	NA
18	1.06	0.04	1.10	0.20	0.20	0.00	19	0
19	0.70	0.68	1.38	1.33	0.70	0.63	100	93
21	0.35	0.40	0.75	0.67	0.35	0.32	100	80
22	0.92	-0.02	0.90	0.33	0.33	0.00	36	0
23	0.00	0.30	0.30	0.38	0.00	0.38	-	100
24	0.22	0.20	0.42	0.50	0.22	0.28	100	100
27	1.16	0.08	1.24	0.52	0.52	0.00	45	0
28	1.90	0.20	2.10	1.47	1.47	0.00	77	0
97	0.65	1.21	1.86	1.23	0.65	0.58	100	48
99	0.60	1.55	2.15	1.07	0.60	0.47	100	30
112	0.28	0.97	1.25	0.18	0.18	0.00	64	0
113	0.84	0.21	1.05	0.67	0.67	0.00	80	0
114	1.28	0.77	2.05	0.22	0.22	0.00	17	0
117	0.64	0.02	0.66	0.90	0.64	0.26	100	100
118	0.50	0.12	0.62	1.11	0.50	0.61	100	100
119	0.64	0.32	0.96	0.64	0.64	0.03	100	9
121	0.54	0.02	0.56	0.10	0.10	0.00	19	0
122	0.00	0.16	0.16	0.06	0.00	0.06	-	38
125	0.60	0.88	1.48	0.73	0.60	0.13	100	15
128	4.80	1.15	5.95	2.14	2.14	0.00	45	0
186	2.20	0.00	2.20	0.70	0.70	0.00	32	0
186A	1.14	0.16	1.30	0.70	0.70	0.00	61	0

Figure 4-1. Cutter head used for A154 dredging (left), and toothed cutter head used for A418 dredging (right). Photos from NKSL (2003 and 2007a, respectively).

Figure 2-4 shows representative photographs of the lower, firmer horizon of lakebed sediments exposed within the A418 pit area subsequent to pool dewatering. The upper two photos illustrate the lower horizon of the lakebed sediments for which A154 dredging was of limited effectiveness. The performance of the A154 dike, and the fact that the A418 dredging was unable to fully remove this material despite the more robust dredge cutter head, indicates that it is the upper, softer sediments horizon that warrants removal. In effect, the lower horizon of lakebed sediments are sufficiently dense as to resist removal by a toothed cutter suction dredge, and the experience of the A154 and A418 dikes demonstrates the lower sediments to represent a suitable foundation for the dike.

For the A154 and A418 dikes construction, the dredge spoils were pumped to storage sedimentation and clarifications ponds west of the North Waste Rock Pile for primary settling, with water subsequently directed to the North Inlet Pond. Dredged spoil from the A21 dredging operation will be pumped to Pond 3 (see Drawing 14300-41D2-1004.2).

Given the shallow water depths along much of the A21 dike alignment, the primary challenge for foundation preparation is expected to be significant concentrations of boulders rather than soft sediments. In fact, such sediments are, on the basis of drilling and geophysical investigations, absent along significant portions of the dike alignment in the shallows.

Where lake depths are less than 2 m, dredging is not possible in any case due to the 2 m draught required by the dredge.

Lakebed foundation preparation planned for the A21 dike is as follows:

- Dredge spoils will be directed to Pond 3
- Where boulders are encountered by the dredge, the boulders will be marked (via divers and floats) for subsequent removal via clam bucket
- Within the shallows (defined as water depths < 3 m), sediments are not present or, where present, are sufficiently thin that removal via dredging is not required
- Boulders in the shallows (water depth < 2 m) within the limits of the core fill or the filter blanket, will be removed, either from land, or from the advancing embankment

- Boulders in deeper water areas (> 2 m) below the footprint of the dike core fill and the filter blanket will, if deemed excessively large based on diver inspections, be removed via clam bucket.

The dredging and boulder removal areas referenced above are shown in plan on Drawing 14300-41D2-1004.1

4.3. Typical Dike Cross-Section

The typical cross-section consists of a zoned rockfill dike placed directly on the till/lower lakebed sediments foundation with a cut-off wall seepage barrier. The composite cut-off used for the A154 and A418 dikes is retained for A21 and includes a plastic concrete diaphragm wall extended with jet grouting at the base with curtain grouting into bedrock as shown on Drawing 14300-41D2-1007.

The embankment section comprises a central crushed stone Zone 1/1B material (50 mm minus) supported by a downstream crushed stone shell Zone 2 material (200 mm minus) underlain by a filter blanket of crushed stone Zone 1A (56 mm minus) or 1C material (50 mm minus) and an upstream quarried rock Zone 3 shell material (900 mm minus). The central portion of the crushed stone Zone 1 material will be vibro-densified prior to the construction of the cut-off wall. The report in Appendix C outlines the changes to material gradations associated with Zone 1.

The main characteristics of the Dike A21 typical cross-section are as follows:

• Crest width (m):	25.0
• Crest elevation (m):	421.0
• Outer slopes:	upstream 1.6H: 1.0V downstream 1.7H: 1.0V
• Top level of central Zone 1 (m):	418.0
• Boundary of Zones 1/2 & 1/3:	0.75H: 1.0V (nominal)
• Depth of toe drainage trench (m):	1 m (min) or bedrock level
• Type of cut-off wall:	plastic concrete diaphragm wall constructed via CSM, extended with jet grout and curtain grouting.

The width of the core is determined at the 418 m elevation where a working platform is required for the construction of the cut-off. The crest width of 25 m is defined by the continuation of the exterior slopes up to the crest elevation at 421 m.

Drawings 14300-41D2-1007 and 14300-41D2-1008 show the embankment design section within the lake and the design sections applicable in the abutment and on-land portions of the dike.

For land areas up to elevation 417 m, a freeboard dike will include a shallow cut-off consisting of an open trench (excavated to refusal on bedrock) backfilled with plastic concrete. For land areas between elevations 417 m and 421 m (following subgrade preparation), permafrost will preclude seepage at depth. The freeboard sections towards the ends of the dike are shown in Drawing 14300-D2-1008, Sections D and E.

4.4. Construction Materials

The construction materials for the A21 dike will be obtained from stockpiles of non-reactive (Type 1) waste rock. The waste rock will be hauled from the North Rock Pile by DDMI mine trucks to stockpiles or directly to the crusher. Primary, secondary and tertiary crushing is required to produce Zones 1, 1A, 1B, and 1C materials. Primary and secondary crushing is needed to produce Zone 2 material. Zone 3 consists of selected pit-run mine rock. The gradation characteristics of Zones 1, 1A, 1B, 1C, 2 and 3 are presented on Figure 4-2, and are enumerated in Table 4-2. The zoning of the dike is shown on Drawings 14300-41D2-1007 and 1008.

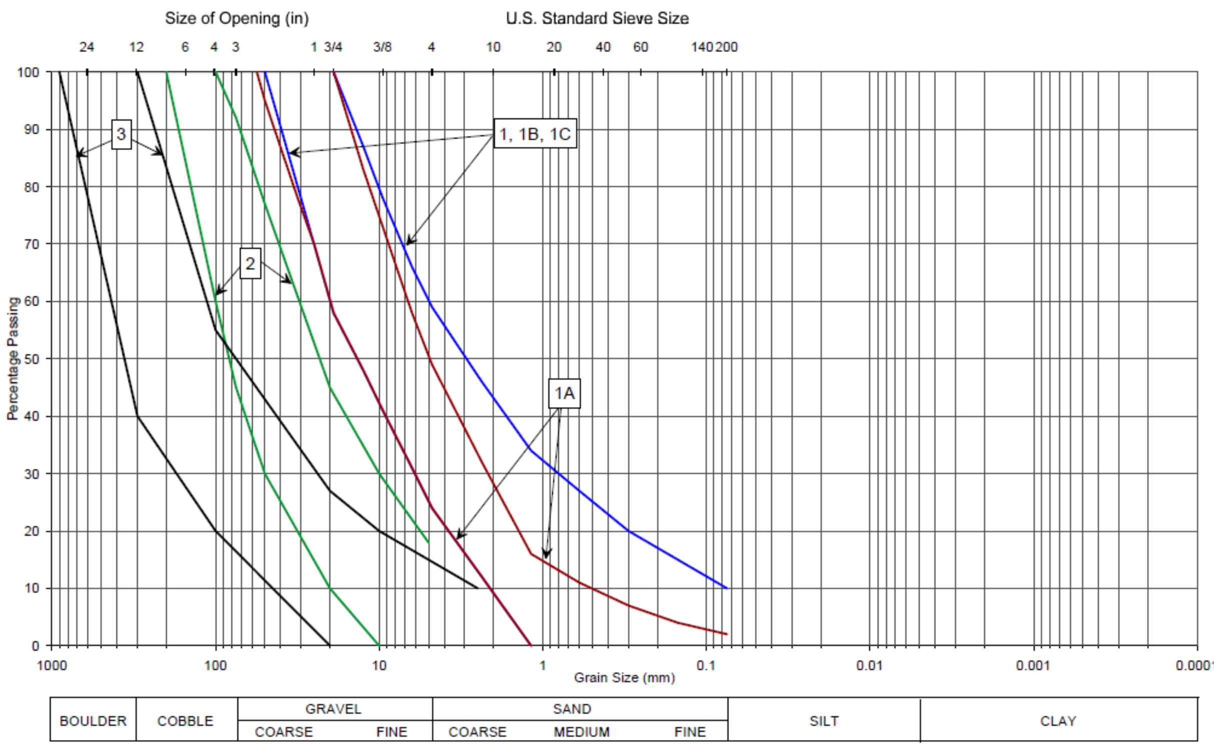


Figure 4-2. A21 dike fill zones: gradation specifications

The gradation specifications for the various zones are unchanged from the 2012 design with the exception of Zones 1, 1B, and 1C, where a finer material is specified in the current design. The rationale for this modified specification is as outlined in Appendix C.

Table 4-2. Dike fill zones: gradation specifications

ZONE	Sieve	% Passing by Weight
1A (1A = filter blanket)	56 mm	100
	50 mm	95 - 100
	31.5 mm	78 - 100
	25 mm	70 - 100
	19 mm	58 - 100
	12.5 mm	48 - 83
	9.5 mm	41 - 73
	6.3 mm	31 - 58
	4.75 mm	24 - 49
	2.36 mm	12 - 32
	1.18 mm	0 - 16
	0.6 mm	0 - 11
	0.3 mm	0 - 7
	0.15 mm	0 - 4
	0.075 mm	0 - 2
1, 1B, 1C (core and portion of filter blanket in non-critical areas)	50 mm	100
	31.5 mm	80 - 100
	25 mm	70 - 100
	19 mm	58 - 100
	12.5 mm	48 - 87
	9.5 mm	41 - 78
	6.3 mm	31 - 66
	4.75 mm	24 - 59
	2.36 mm	12 - 46
	1.18 mm	0 - 34
	0.6 mm	0 - 27
	0.3 mm	0 - 20
	0.15 mm	0 - 15
	0.075 mm	0 - 10
2 (downstream shell)	200 mm	100
	100 mm	60 - 100
	75 mm	45 - 92
	50 mm	30 - 77
	20 mm	10 - 45
	10 mm	0 - 30
	5 mm	0 - 18
3 (upstream shell)	900 mm	100
	300 mm	40 - 100
	100 mm	20 - 55
	20 mm	0 - 27
	10 mm	0 - 20
	2.5 mm	0 - 10

Note that Zones 1, 1B and 1C have identical grain size envelopes. The difference between these three zones is that Zones 1 and 1B are the end-dumped dike core zone, whereas Zone 1C is the clamshell-placed filter blanket. The usage of the various Zone 1 materials is as given in Table 4-3.

Table 4-3. Zone 1 materials usage

Zone	Method of Placement	Where Placed	Water Depth (m)
1	End-dumped core into the lake	Lakebed elevation < El. 410 m	≥ 6
1A	Clamshell placement (filter blanket)		
1B	End-dumped core into the lake	Lakebed elevation > El. 410 m	< 6 m
	Fill placed in the dry above initial dike platform at 417.5 m	Along entire length of dike	Not applicable
1C	Clamshell placement (filter blanket)	410 m < lakebed El. < 414 m	< 6 m

Although a maximum allowable fines content (% by dry weight finer than 0.075 mm) of 2% is specified for Zone 1A, fines contents as high as 4% are acceptable provided that the average, as confirmed by testing, is no more than 2%. The 2% fines content limitation is based on achieving a sufficiently high hydraulic conductivity as to quickly dissipate positive shear induced pore pressures generated due to contractant behaviour of the material and, thus, limit the potential for static liquefaction. Test work for the A154 and A418 dikes as documented by NKSL (1999) formed the basis for this specification.

Zone 6 is the drain rock that will be placed around the toe drain pipe (see Drawing 14300-41D2-1022). The gradation specification for Zone 6 is as given in Table 4-4.

Table 4-4. Zone 6 gradation specification

Size	Percent Finer Than By Dry Weight (%)
28 mm	100
20 mm	90 - 100
10 mm	25 - 60
5 mm	0 - 10
2.5 mm	0 - 5

4.5. Fill Placement Sequencing

4.5.1. Filter Blanket (Zone 1A/1C)

The first portion of the embankment to be constructed is the filter blanket (see Drawing 14300-41D2-1005 for plan extent, and Drawing 14300-41D2-1007 for its position within the dike section).

Zone 1A/1C fill will be placed on the prepared lakebed foundation by clamshell from a barge or from the edge of the advancing embankment. The filter blanket material must be placed rather than dumped to avoid segregation detrimental to its function as a filter between the underlying lakebed till and the overlying Zone 2. GPS systems will be used to control the placement to achieve both continuity of the blanket and avoidance of over-building with the premium cost material.

The filter blanket in areas where the lakebed (following dredging) is below El. 410 must conform to the low fines content specifications for Zone 1A as given in Table 4-2. The 2% fines content limitation for Zone 1A is a necessary constraint as the filter blanket material will be in the loosest possible state and, unlike the core of the dike, cannot be treated via vibro-densification, and is not restrained to either side by rockfill shell zones. The filter blanket underlies the downstream shell of the dike and therefore should liquefaction occur could potentially result in failure of the embankment.

At lakebed levels above El. 410 m, with lesser dike heights, concerns associated with potential liquefaction and dike instability are lessened, hence the gradation specification for Zone 1C is less restrictive than for Zone 1A.

4.5.2. Embankment Construction in the Wet

Fill will be placed “in the wet” for Zones 1/1B, 2 and 3 to about El. 417 m, advancing from both abutments. In order to maintain zone boundaries as per the design, Zone 1/1B fill shall be placed slightly in advance of the other zones, using the “slip displacement” method successfully employed for the A154 and A418 dikes. This method, via simultaneous advance of the Zones 1, 2 and 3 working fronts, achieves (based on underwater profiling and tracking of zone volumes placed) the nominal 0.75H:1V contacts between Zone 1 and Zones 2 (to the downstream) and 3 (to the upstream) shown on Drawing 14300-41D2-1007.

Zone 3 and the crushed materials (Zones 1 and 2) will be dumped on the platform just behind the leading edge. A dozer will push the material forward and downward to create a slip displacement by which the rolling segregation of the materials can be minimized.

The dike outer slopes (1.6H:1.0V upstream and 1.7H:1V downstream) shall be monitored during the placement of Zones 2 and 3. The slopes may initially be constructed slightly steeper (particularly in the case of the coarser Zone 3) and later flattened to the neat design line at upper elevations reachable using an excavator. Trimming of the downstream slope to the design slope will be carried out after dewatering of the pool.

In two areas of the dike (see Section 8.2.6 and

Table 8-4), lakebed slopes are adverse for stability of the upstream Zone 3 shell. To achieve acceptable factors of safety against embankment failure in these areas, toe buttresses will be placed underwater, via clamshell, prior to general embankment advance in those areas. Drawing 14300-41D2-1006.1 shows the areas where the toe buttresses are required; Section 8.2.6 specifies the configurations.

With the first stage of embankment construction completed to El. 418 m, the dike core (Zone 1) will be compacted via vibro-densification (see Section 4.6). This is required to reduce deformation of the dike core fill and plastic concrete cut-off during pool dewatering.

4.5.3. Embankment Construction in the Dry

At the abutments where the ground is above the MNWL, the embankment will be raised "in the dry" to a level corresponding to the working platform elevation, EL. 418.0 m. Once the cut-off wall installation is completed (jet grouting will be completed in early 2018), the embankment will be completed to its final crest El. 421 m, at which point in time dike instrumentation will be installed in advance of pool dewatering.

All zones shall be placed simultaneously in lifts of not thicker than 0.5 m for Zone 1B and 2 and 1.0 m for Zone 3 and compacted by a minimum of six passes of a 20 T vibratory compactor.

4.6. Vibro-Densification of the Dike Core

Vibro-densification of the 56 mm minus core of the A154 and A418 dikes was required to facilitate stability of the bentonite slurry supported cutoff trench excavation (for plastic concrete diaphragm wall construction), minimize slurry loss from the trench, and to stiffen the core zone to reduce deformations induced in response to pool dewatering to the downstream of the completed cutoff wall. For the CSM method, vibro-densification is still required, but only for the purposes of stiffening the Zone 1 core.

Vibro-densification will be achieved via suspending an approximately 25 m long probe from a 200 tonne crane. Vibro-densification will involve application of high energy vibrations coupled with water supplied under pressure to the tip of the probe, and addition of Zone 1B material (via a small wheeled loader) to the depression formed at the surface of the fill (at El. 418 m when vibro-densification is carried out) as the probe is slowly retracted and the underlying fill is densified.

The spacing for the vibro-densification probe holes is as shown on Drawing 14300-41D2-1019, and is consistent with the successful experience on the A154 and A418 dikes.

For each of the A154 and A418 dikes' construction, QC for the vibro-densification involved Becker Hammer Penetration (BPT) testing. The acceptance criteria was a BPT blow count of 20 per 30 cm of penetration over the full depth of the dike. BPT testing occurred at considerable expense. A21 QC and QA will involve checking the operating parameters of the probe, tracking the volume of Zone 1B material added to compensate for the volume reduction, calibrating this against the tracked volumes for the previous two dikes and completing geotechnical penetration and/or cross-hole geophysical testing. Acceptance criteria for the vibro-densification will be finalized during field trials.

4.7. Toe Berm

The toe berm will be constructed in the dry at the downstream toe of the dike once the pool dewatering is complete. It will be in the order of 2 m in height, or as necessary to maintain positive grades for the seepage collection pipe embedded within the toe berm. In areas proximal to the dike pumping stations (DPS-07 to DPS-09, Drawing 14300-41D2-1022) the thickness of the toe berm will be greater, comparable to the toe berm heights for the A154 and A418 dikes, to protect the pumping stations from freezing and to maintain suitable road access and dewatering pipeline grades. Thermal protection requirements for lesser height segments of the toe berm are described in Section 5.10. Toe berm profile and sections are provided on Drawing 14300-41D2-1022. The crest of the toe berm will have a width of 15 m with a downstream slope of 2H:1V.

4.8. Infield Seepage and Runoff Management

The area between the downstream toe of the dike and the pit crest will require grading to manage seepage and runoff, as shown on Drawing 14300-41D2-1021. The dike seepage will be routed through the perforated pipe below the toe berm directly to a dike pump station. The runoff will be routed to collection basins (Drawings 14300-41D2-1022 and 1024) at the toe of the dike pump stations and then pumped to the North Inlet Water Treatment Plant. The pump station and collection basin capacity is based on the highest of the following:

- Maximum water level in the collecting pond during the most critical rainfall event with a 100-year return period, with the pumps running at full capacity
- Maximum water level in the collecting pond during the most critical snowmelt event with a 25-year return period and the pump running at full capacity
- Water level reached in the pond after 24 hours during the most critical rainfall event with a 100-year return period and all pumps out of order
- Water level reached in the pond after 24 hours during the most critical snowmelt event with a 25-year return period with all pumps out of order.

Drawing 14300-41D2-1024 shows representative sections of the collection basin sections. The pump stations, including associated pipelines, mechanical and electrical aspects, are being designed by others.

4.9. Relief Wells

As established by NKSL (1999) in the design of the A154 dike, relief wells are to be installed from the downstream crest of the A21 dike, in areas where the dike foundation elevation is less than 402 m. The relief wells, installed at 40 m spacing, are to extend into bedrock by a minimum of 5 m (see Drawing 14300-41D2-1025-2, Stage 8). The slotted well screen will be located in bedrock, with another slotted section to allow discharge into the Zone 2 fill. The relief wells will be installed as a contingency, in advance of pool dewatering, to provide for a means of seepage pressure relief in areas where high exit gradients could develop, and thus provide an additional line of defense to the filter blanket. The relief wells can also be pumped during the pool dewatering stage if necessary to reduce foundation pore pressures to safe levels.

This same measure was incorporated into the designs of the A418 dike (NKSL, 2004). For the A418 dike construction, relief wells were also installed (NKSL, 2007c), post pool dewatering, from the crest of the toe berm between Sta. 0+650 m and 0+850 m in an area of upward seepage gradients (see Section 8.3.5).

5.0 CUT-OFF WALL DESIGN AND THERMAL DESIGN ASPECTS

5.1. General

The cut-off wall design for the A21 dike is similar to the adapted design for A154 and A418 dikes, which comprised the following components:

- A plastic concrete cut-off wall, formed by tremie placement of batch plant concrete into bentonite slurry-supported trenches that extended through the dike core zones and as much as 3 m into the underlying lakebed tills, or to bedrock if encountered before.
- A single row grout curtain installed at least 15 m (or 0.75 times the elevation difference between the dredged lakebed and the lake surface) into bedrock.
  - Given the relatively shallow water along the A21 alignment, most of the dike will have the 15 m grout curtain depth.
  - In two areas of the dike alignment (Sta. 0+525 to 0+850 and Sta. 1+325 to 1+500), a 25 m depth is specified as described in Section 5.2.
- A jet grout barrier overlapping with the overlying plastic concrete wall (by 1 m for A21 with CSM, and 0.5 m for the previous dikes with the plastic concrete diaphragm wall) and the underlying bedrock grout curtain (by 1.5 m) to form a single, vertically and laterally continuous, three-phase cut-off.

The cut-off wall construction methodology planned for the A21 dike will replace the slurry-supported trenches by application of Cutter Soil Mixing (CSM) as the construction method to form the barrier within the dike core, and extending 3 m into the underlying lakebed till (or to bedrock if lakebed till thickness is less than 3 m). The CSM cutter head will refuse on cobbles and boulders in the till, so large diameter boreholes will be pre-drilled through the dike core fill and into the underlying lakebed till to allow the CSM wall to be extended through the pre-drilled zone to 3 m into the underlying lakebed tills (or to bedrock if lakebed till thickness is less than 3 m). In areas where the dike is underlain by glaciofluvial deposits, to be identified during pre-drilling, the plastic concrete cut-off wall may extend deeper than 3 m. The bedrock grout curtain design remains unchanged from the A154 and A418 dikes, apart from the areas of deeper curtain grouting as indicated above. Jet grouting will be used, as per the previous dikes, to provide for the continuity between the CSM and grout curtain portions of the cut-off wall.

The cut-off wall construction sequence planned for the A21 dike is summarized as follows:

1. Pressure grouting into bedrock through the vibro-densified Zone 1 dike core and the underlying lakebed till, to a minimum depth below top of sound bedrock of 15 m or 25 m in specified areas (see Drawing 14300-41D2-1020.1). The depth of grouting may be deepened in certain areas if water pressure testing indicates the base of the nominal 15 m or 25 m zone to be in conductive bedrock. The grout curtain constitutes the first phase of the cut-off wall construction. An advantage of

constructing the grout curtain first is that the depth of the contact between the lakebed till and bedrock can be defined via the grout boreholes.

2. To expedite the CSM process and achieve a key of the plastic concrete into the lakebed till, large diameter boreholes approximately 1180 mm diameter will be drilled through the vibro-densified Zone 1 dike core to 3 m into lakebed till (or to bedrock, if till is less than 3 m in thickness). The drilling process will remove Zone 1 and till (with cobbles and boulders) material from within the borehole and the borehole will be subsequently backfilled with 12.5 mm minus crushed material prior to drill casing extraction. The pre-drilling hole pattern is shown on Drawing 14300-41D2-1020.2. In areas where the dike is underlain by glaciofluvial material, pre-drilling may extend into the foundation soils more than 3 m.
3. After pre-drilling is complete, a cut-off wall comprising overlapping panels (see Drawing 14300-41D2-1020.2) will be constructed via the CSM method from the crest of the dike to the base of the pre-drilled holes.
4. A jet grout zone (overlapping jet grout columns, see Drawing 14300-41D2-1020.3) will provide continuity between the base of the CSM cut-off wall and the underlying grout curtain. The extent of the jet grout portion that completes the cut-off wall will be by a minimum of 1 m into the overlying CSM wall above the bedrock surface, and a minimum of 1.5 m into the underlying bedrock grout curtain. Contingencies for localized widening of the cutoff specifically around such obstructions (e.g. large boulders) via jet grouting should also be accounted for.

Each of these portions of the cut-off wall is described more completely in the following sections.

5.2. Bedrock Grout Curtain

The seepage cut-off wall for the dike will be keyed into bedrock via a single row grout curtain. This is to seal off fractures and joints in upper bedrock formed as a result of frost jacking, stress relief due to glacial rebound, and other origins. The curtain grouting cannot be relied upon to effectively treat the lakebed till to bedrock interface. Reliance is instead placed on jet grouting to affect a cutoff at that interface. Monitored performance of the A154 and A418 dikes has indicated this approach to be highly effective.

Drilling and grouting for the bedrock grout curtain in advance of the other cut-off wall components will define the bedrock surface elevation. This in turn will facilitate drilling for the jet grouting operations.

For the A21 dike, the grout holes will be drilled to a minimum depth of 15 m (measured vertically) below the surface of the bedrock for most of the dike alignment. Two areas (Sta. 0+525 to 0+850 and Sta. 1+325 to 1+500) have bathymetric lows on the upstream side of the dike that extend below the elevation at which the 15 m grout curtain would terminate, resulting in reduced seepage path length below the base of the grout curtain. To mitigate this, these two areas will have the primary grout holes extended an additional 10 m to terminate a minimum of 25 m below the surface of the bedrock as shown on Drawings

14300-41D2-1009.1 through 1009.6; deepening of secondary and higher order grout holes will be determined based on primary hole grout takes. Selected grout holes may be extended to greater depths if water pressure tests indicate relatively high hydraulic conductivities in bedrock at the 15 or 25 m depth, or in any areas of relatively high hydraulic conductivity at depth as identified in the 2007 geotechnical investigations along the A21 dike alignment (see Golder reports in Appendices A and B). Those investigations included drilling and water pressure testing to depths of up to 30 m. Additional drilling and water pressure testing will be undertaken in advance of the curtain grouting, as described in Section 14.6.

The grout curtain installation details are presented on Drawing 14300-41D2-1020.1 A trial curtain grouting program will be executed to establish an optimized primary hole spacing. At present, the primary spacing is set at 6 m, the same as that used for the A154 dike curtain grouting program, and half the spacing initially used for the A418 curtain grouting program.

Curtain grout holes will be drilled through Zone 1 of the dike fill at 75^{o} measured from the horizontal, except at the abutments where, if required, the hole inclination in the plane of the cut-off may be changed to better match the grout hole pattern with the permafrost boundary. Additional holes and/or modified inclinations are typically required at bends in the dike alignment to achieve closure of the grout curtain. Based on predominant structure orientation relative to the axis of the dike, some optimization in certain areas may be required to maximize the likelihood of the grout holes intersection both sub-horizontal and sub-vertical geologic defects within the bedrock. This optimization will be achieved on the basis of the structural mapping used for the open pit geotechnical design, and on the basis of core holes drilled from the dike embankment in advance of the curtain grouting operation.

The grout holes will have a minimum diameter of 50 mm. The grout curtain holes will be water-pressure tested at 5 m intervals using a double packer system after the hole drilling is completed, prior to grout injection. Grout injection will be carried out in 5 m increments in a stage up manner (starting at the bottom and working upwards) using a single packer. Injection of the grout will continue at each setting until the refusal criterion is achieved. Generally, a relative thin grout mix (higher water to cement ratio) will be used at the start of each stage, and the mix progressively thickened thereafter. Results of water pressure tests will guide the initial grout mix and the rate at which the mix is thickened. Water to cement ratios will typically vary between 0.4:1 and 0.8:1. Considerable discretion will be given to the grouting engineer supervising the work in terms of mixes and the timing for thickening on the mixes depending on takes, flow rates and pressures. The program will be supervised by an engineer experienced in injection grouting.

The refusal criterion is 0.2 litre/min per m of stage for 10 minutes at maximum required pressure. Takes of 25 kg of cement or more per lineal meter will require supplementary holes (tertiary and quaternary). The length of additional holes (tertiary or quaternary) shall be such as to exceed by at least 5 m the bottom elevation of the stage in which an absorption greater than 25 kg/m has been recorded. Tertiary holes will also be required when the permeability is greater than 3 Lugeons. If the rate of injection does not achieve the refusal criterion after the injection of 1,000 litres of grout then the content of super-plasticizer will be decreased, and successively thicker mixes will be injected. The required grouting pressure at the mid-point of a stage is calculated at plus 20 kPa per meter measured vertically from the top of the grouting platform to the top of bedrock, plus 25 kPa per meter, measured vertically below the bedrock surface. The minimum pressure required at mid-stage is 75 kPa above the hydrostatic head from water level and the maximum is 1,500 kPa.

Drilling deviation of every curtain grout hole will be measured prior to starting of the grouting process by introducing an inclinometer and measuring the verticality of the hole. If measured deviation is larger than the specified tolerable limits, or enough to leave a potential gap in the curtain, then the hole may be abandoned after backfilling with grout and an additional hole drilled and grouted to close any potential gap in the wall.

The required curtain grout mix design consisting of cement-bentonite-water should be tested in the laboratory prior to the grouting operation to verify its permeability.

The curtain grout mix will be the same as that employed at the A154/A418 dikes, which was generally as follows (per m^3):

- Water to cement ratio varied as discussed above, with the grout mix being progressively thickened per stage of grouting
- Bentonite from 0 to 3%
- Super-plasticizer from 0 to 2000 ml per 120 kg of cement
- Whelan gum from 0 to 0.1%.

Clean sand (less than 5% fines) can be added to the grout mix in areas of high grout takes.

On completion of grouting to the bedrock surface, the remainder of each grout hole shall be backfilled to the working platform level with a grout mix having a water/cement ratio of 0.5, by tremie grouting through a pipe extending to the top of the pressure grouting.

All parameters pertinent to the grouting operations (grout pressure, grout take, etc.) are to be monitored in real-time using electronic monitoring systems. Lugeon stage water pressure tests will be conducted in some holes to evaluate before and after-grouting hydraulic conductivities of the bedrock.

5.3. Pre-Drilling and Backfill

To facilitate penetration of the CSM cutter head through the dike core and into the till layer, pre-drilling into the lakebed till will be required. Till at the Diavik site includes cobbles and boulders, the removal of which are required so that the CSM equipment can penetrate the required 3 m into the till. The average till thickness along the A21 dike alignment is in the order of 5 m and thins out near the abutments; however the till thickness in some reaches along the cut-off wall alignment ranges, based on currently available site investigation data, from approximately 5 to 11 m. Glaciofluvial deposits are also expected to be encountered within the dike foundation as were experienced at the A418 and A154 dikes. Additional investigations are planned prior to cut-off wall construction, as discussed in Section 14.0.

The pre-drilled boreholes are expected to have a diameter of 1180 mm drilled at a center to center spacing of 870 mm (see Drawing 14300-41D2-1020.2). The holes will be extended from the working platform at El. 418.0 m to 3 m into the lakebed till, or to refusal on bedrock for till less than 3 m in thickness. If glaciofluvial material is encountered within the dike foundation till, the pre-drill holes may be extended based on field observations at the time of pre-drilling, to allow the CSM cut-off to be extended into such zones. As each hole is advanced, casing will be installed through the Zone 1 dike core and the lakebed till to prevent hole collapse. Depending on the drill rig capacity, casing installation may be facilitated by an oscillator which will slowly push and oscillate the casing into the subsurface. Both Zone 1 and lakebed till material within the borehole will be removed by drilling equipment which may include drilling augers, core barrels, chisels or other specialized tools when boulders are encountered.

During casing removal, the hole will be continuously backfilled, to a higher level than the bottom of the casing with graded 12.5 mm minus crush, which will form the aggregate component of the CSM cut-off. The borehole walls will remain supported at all times during this operation and prevent adjacent soils from collapsing into the hole.

5.4. Cutter Soil Mixing Cut-off Wall

The uppermost portion of the cut-off wall through the dike core zone and the upper 3 m of the lakebed till will be constructed using the CSM methodology. CSM equipment will initiate construction of the cut-off wall from the working platform at elevation 418.0 m and the cut-off wall will be extended from elevation 417.5 m to 3 m below the till surface as shown on Drawings 14300-41D2-1009.1 to 1009.6. As noted above, should there be significant amount of glaciofluvial zones within the lakebed till the pre-drill holes would be extended, facilitating deeper CSM application. A 0.5 m deep trench from elevation 418.0 m to 417.5 m will be excavated in advance of the CSM operation such that waste generated during the CSM penetration and mixing process can be stored and collected from the trench with an excavator or pump and disposed of in areas designated by DDMI. The length of the CSM cut-off wall will be approximately 1,800 m with depths ranging from approximately 2.5 m to 26 m. The average depth will be in the order of 12 m to 15 m.

The cut-off wall will be formed by a series of 0.8 m thick primary and secondary rectangular panels each being 2.8 m long, with minimum overlap of 0.2 m between the panels as shown on Drawing 14300-41D2-1020.2. Potential locations of the cement/bentonite mixing plants to support the CSM operation are shown on Drawing 14300-41D2-1014.

The design approach for the cut-off at the abutments consists of preservation (and enhancement via additional cooling) of marginal permafrost near the talik boundary, and aggradation of permafrost towards the lake (i.e. within talik) to achieve overlap between the cut-off wall and permafrost. This will be achieved by installation of thermosyphons at the abutments extending to 15 m landward of the talik-permafrost boundary, and 16 m to the lake side (see Drawing 14300-41D2-1018).

The CSM wall will be extended into the permafrost zone, terminating at base El. 412.0 m or to bedrock, as shown on Drawings 14300-41D2-1009.1 and 1009.5. Beyond the CSM cut-off wall abutment cut-off design sections apply as discussed in Section 5.6, and shown on Drawing 14300-41D2-1008.

A small guidewall will be placed along the alignment of the CSM cut-off wall immediately adjacent to the waste collection trench to assist with alignment and to support tracking panel completion. The size of guidewall will be established by the contractor; however it is expected that this wall will be substantially smaller than the guidewalls used previously for the excavate-and-replace construction method of plastic concrete cut-off wall construction as implemented for the A154 and A418 dikes.

5.5. Jet Grouting

Jet grouting is required to complete the seepage cut-off between the CSM wall and the single row bedrock grout curtain. The A21 jet grouting details are presented on Drawing 14300-41D2-1020.3. A continuous wall will be created by overlapping jet-grouted columns. This is achievable due to the effectiveness of the cutting jet with the nature of the soil, where the diameter of the grout columns could vary with depth according to changes in the soil conditions. Extending the jet grouting into bedrock will clean out joints in the bedrock surface, replacing the joint infill with jet grout, and to achieve an effective overlap with the single row grout curtain within bedrock. Within the lakebed till, a high velocity horizontal water jet will be introduced to cut and remove the fine portion of the till at the bedrock contact, leaving in place the coarser portion of the soil.

The continuity of the jet grout portion of the cut-off is highly dependent on operator expertise and QA/QC procedures.

The jet grout cut-off parameters are to be as follows:

- The upper limit of the jet grouting will be at least 1.0 m into the CSM wall so that continuity in the cutoff is achieved
- A triple fluid jet grouting system will be used for the project
- With this system, the jet grout hole spacing should not exceed 0.75 m
- The lower limit of the jet grouting will be a minimum of 1.5 m into bedrock, to overlap with the grout curtain
- Minimum overlap with curtain grouting - 1.5 m
- Jet grout holes inclination - vertical
- Target jet grout column diameter - 1.25 m
- Transverse jet grout cut-off width - 0.8 min. at columns overlaps.
- Rod withdrawal rate - not to exceed a rate of approximately 8 m/min.

The jet grouting technique consists of drilling 108 mm diameter holes using a hydraulic drifter installed on the drill rig with low-pressure water jet until the required depth is reached (i.e. a minimum penetration of 1.5 m into the bedrock). The jet grout will then be raised to achieve the key into the base of the CSM wall. The jet grout cut-off will be extended into frozen abutment till as indicated on the drawings.

The shape and size of the jet grout cut-off are dependent upon the following parameters:

- Nature of the material being grouted
- Withdrawal rate of the drill rods
- Rotation speed of the drill rods
- Hole spacing and deviation from vertical.

The above parameters must be defined prior to installing the cut-off wall. This may be carried out by execution of trial columns, but more effectively the site-specific experience already gained from the A154/A418 construction works. The withdrawal rate and rotation speed must be varied until the design requirements are met. Upon completion of the trial columns (required if a new specialty jet grouting subcontractor not yet experienced on the Diavik site undertakes the A21 jet grouting), rotary core drill holes will be carried out and recovered cores are tested. Because of the inability to visually confirm the continuity of the jet grout component of the cut-off, a comprehensive monitoring and QA/QC program is required, during the drilling and construction of each column, to provide confidence in the continuity of the jet grout cut-off. A data acquisition system is required to continuously monitor the following parameters during the jet grouting operation:

- Clock time
- Drilling speed (meters per minute)
- Thrust on drill rods (kN)
- Torque (kN m)
- Drill rod RPM's (revolutions per minute)
- Drilling fluid pressure (MPa).

The following is the list of equipment to be mobilized for the jet grouting cut-off wall:

- UBW 08 drill rig or equivalent
- Portable cement silo
- Portable mixing plant
- Motorized pump
- Air compressor.

Drilling deviation of every jet grout hole will be measured prior to starting of the grouting process of any columns by introducing an inclinometer and measuring the verticality of the hole. If measured deviation is larger than the tolerable limit of 1% or enough to leave potential gap in the cut-off wall, then the hole may be abandoned after backfilling with grout and an additional column constructed to close any potential gap in the wall. In practice, given drilling conditions within the lakebed till experienced during jet grouting for the A154 and A418 dikes, in particular due to the presence of boulders, this deviation has proven very difficult to achieve. As such, an essential part of the cut-off wall operation is to plot in three-dimensions the plastic concrete diaphragm panels, jet grouting data, and curtain grouting data to evaluate if, as a result of out of specification deviation of jet grouting holes, a potential gap within the cut-off exists. If such potential is identified, then additional jet grout column(s) are required to eliminate the potential gap.

The required jet grout mix design consisting of cement-bentonite-water should be tested in the laboratory prior to the wall installation to verify its strength, ductility and hydraulic conductivity.

The jet grout mix is expected to be the same as that employed at the A418 dike, which was as follows (per m^3):

- Water - 450 kg/m^3
- Cement - 360 kg/m^3
- Bentonite - typically 4.5-5% % by weight of water, or 20-23 kg/m^3.

5.6. Abutment Cut-Off Design Sections

5.6.1. General

Abutment design sections are shown on Drawing 14300-41D2-1008. The rationale underlying these design sections, and where they apply, are outlined in the following sections, and indicated on the cut-off wall profiles on Drawings 14300-41D2-1009.1, 1009.5 and 1009.6.

5.6.2. Design Approach

The design approach for the dike sections landward of the abutment permafrost boundaries, as devised by NKSL (1999, 2004) has proven successful for both the A154 and A418 dikes. In general, the same approach was adopted for the A21 dike. Considerations for the design and construction of the abutment sections for the Diavik dikes are as follows:

- An effective transition from an unfrozen section (i.e. the cut-off wall within the lake talik) to a frozen section in permafrost on the dike abutment is required. It is critical that a seepage “window” between the two not be created at the boundary defining talik versus permafrost.
- Protection and cooling of marginal permafrost near the talik boundary is required. Marginal permafrost must be protected against thaw due to seepage-induced heat flux and thermal changes induced by embankment construction.
- Where permafrost is present, the design and construction elements are to encourage preservation of that permafrost and the effective seepage barrier it provides. The cut-off in the dike is to extend through the active layer to tie into perennially frozen ground.
- Protection against frost heave in lakebed till in the talik in shallow water sections of the dike, near the abutments, requires consideration. Once exposed on the pit-side of the cut-off, permafrost will begin to aggrade into the previously unfrozen lakebed till. Given the inferred frost susceptibility of the till, there is potential for frost heave to

induce dislocations in the cut-off wall. NKSL (1999) concluded, on the basis of thermal analyses, that synthetic insulation was required to prevent significant frost heave when the till was encountered above El. 409.5 m. Updated geothermal modelling (Appendix L) showed, however, that by excluding thermal insulation there is limited risk for damage to the cut-off wall.

One aspect of the A21 dike that differs from the A154 and A418 dikes is that a substantial portion of the A21 dike alignment lies to the landward side of the inferred talik boundary on the south abutment, as shown on Drawing 14300-41D2-1033.1. Based on A21 alignment thermistor data (provided in Appendix L), the inferred location of the talik boundary towards the south abutment is at about Sta. 1+600 m, with the south abutment termination of the dike at Sta. 2+228 m. At the north abutment, only about 40 m of the dike alignment lies to the landward side of the currently estimated talik boundary, coincident with the shoreline at that abutment. The relatively short distance between the talik boundary and the termination of the dike at the north abutment is similar to the abutments for the A154 and A418 dikes. For the south abutment, however, 628 m of the total 2,228 m length of the dike lies to the landward side of the talik boundary.

5.6.3. Abutment Conditions

The A21 dike will have two abutments, north and south. The alignment crosses a small island at about Sta. 0+500 m, but data from two thermistors on that island indicates a talik, with only seasonally frozen ground. As such, the conventional dike cut-off will be applied there. This is in contrast to the A154 and A418 dikes, which also traversed small islands, but those islands were sufficiently large as to allow the development of permafrost to some depth.

Cross sections through the south abutment of the dike and underlying foundation (including the pit wall) were developed at 100 m intervals (see Drawing 14300-41D2-1033.2) from Station 1+600 to Station 2+100 to illustrate the south abutment setting. These sections are shown on Drawings 14300-41D2-1033.3 and 1033.4. Current ground level at these stations ranges from El. 415 m (slightly below typical lake level) at Sta. 1+600 m, to El. 417.5 at Sta. 2+100 m. These sections also show the MNWL and the MDWL.

The on-land sections of the A21 dike abutment will be constructed above the shoreline, at nominal El. 415.8 m (MNWL). The on-land portions of the abutments extend from Sta. 0+000 to 0+040 (40 m length) for the north abutment and from station 1+823 to 2+228 (405 m length) for the south abutment. Based on current topography, the on-land segments of the dike in the south abutment area will be constructed:

- Above foundation El. 417 m (MDWL) from Sta. 2+050 m to Sta. 2+228 m
- Between foundation El. 417 m and 415.8 (MNWL) from Sta. 1+823 m to Sta. 2+050 m.

From Sta. 1+823 m to 1+600 m, Drawing 14300-41D2-1032.1 indicates the dike to be in the lake, but the water is sufficiently shallow in this area that permafrost is present, as indicated by thermistor strings installed in boreholes at about Sta. 1+700 m (borehole A21-D07-02), Sta. 1+823 m (borehole BH-160T), and Sta. 1+960 m (borehole A21-D07-01T). The locations of these thermistor installations are shown on Drawings 14300-41D2-1010.6 and 14300-41D2-1033.2.

Photographs of the abutment areas are provided in Figure 5-1 (mostly south abutment) and Figure 5-2 (north abutment). The south abutment photos show extensive boulder lags in the south abutment area. Once these boulder lags are removed (where removal is necessary), current elevations could be lowered by a meter or more. A much more abrupt abutment condition is evident at the north abutment.

North abutment of A21 dike. Bedrock exposed at surface. Arrow indicates thermistor A21-D07-05.

South abutment area - boulder lags in shoals area, at about Sta. 1+560 m.

Arrow indicates thermistor installed in borehole A21-D07-02, at about Sta. 1+700 m.

South abutment, photo taken looking towards the lake, from about Sta. 1+850 m.

Panorama of south abutment peninsula and shoals area.

Panorama looking west along the south abutment peninsula. Arrows indicate thermistors BG-160T (foreground) and A21-D07-01 (background). Thermistor A21-D07-02 is further east of the location from which the photo was taken (about Sta. 1+750 m).

Figure 5-1. A21 dike abutment photographs (August 2007).

North abutment of A21 dike, looking south. Arrow indicates thermistor A21-D07-05.

North abutment of A21 dike, looking west. Arrow indicates thermistor A21-D07-05.

Figure 5-2. A21 dike north abutment photographs (August 2007).

5.6.4. South Abutment Dike Design Sections

5.6.4.1. Design Section E: Foundation El. > 417 m

This section applies for abutment areas at elevation 417 m (MDWL) or higher, once stripping to a suitable foundation is completed. For this portion of the dike, no cut-off wall is required as this constitutes only the freeboard portion of the dike. The active layer is expected to penetrate below the dike into the till, but this is of no significant consequence. If any permafrost degradation occurs at these abutment areas, the potential settlement is expected to be minor due to the low height of the dike at this area (< 2 m), and the minimal thickness of till above bedrock. Settlement that does occur can be compensated for by raising dike section height as required. Thermal protection is not required, as this area is expected to freeze after construction during the first winter season, and the terrain is such that there is no ice-rich ground along the alignment that requires protection from annual freeze-thaw cycles. This design section would apply from approximately Sta. 2+050 m to Sta. 2+228 m, based on the current ground elevations.

5.6.4.2. Design Section D: 415.8 m < Foundation Elevation < 417 m

This cross section is proposed for the abutment areas at elevations (following foundation preparation) between the MNWL of El. 415.8 m (shoreline), and the MDWL of El. 417 m. This portion of the abutment will be constructed on potentially unsaturated till that, even if frozen, may have an unacceptably high permeability and permit seepage, with the associated heat flux potentially degrading the permafrost and exacerbating seepage.

For this section, the cut-off wall will be constructed to terminate in till at El. 412 m, or in bedrock if encountered above that elevation. The latter is expected to be the case for the entire stretch of dike between Sta. 2+050 m and the shoreline at Sta. 1+823 m. The cut-off for design section D will comprise trench excavation through till to bedrock, and backfilling with material to be determined. With a working platform at El. 418 m, the maximum depth of excavation to reach bedrock, based on currently available information, would be about 4 m. This design section would apply from approximately Sta. 1+823 m to 2+050 m, based on the current ground elevations.

5.6.4.3. In-Lake Dike Section on Permafrost - Design Section C

At the south abutment, from the shoreline (approximately Sta. 1+823 m) to the inferred permafrost-talik boundary at about Sta. 1+600 m, the dike alignment is within shallow water (less than 2 m in depth), but is situated atop permafrost. The inferred permafrost-talik boundary is at approximately Sta. 0+040 m at the north abutment which coincides within 10 m of the shoreline. The depth of the active layer in these areas is governed primarily by lake water temperature.

Design section C applies from the permafrost-talik boundary to 15 m to the landward side of that boundary, within the zone that is covered by thermosyphon banks at the north abutment, and approximately 223 m to the landward side of the boundary at the south abutment. This was the approach established for both the A154 and A418 dikes. This section includes the plastic concrete cut-off to the achievable depth within the lakebed till, and jet grouting to extend the cut-off to below the bedrock surface. The single row bedrock grout curtain is not included because the bedrock is below the active layer and would therefore be perennially frozen, rendering curtain grouting non-feasible.

The portion of this design section extending 15 m landward of the thermosyphon banks (centered on the permafrost-talik boundary at both abutments) must be the first portion of the cut-off completed, so that the thermosyphons can be installed, and the refrigeration plant activated to cool the marginal permafrost prior to pool dewatering. At the south abutment, this design section would apply from approximately Sta. 1+600 m to 1+823 m, based on the current ground elevations. At the north abutment, this design section would apply from 0+025 m to 0+040 m, based on the current ground elevations.

5.6.5. North Abutment Design Sections

A single thermistor string (A21-D07-05) is located at the north abutment of the A21 dike alignment. Based on interpretation of data from this thermistor string, the talik-permafrost boundary is interpreted to be near vertical and essentially coincident with the shoreline at Sta. 0+040.

To the lake side of the inferred talik-permafrost boundary at Sta. 0+040, the typical three-phase cut-off dike section will apply. From the landward side of the boundary, the sequence will be unchanged from AMEC (2012), and will comprise the following:

- Design section C (Drawing 14300-41D2-1008) from Sta. 0+040 to Sta. 0+025. This is section will include half of the 30 m length of dike covered by the north abutment thermosyphons bank.
- From Sta. 0+025 to Sta. 0+000, the ground elevation is 419 m or higher, once stripping to suitable foundation is completed. Therefore, design section E (Drawing 14300-41D2-1008) will apply for this portion of the abutment.

5.7. Cut-off Wall Design Sections Summary

Table 5-1 provides a summary of the dike alignment and the applicable design sections. This table also provides the range of lake depths, inferred lakebed till thickness, and thermal protection issues. Drawings 14300-41D2-1009.1 through 1009.6 show the cut-off wall profile inferred on the basis of the interpreted geologic profile along the cut-off wall alignment.

Table 5-1. A21 dike cross sections summary

Station	Max. & Min. Existing Elevations (m)	Lake Depth (m)	Expected Till Thickness (m)	Hydraulic Head at Base of Till (m) *(Based on MNWL)*	Applicable Section (See Dwgs. 14300-41D2-1007 & 1008)	Section Description	Thermal Protection Requirements	Thermal Protection Construction Considerations
North Abutment 0+000 - 0+025	EL. 421 - EL. 419	-	2 m	0	E	• Permafrost section (Land) • Min. ground elevation after preparation is expected to be higher than MDWL(EL. 417 m) • No cut-off wall is required	• No thermal protection is required • Frost heave not expected to be critical	
0+025 - 0+040	EL. 419 - EL. 416	-	2 m	0	C	• Permafrost section (Land) • Active freeze-thaw zone within lakebed till • CSM cut-off wall to be extended 3.0 m into the till foundation. • Jet grouting is required to key the CSM cut-off wall in bedrock	• 30 m of thermosyphons from 0+025 to 0+055 is required to overlap the Permafrost - Talik transition	• Thermosyphons installed in May 2017 once cut-off completed there - with active freezing during the summer
0+040 - 0+070	EL. 416 - EL. 415	0 m - 1 m	2 m	2 m - 3 m	B	• Non-Permafrost section (Talik) • CSM cut-off wall/grout curtain combination (with jet grout transition zone) to mitigate seepage • No filter blanket is required (ground is at EL. 414 m or higher)	• 30 m of thermosyphons from 0+025 to 0+055 is required to overlap the Permafrost - Talik transition	• Thermosyphons installed in May 2017 once cut-off completed there - with active freezing during the summer
0+070 - 0+184	EL. 415 - EL. 409	1 m - 7 m	3 m - 4 m	5 m - 11 m	A	• Non-Permafrost section (Talik) • CSM cut-off wall/grout curtain combination (with jet grout transition zone) to mitigate seepage • Filter blanket is required	• None	
0+184 - 0+270	EL. 415 - EL. 414	1 m - 2 m	4 m - 6 m	5 m -10 m	B	• Non-Permafrost section (Talik) • CSM cut-off wall/grout curtain combination (with jet grout transition zone) to mitigate seepage • No filter blanket is required (ground is at EL. 414 m or higher)	• None	
0+270 - 0+385	EL. 414 - EL. 410	2 m - 6 m	3 m - 6 m	5 m - 12 m	A	• Non-Permafrost section (Talik) • CSM cut-off wall/grout curtain combination (with jet grout transition zone) to mitigate seepage • Filter blanket is required	• None	

Table 5-1. A21 dike cross sections summary, continued

Station	Max. & Min. Existing Elevations (m)	Lake Depth (m)	Expected Till Thickness (m)	Hydraulic Head at Base of Till (m) (Based on MNWL)	Applicable Section (See Dwgs. 14300-41D2-1007 & 1008)	Section Description	Thermal Protection Requirements	Thermal Protection Construction Considerations
0+385 - 0+526	EL. 416 - EL. 414	0 m - 2 m	5 m - 6 m	5 m - 8 m	B	• Non-Permafrost section (Talik) • CSM cut-off wall/grout curtain combination (with jet grout transition zone) to mitigate seepage • No filter blanket is required (ground is at EL. 414 m or higher)	• None	
0+526 - 1+532	EL. 395 - EL. 414	2 m - 21 m	4 m - 12 m	6 m - 33 m	A	• Non-Permafrost section (Talik) • CSM cut-off wall/grout curtain combination (with jet grout transition zone) to mitigate seepage • Filter blanket is required	• None	
1+532 - 1+600	EL. 415 - EL. 414	1 m - 2 m	4 m - 6 m	5 m - 8 m	B	• Non-Permafrost section (Talik) • CSM cut-off wall/grout curtain combination (with jet grout transition zone) to mitigate seepage • No filter blanket is required (ground is at EL. 414 m or higher)	• 30 m of thermosyphons from 1+585 to 1+615 is required to overlap the Permafrost - Talik transition	• Thermosyphons installed in August 2017 once cut-off completed there - with active freezing immediately after installation
1+600 - 1+823	EL. 416 - EL. 414	0 m - 2 m	2 m - 5 m	2 m - 7 m	C	• Permafrost section (Lake) • Active freeze-thaw zone within lakebed till • CSM cut-off wall to be extended 3.0 m into the till foundation • Jet grouting is required to key the CSM cut-off wall in bedrock	• 30 m of thermosyphons from 1+585 to 1+615 is required to overlap the Permafrost - Talik transition	• Thermosyphons installed in August 2017 once cut-off completed there - with active freezing immediately after installation
1+823 - 2+050	EL. 417 - EL. 416	-	2 m	0	D	• Permafrost section (Land) • Min. ground elevation is lower than M.D.W.L (EL. 417 m) • Cut-off trench is required	• Thermal protection is required (for on-land portion)	• Construct cut-off trench in 2016 • Allow to freeze over winter
2+050 - 2+228 South Abutment	EL. 422 - EL. 417	-	2 m	0	E	• No cut-off wall required, as min. ground elevation is higher than M.D.W.L (EL. 417 m)	• No thermal protection is required • Frost heave is not critical	

5.8. Abutment Thermosyphon Groups

The transition between the talik and the permafrost (see Drawing 14300-41D2-1033.1) occurs beneath the dike abutments at locations inferred on the basis of thermistor data (see Appendix L) on the south and north ends, around the points in the shallow waters where the nominal lake depth is about 2 m. This is approximately at a lake bottom elevation of 413.8 m. However, due to the heterogeneous lake bottom topography, the contact is likely sub-vertical at both abutments, which is confirmed by the thermistor data available to date at the A21 dike alignment, and consistent with experience at the A154 and A418 dike abutments.

As established for the A154 and A418 dikes, the design approach at the contacts consists of preservation (and enhancement via additional cooling) of marginal and warm permafrost near the talik boundary, and aggradation of permafrost towards the lake (i.e., into the current talik) to achieve overlap between the three phase portion of the cut-off wall constructed within the talik, and permafrost. This is achieved by installation of thermosyphon groups at the abutments extending approximately 10 m to the lake side of the talik-permafrost boundary, and 12 m to the land-side, as shown on Drawings 14300-41D2-1015, 1017 and 1018. Further, existing permafrost along the shallow water and on-land sections of the dike will be protected from degrading, as this may cause thaw-induced settlements, pore pressure build up and dike deformations. Coarse, air-permeable rock fill will promote cold air convection along the dike shoulders. This fill promotes heat extraction from the foundation in winter and serves as insulation during the summer months, locking the cold into the structure and preserving the permafrost.

The cut-off wall (per design section C) will be extended a minimum of 15 m (horizontal) into the permafrost zone, 3 m land-ward of the thermosyphon group, and will be extended 1.5 m into bedrock.

The thermosyphon groups installed for the A154 and A418 dikes have performed successfully and therefore the proposed design for the A21 is based on the previous arrangement. The construction schedule currently proposed allows 4 month (June - September) of active freezing for the north group, but only 2 months (August and September) of active freezing for the south group before pool dewatering commences. A high level thermal assessment indicated that a closer freeze-pipe spacing in vertical as well as horizontal direction is required on warm side of the south abutment compared to previous settings and design recommendations prepared by AMEC in 2007 (Appendix K). As shown on the drawings, the design incorporates two rows of thermosyphons to either side of the cutoff wall alignment. Two pairs will be located approximately along the permafrost boundary, 3 to 4 pairs on the warm, upstream (i.e., lake) side and the remaining three pairs on the cold, downstream (i.e., abutment) side of the boundary. Tube spacing will be 3 m (north abutment) and 2.5 m (south abutment) on the warm side, 4 m on the cold side, with a 2.5 m (north) and 2 m (south) offset on both sides of the cut-off wall reference line. Two additional thermsyphons will be installed along the permafrost-talik contact, outside the central row. The pattern adopted for the Dike A21 north abutment is the same as that applied at the A154 and A418 dikes, comprising of 7 pairs (one upstream and one downstream freeze pipe) of thermosyphons and two more on the contact for a total of 16 tubes. On the south abutment one extra pair is proposed to allow for a shorter freezing time, currently proposed.

5.9. Thermal Protection for Unfrozen Lakebed Till Foundations

5.9.1. General

As the site is located in the continuous permafrost zone, in the shallower dike sections the frost line will eventually penetrate the whole embankment and reach the potentially frost-susceptible lakebed till, which is currently protected from freezing due to the Lac de Gras talik. Where the dike embankment is low, the frost would likely eventually reach the area where the cut-off wall is embedded within the till. As the till freezes, frost heave may occur, causing dislocations within the cut-off wall. The major driver for frost heave is water that freezes after having migrated from an unfrozen zone to a growing ice lens (Henry, 2000). However, if the frozen conditions can be maintained or sources of free water hindered, the risk for damaging the cut-off wall is limited to the initial freezing stage only. An increased level of risk of cut-off damage exists if the active layer reaches into the till layer, unlimited access to water exists, and freeze-thaw cycles occur repeatedly within the sections of the cut-off wall where it is embedded in the lakebed till. Ice lenses and frost jacking can then damage the cut-off wall with resultant increased seepage.

5.9.2. Approach Taken for A154 and A418 Dikes

The approach taken for the A154/A418 dikes was to provide synthetic insulation (extruded polystyrene sheets), buried to a shallow depth within the crest of the dikes in shallow water areas.

Thermal simulations were carried out during the design of the A154 dike (NKSL, 1999) to establish the risk of frost heave in shallow foundations for various depths below the crest considering the frost susceptibility and segregation potential of the till and the effective stress level in the foundation. NKSL concluded that synthetic insulation would be provided to prevent significant frost heave when the till was encountered above Elevation 409.5 m. The following extruded-expanded polystyrene thicknesses were proposed for the A154 and A418 dikes depending on the height of embankment above the till foundation:

- Till elevation above 412 m (water depth about 4 m): 300 mm thickness of synthetic insulation
- Till elevation between 412 m and 410 m (water depth 4 to 6 m): 200 mm
- Till elevation between 410 m and 409 m (water depth 6 to 7 m): 100 mm
- Till elevation below 409 m (water depth > 7 m): no insulation required.

Considering the shallow water alignment for the A21 dike, application of the A154/A418 criteria would require thermal insulation to be placed within the dike along about 1,300 m of the dike alignment. At a width of 8 m per lineal m of dike length, this translates to ~10,400 m^2 of thermal insulation, and a volume of about 2,800 m^3. The cost associated with trucking this material to site, and its handling during placement, is substantial. Accordingly, the need for such a design measure was revisited as part of this design update.

5.9.3. A21 Approach

In order to induce major frost jacking and ice lens formation, sufficient water must be available to drive the frost heave mechanism. The availability of water for major frost jacking, in the shallow water portions of the dike where frost jacking is of concern, is limited on the basis of the following:

- The in-field area between the dike toe and the pit rim is limited, and grading will be such to direct water away from the dike. Permafrost aggradation into the till downstream of the dike will limit infiltration.
- Piezometer data from the other dikes indicates that downward seepage gradients develop in the till relatively soon after pit development, particularly in shallow water areas where there are minimal driving heads.
- The proximity of the dike to the open pit (reduced setback relative to A154 and A418), and the single-phase pit development approach, will provide for relatively rapid depressurization of the dike foundation as pit sinking proceeds.

On this basis, and with allowance for appropriate monitoring (discussed in Section 10.3 and 10.5), BGC concludes that the inclusion of synthetic insulation as a means of protecting the cut-off against frost heave within the lakebed till can be deleted from the design.

5.10. Thermal Protection for Toe Drain System

5.10.1. General

The toe drain system construction will only be possible after completion of the cut-off wall and pumping of the in-dike pond. Based on the proposed schedule (Section 6.0, Note: Construction schedule provided to BGC on June 4, 2014

Figure 6-1), the toe drain and the berm will be constructed during spring, 2018. As permafrost aggradation will occur into the lakebed till following removal of lake water, the toe drain system could lose its functionality due to freeze-back of the toe drain pipes and surrounding fill. A toe berm will therefore be required in order to thermally protect the drain system from freezing during the initial phase when elevated pore pressures in the till under the downstream side of the dike can jeopardize the dike integrity. Freezing of the drain system would reduce the capability of reducing pore pressures within the foundation of the dike. The toe drain system will be required during the initial few years until such time as the drawdown effects of the open pit alter the seepage regime to an extent that the toe drain system becomes effectively redundant.

5.10.2. Approach Taken for A154 and A418 Dikes

For the A154 and A418 dikes, the toe berm was constructed to a minimum nominal height of 4 to 5 m above the toe drain trench, using Zone 2 (200 mm minus) rockfill. With this thickness of rockfill overlying the toe drain, no additional thermal protection was judged necessary. Using this same minimum toe berm thickness for the A21 dike is considered impractical as it would in effect replicate the height of the dike itself for a considerable portion of the dike alignment. The toe berm is not required for downstream slope stability.

5.10.3. Approach Suggested by AMEC (2007) for A21 Dike

AMEC (2007, 2012) proposed the inclusion of extruded polystyrene within the toe berm, with sheets immediately above the toe drain, as a means of compensating for the reduced toe berm thickness. This would involve additional expense, both in polystyrene sheets shipping and handling, and in placement within the toe drain system.

5.10.4. Re-Evaluation of Toe Drain Thermal Protection Requirements

As a result of the foregoing, the need for thermal protection for the toe drain system was reviewed. The review included the tasks listed below.

- The thermistor and piezometer data for the A154 and A418 dikes were reviewed to assess the efficacy of the toe berm in retarding permafrost aggradation into the toe drains for those dikes, and the duration over which the toe drains were actually required for relief of foundation pore pressures. This review is presented in Section 8.6.3.2, with additional review of piezometer data in Section 8.3.5.
- Thermal analyses of the toe berm and toe drain system were undertaken. These analyses are summarized in Section 8.6.3.5, and presented in detail in Appendix L.

5.10.5. Thermal Design for A21 Toe Berm

The design approach adopted for thermal aspects of the toe berm and the toe drain, as further detailed in Section 8.6.3.5, was to encourage warming of the toe berm foundation in the summer months, and retard heat extraction from the ground in the winter months. This is to be achieved by the measures listed below.

- The toe berm will be capped using coarse processed kimberlite (CPK), 1 mm to 6 mm in size and trucked from the process plant to the processed kimberlite containment facility. The CPK is dark grey in color and is known to get very warm during the summer months. A dark surface of CPK atop the toe berm during the summer would add more heat to the fill and the underlying ground where the toe drain is located. The CPK could also be placed over the toe berm slope which would provide for further warming, and would also retard cold air convection (heat extraction) from the toe berm in the winter months.

- The temperatures of the fill placed immediately around the toe drain should not be colder than 1°C and the toe berm material itself not below -2°C at the time of placement to prevent enclosure of frozen ground that may not thaw the following summer. With pool dewatering currently scheduled for the winter months, construction of the toe drain and toe berm system in the spring of 2018 should be feasible.
- Snow fences will be used to collect snow along the toe of the dike and the top of the toe berm to insulate the underlying toe drains during the winter. A 2 m high snow fence should be placed about 1 m downslope from the edge of the dike crest along the entire length of the dike where the toe berm is present. The fence should have a porosity of 50%. A temporary snow fence using Tensar® Snow Fence, or comparable, can be used for the initial three winters during which it is required to collect sufficient snow for the toe berm. Ground temperature monitoring will be used to assess the effectiveness of the snow collected using the snow fence on the berm temperatures.

The construction schedule for the toe berm is an important consideration in prediction of thermal performance and specification of thermal protection measures. For both the A154 and A418 dikes, the toe berm was constructed in the winter months, locking in cold ground temperatures, despite which the systems performed adequately. The dewatering schedule for A21, as mentioned above, calls for pool dewatering in the winter months, such that A21 toe berm construction could take place in spring or possibly early summer. This would be highly advantageous to avoid locking cold temperatures into the ground below the toe berm. It would also avoid winter construction of these features which would likely reduce costs given more favourable working conditions.

5.11. Frozen Cut-Off on Dike Abutments

On the dike abutments, below the MDWL, the cut-off will eventually be keyed into frozen ground. However, the till may be part of the active layer and be unfrozen during the construction of the cut-off wall, depending on the construction schedule. The cut-off wall will be located at the reference line of the dike where the fill is thickest. Therefore, the freezing of the till within the active layer where the cut-off is embedded will start after the freezing of the till under the upstream and downstream slopes of the dike. Because the till layer under the dike slopes will be frozen, the availability of water available to generate damaging frost jacking at the cut-off during the initial winter following the summer construction will be minimal.

The proposed design intent is to:

- Promote rapid permafrost formation in the initially unfrozen lakebed till
- Limit the potential number of freeze-thaw cycles within the till at the location of the cut-off wall.

The time of construction and related water pumping will play a key role in the permafrost aggradation. As most of the dike construction will occur during summer, it is expected that the dike will be constructed on unfrozen till in summer 2016 to crest El. 418 m, and the cut-off constructed prior to the onset of winter. At the end of the winter, the dike will be raised to crest El. 421 m to lock in the cold temperatures. This additional cover of 3 m will be, per thermal modeling presented in Appendix L, sufficient to protect the till from the active layer penetration and permafrost will aggrade in the till around the cut-off, enhancing the seepage barrier.

5.12. Freeboard Dike on Permafrost

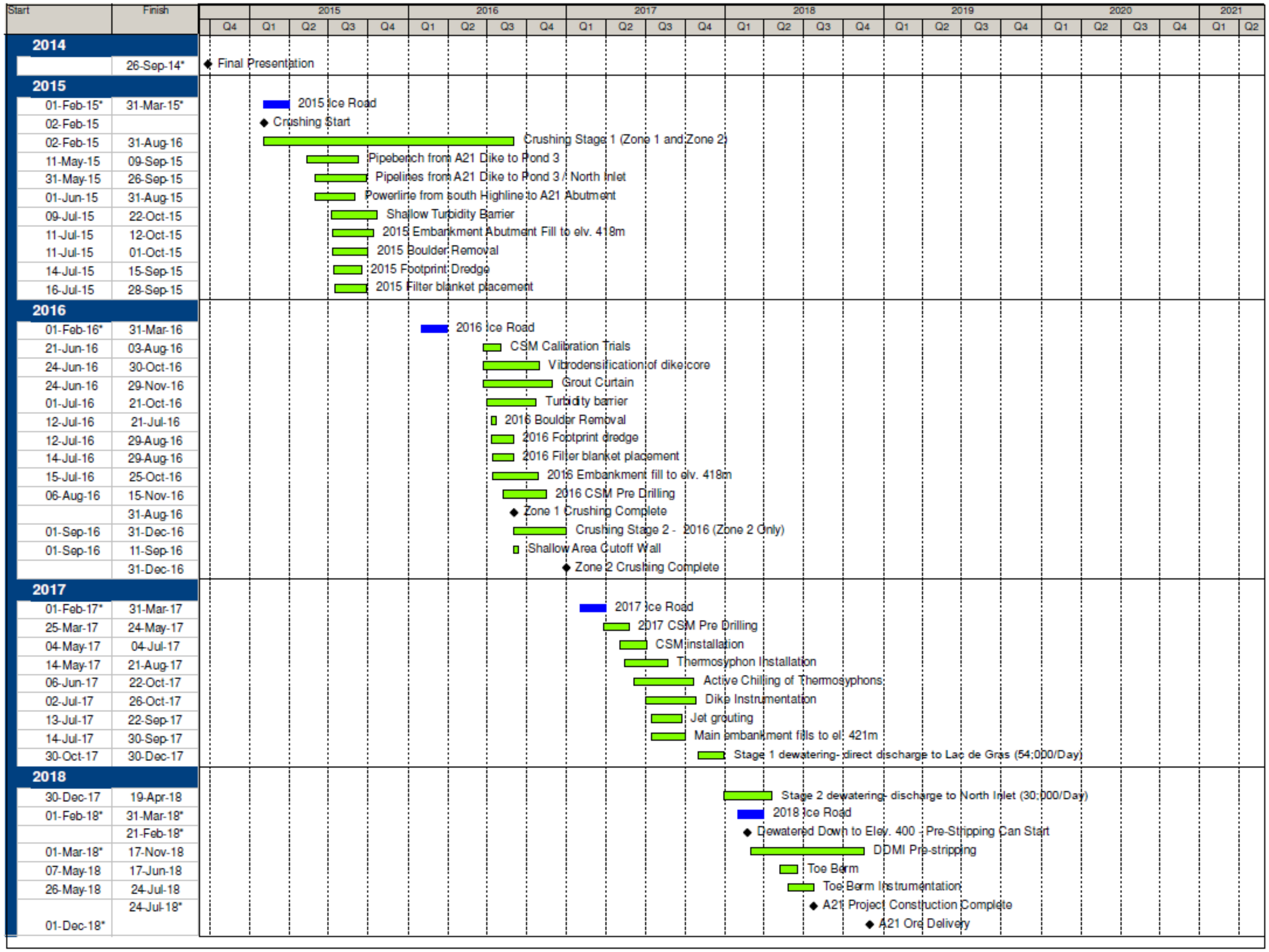
A freeboard dike on permafrost is required on South Island to achieve rim closure at El. 421 m on both abutments of the dike. Since foundation preparation will take place during summer when the ground surface is thawed, caution will be required to minimize disturbance of the underlying permafrost. Excavation of boulders and organic soil will be achieved by supporting the excavation equipment on a granular pad that is advanced as the stripping operation progresses. No frozen material is to be excavated, unless ice-rich soils or massive ice are encountered. Investigations of the A21 abutment areas have given no indication of such conditions - bedrock appears to be within 2 m of the ground surface, based on GPR surveys (Golder, 2007; and Drawing 14300-41D2-1010.5) and the three south abutment thermistor boreholes (Drawing 14300-41D2-1010.6).

The stripped foundation and the construction pad will be allowed to freeze back for one winter season before completing the foundation treatment and the capping fill during the next spring. This is to prevent thawed soil being buried beneath the dike.

6.0 DIKE CONSTRUCTION SEQUENCE AND SCHEDULE

6.1. General

This section outlines the sequence and techniques to be used for the construction of the A21 dike. Construction sequences are schematically shown on Drawings 14300-41D2-1025.1 and 1025.2. A Gantt chart schedule is shown in Figure 6-1.



Note: Construction schedule provided to BGC on June 4, 2014

Figure 6-1. A21 dike construction schedule.

6.2. 2014 Activities and Milestones

1. Approval to proceed with feasibility study update: January 2014.
2. Completion of feasibility study and TEG/BED review meeting: August 2014.
3. Rio Tinto Investment Committee (IC) Project approval and funding: October 2014.
4. Undertake detailed construction and logistics planning, and identify early works to be undertaken in 2015. These works to include completion and grading of causeway to South Island, roads on South Island, laydown/stockpile areas and pipelines required to support dredging in 2016 as well as foundation preparation and embankment construction for the northern most portion of the dike.
5. Initiate crusher upgrades in late 2014.
6. Issue tender documents for pipeline installation, power line installation, camp/office renovation, crusher upgrades and dike dredging/foundation preparation. Evaluate bids and award contracts in time for 2015 ice road.

6.3. 2015 Activities

7. Contractor mobilization on the 2015 ice road. Off-load and assemble/erect equipment and facilities.
8. Install pipe benches and pipelines from A21 to Pond 3 and North Inlet.
9. Install powerline from the A21 decline to the dike construction office area.
10. Establish site facilities and offices for DDMI team and contractors.
11. Construct South Island roads and laydown/stockpile areas.
12. Complete crusher upgrades in early 2015 and commence crushing, via the DDMI crushing plant and begin stockpiling of crushed product using DDMI equipment.
13. Construct a portion of the A21 embankment extending from the north abutment to Island A to approximately Sta. 0+500 m. Descriptions of each construction activity are outlined in Section 6.4. Embankment construction activities in 2015 include:
    a. Installation of shallow turbidity barrier.
    b. Foundation preparation including both boulder removal and dredging via clamshell.
    c. Filter blanket placement (Zone 1A/1C).
    d. Embankment fill placement (Zones 1, 2 and 3) to El. 418.0 m.
14. Undertake updated bathymetric survey of the A21 dike alignment (see Section 14.2), and conduct overwater acoustic surveys along and transverse to the cut-off wall alignment (see Section 14.5).

6.4. 2016 Activities

15. Contractor mobilization on the 2016 ice road. Off-load and assemble/erect equipment and facilities.

16. Carry out Ohmmapper geophysical surveys (from the winter lake ice) on the dike abutments to further delineate permafrost-talik boundary to refine locations for abutment thermosyphon groups (see Section 14.4).
17. Install turbidity curtain to protect Lac de Gras water quality during embankment construction (as soon as the lake is ice free). See Drawings 14300-41D2-1003.1 through 1003.3.

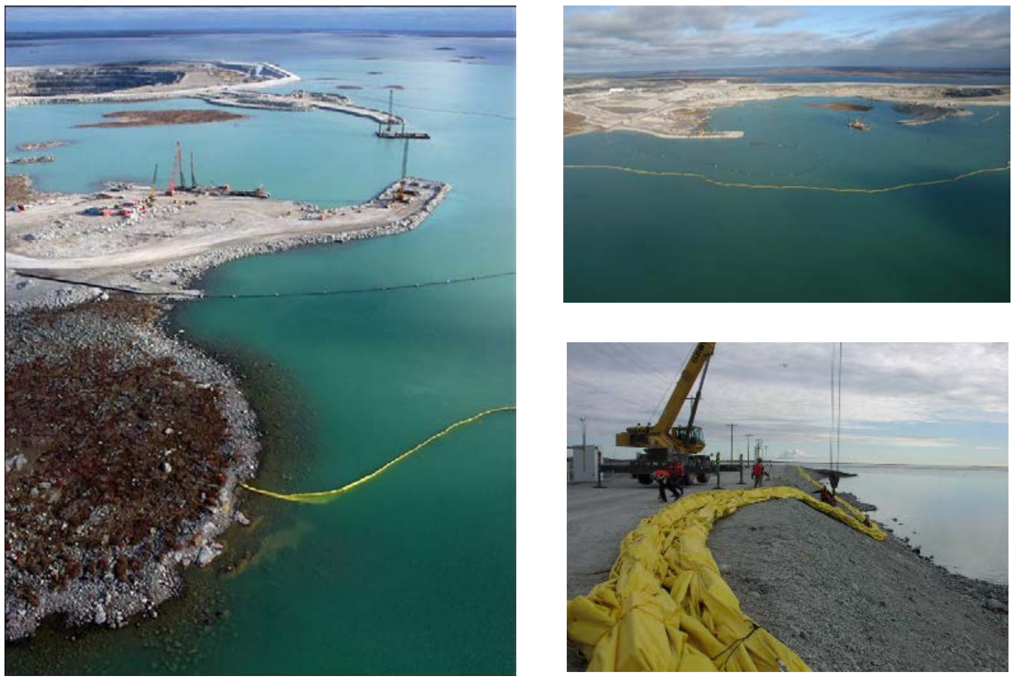
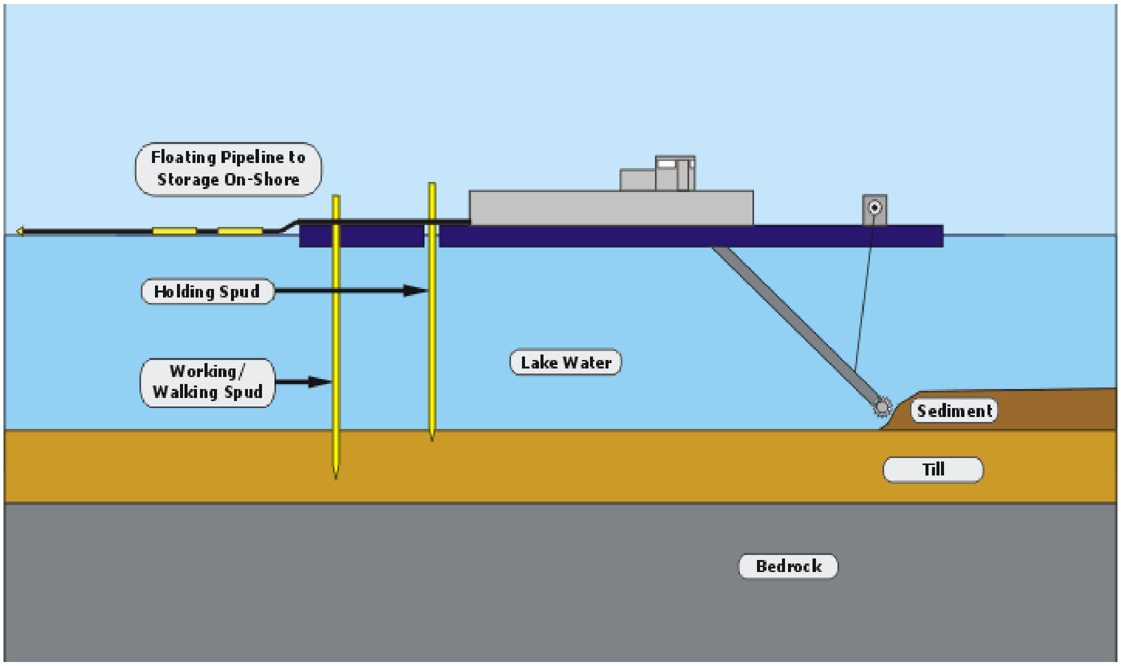


Figure 6-2. Turbidity barrier deployed around A418 dike

18. Removal of lakebed sediments (in deeper water areas) by cutter suction dredging (or applicable method) and boulders removal via land and marine based operations as required. Refer to Drawing 14300-41D2-1004.1 for extents of lakebed foundation preparation. Boulders are to be side-cast to the downstream side of the dike alignment. Soft sediments hydraulically removed will be pumped to and discharged in Pond 3 (Refer to Drawing 14300-41D2-1004.2 for pipeline routing).



Schematic representation of dredging set-up.

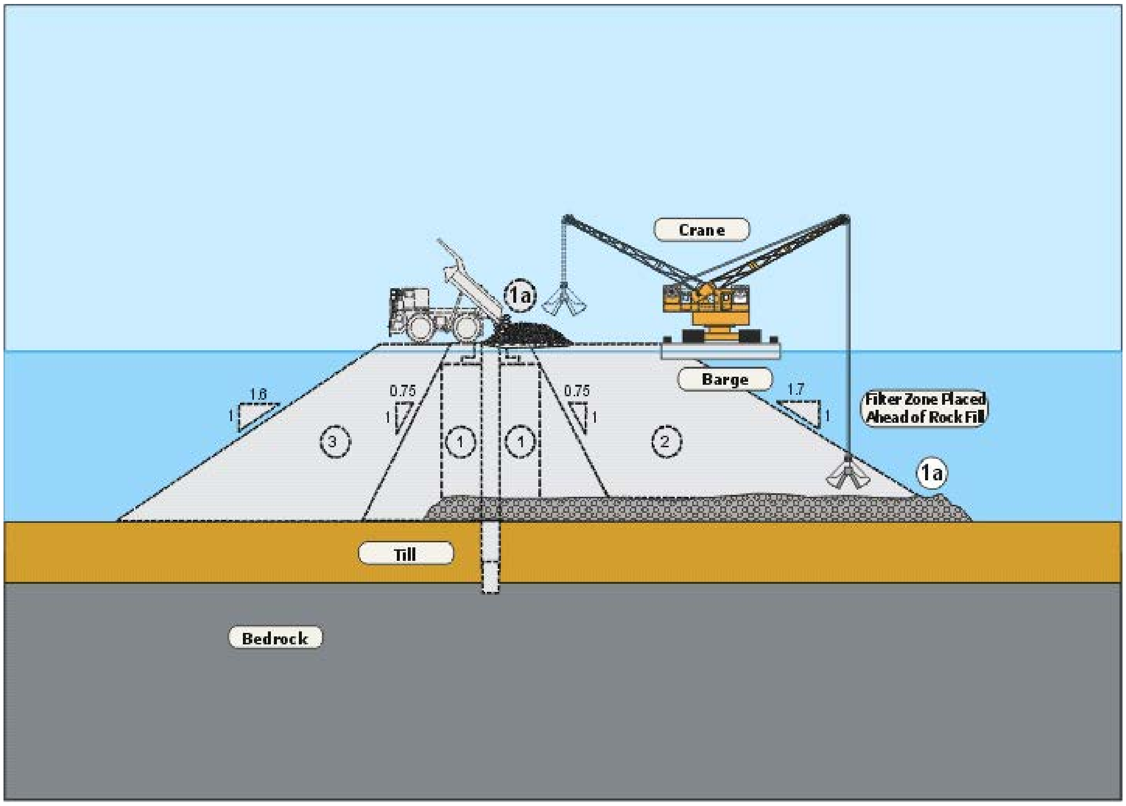
Boulder removal via clamshell, marine operation.

Boulder removal near abutments with excavator.

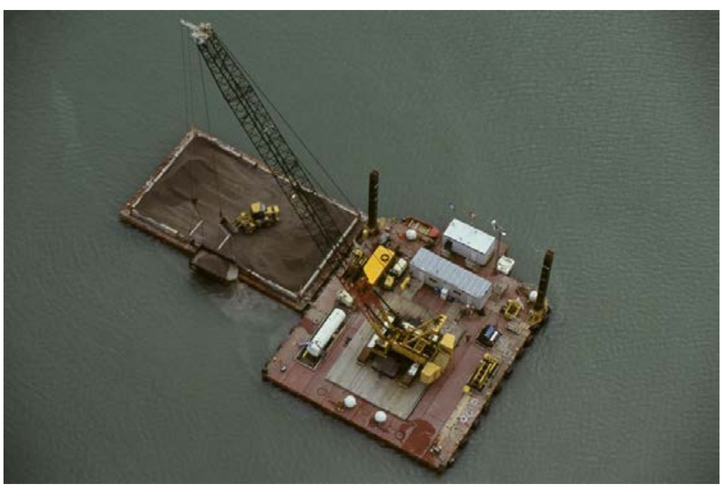
Figure 6-3. Foundation preparation phase of dike construction

19. Placement "in the wet" of 1 m thick filter blanket (Zone 1A below El. 410 m and 1C between El. 410 m and 414 m), within limits of the Zone 1/1B core, and below the Zone 2 downstream shell of the dike (see Drawing 14300-41D2-1005 for plan extent of filter blanket, and Drawing 14300-41D2-1007 for the filter blanket configuration in section). Placement will be carried out using a barge mounted crane equipped with a clamshell or a skip.

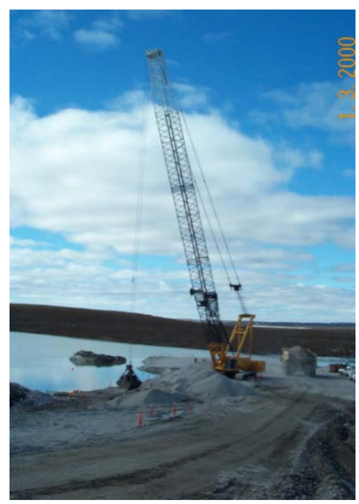
20. Fill placement “in the wet” to El. 418 m. The fill placement shall be carried out by advancing the central Zone 1 (0-50 mm) ahead of the downstream shell (Zone 2, 200 mm minus crushed rock) and upstream shell (Zone 3, 900 mm minus run-of-mine rock), with the sequence of placement such that the zone contact slopes are as indicated on Drawing 14300-41D2-1007. Placement methodology employed to achieve these contact slopes is referred to as the “slip displacement” method. Careful survey control and placement methods will be required such that theoretical design slopes can be achieved especially for material placed above El. 417 m. Complete enclosure of the A21 pool by late 2016. Removal of the turbidity curtain is required prior to lake freeze-up.
21. Foundation preparation on the abutment sections (including cut-off trench excavation and backfill) will be undertaken in fall of 2016. The prepared foundations will then be left over the winter of 2016-2017 to allow re-freezing of the foundations. The freeboard sections of the dike will be constructed immediately thereafter, in 2017.
22. Densify the Zone 1 core by vibro-densification to El. 417 m. See Drawing 14300-41D2-1019 for vibro-densification pattern. Vibro-densification will not be required where the lakebed till is above elevation 414.3 m (i.e. water depths less than 1.5 m).



Schematic of filter blanket placement.

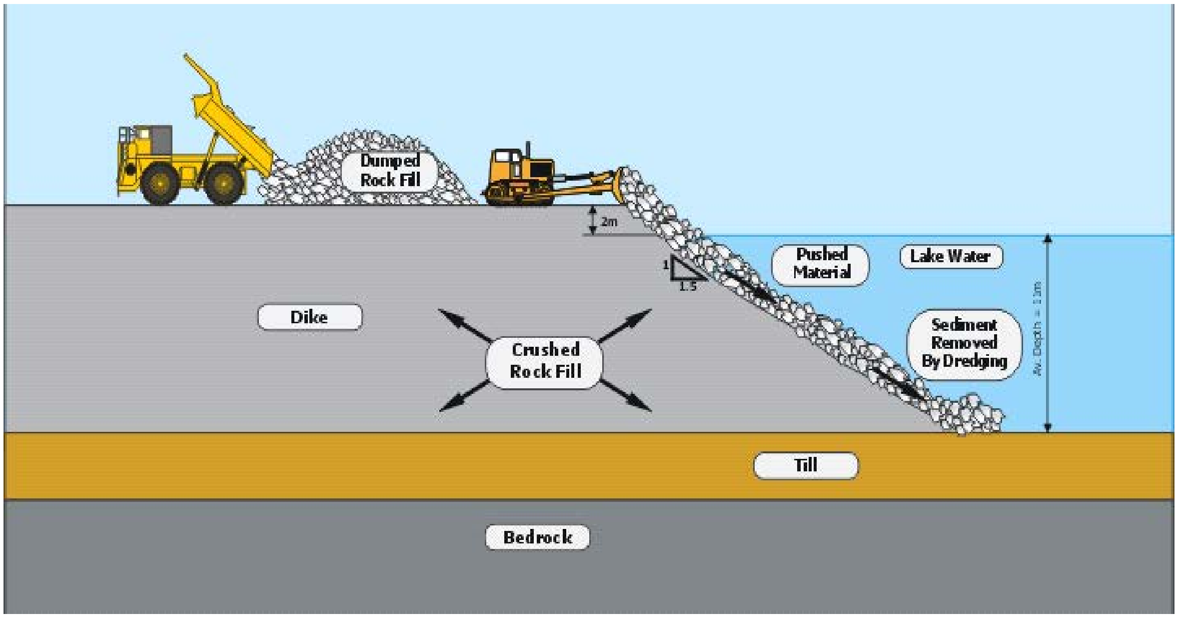


Marine filter blanket placement operation.



Filter blanket placement via crane and clamshell.

Figure 6-4. Filter blanket placement.

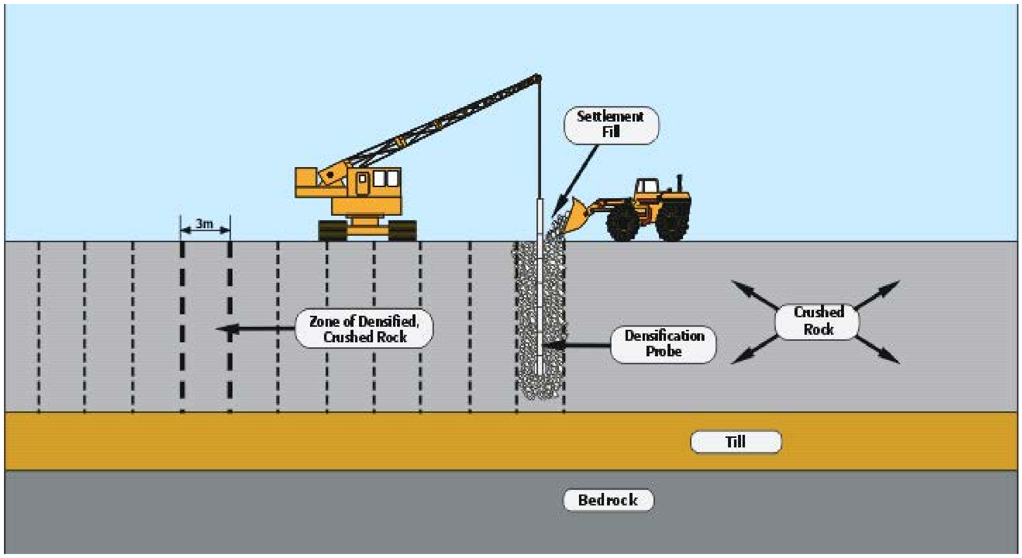


Schematic of rockfill advancement into the lake.

Embankment advancement (nearing closure of the A418 dike). Zone 3 to left, Zone 1 to center (being dumped by truck), Zone 2 to right.

Rockfill being pushed into the lake. To left is Zone 1, to right is Zone 3.

Figure 6-5. Dike embankment advancement.



Schematic of Vibro-Densification for dike core fill

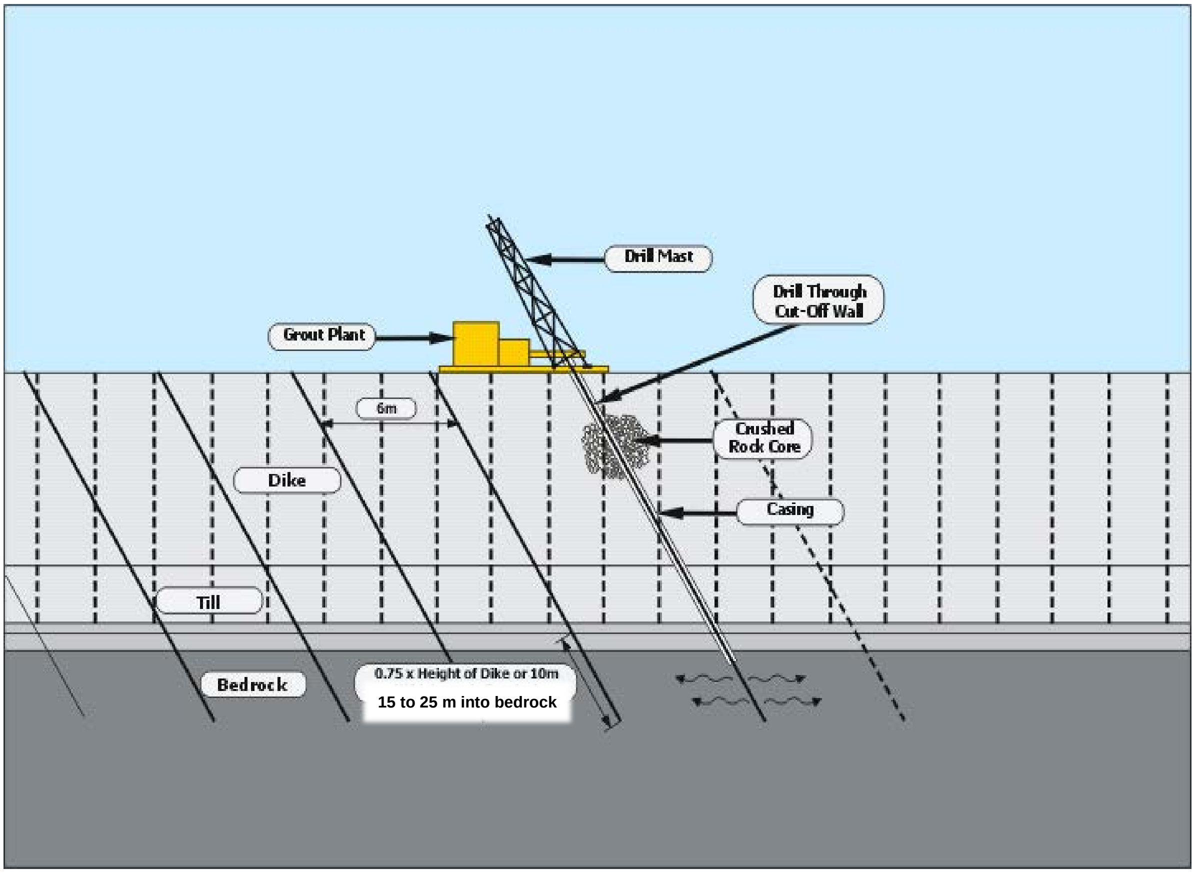
Vibro-Densification

Figure 6-6. Vibro-densification of dike core fill.

23. On-land reaches of the plastic concrete cut-off wall will fill a shallow, continuous trench (to bedrock) excavated by a hydraulic excavator rather than panels constructed via CSM equipment. See Typical Section D on Drawing 14300-41D2-1008.
24. Fabricate and install the concrete guide wall in preparation for CSM application.
25. Curtain grouting into bedrock (see Drawing 14300-41D2-1020.1 for grouting details). Drilling is through the embankment fills, prior to the plastic concrete cut-off wall construction. Curtain grouting is carried out into the winter.
26. Complete CSM and jet grout trial at the northern portion of the embankment fill. Purpose of trial is to confirm construction procedures for pre-drilling, CSM

application and jet grouting as well as optimize CSM mix design and QA/QC procedures. A portion of the trial will be located at the northern thermosyphon bank which will allow for earlier thermosyphon installation at this location in 2017.

27. Pre-drill the cut-off wall alignment through the embankment fill to a minimum of 3.0 m into the lakebed till or to bedrock. This will be initiated in 2016, with the balance to be completed in 2017. See Drawing 14300-41-D2-1020.2 for the pre-drilling pattern. The fully cased pre-drilled holes will be backfilled with 12.5 mm minus graded crush material produced by the DDMI crusher.



Schematic section of bedrock curtain grouting.

Curtain grouting rig (foreground). Winterized shelter for jet grouting in the background.

Figure 6-7. Bedrock curtain grouting.

6.5. 2017 Activities

28. Ice road mobilization, and demobilization of equipment not required beyond 2016.
29. Continuation and completion of pre-drilling prior to CSM application.

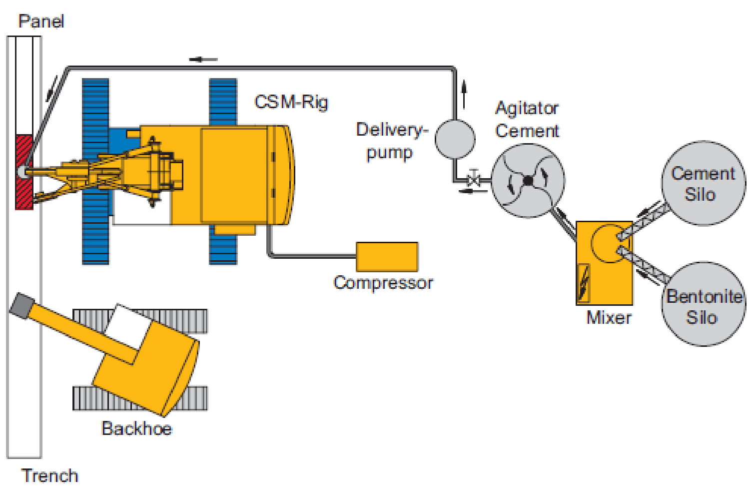
Typical pre-drilling equipment (Rennersdorf, Germany)

Typical back-filling equipment (Rennersdorf, Germany)

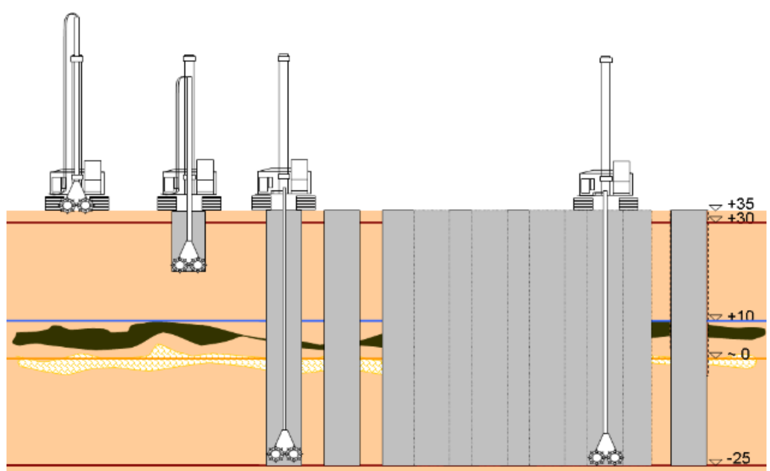
Figure 6-8. Pre-drilling and backfilling.

30. Construct the plastic concrete cut-off wall through the backfilled material using the CSM construction method.

Typical CSM Equipment



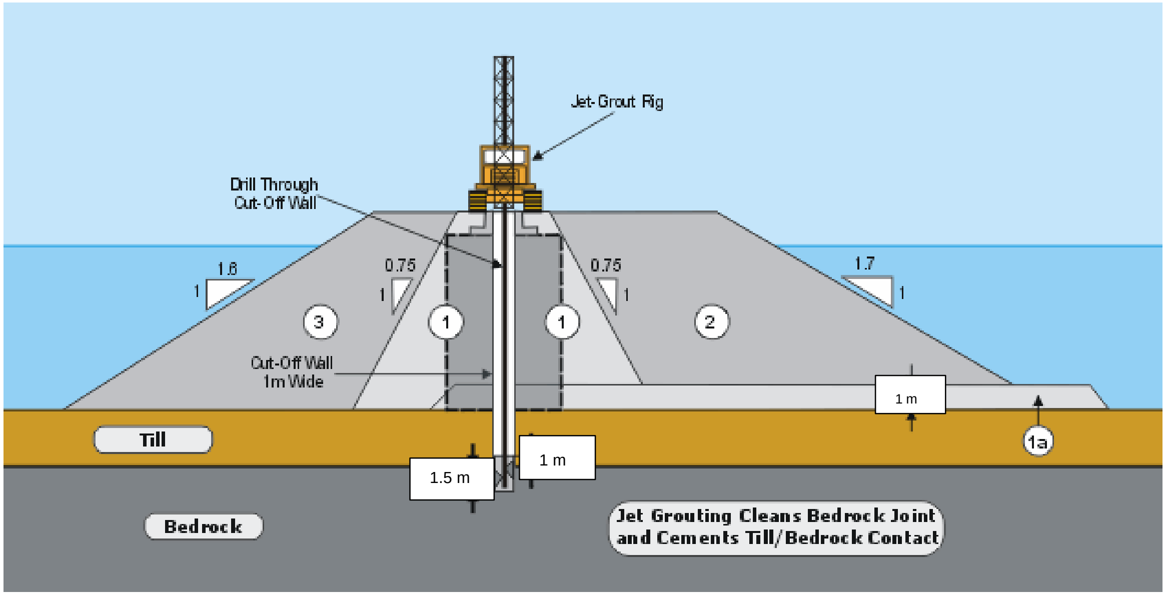
CSM plant schematic



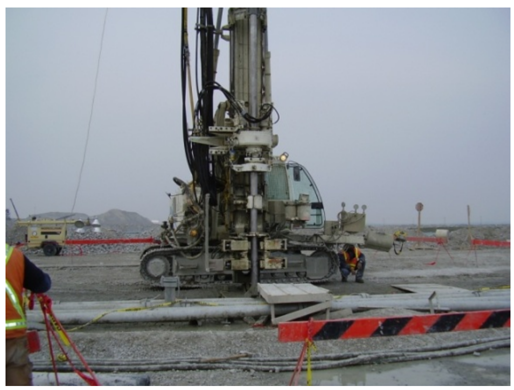
CSM panel sequence schematic

Figure 6-9. CSM equipment and operation.

31. Construct the jet grout component of the cut-off wall. The jet grout portion of the cut-off will be constructed by drilling through the wall, keying into the CSM wall base above (minimum 1 m overlap with the wall), and 1.5 m into bedrock, overlapping with the already-constructed bedrock grout curtain (see Drawings 14300-41D2-1019 and 14300-41D2-1020.3).



Schematic section of jet grouting operation.



Jet grout rig.

Jet grouting rig working inside winterized shelter.

Batching grout.

Figure 6-10. Jet grouting.

32. Installation and activation of the thermosyphon banks (see Drawings 14300-41D2 - 1015, 1017 and 1018). Activation of the chilling units to advance permafrost into adjacent cut-off wall prior to beginning initial dewatering in late 2017.

Installation of compressor unit.


Insulation in vicinity of thermosyphon groups


Thermosyphon radiators and compressor unit


Metal bracing on radiators to stabilize during high winds.

Figure 6-11. Thermosyphon groups.

33. Completion of the dike embankment to final El. 421 m by placing and compacting Zone 2 fill in lifts of 0.5 m thickness and Zone 3 fill in lifts of 1.0 m thicknesses. Construction of the road topping (Zone 1B material) at the crest to complete the embankment.
34. Installation of relief wells (see Drawing 14300-41D2-1029) for relief well design and typical section).
35. Installation of dike instrumentation installation from dike crest to allow for monitoring of dike and foundation pore pressures during pool dewatering. See Drawings 14300-41D2-1026 through 14300-41D2-1031 for A21 dike instrumentation.

36. Pool dewatering of the A21 pool starting in late 2017.

6.6. 2018 Activities

37. Continuation of pool dewatering.

Initial dewatering - direct discharge to Lac de Gras when water quality requirements satisfied.

Floating pump barges used for initial dewatering.

Figure 6-12. Pool dewatering of A418 (fall, 2006).

38. Construction of the toe drainage trench, toe berm along the downstream of the dike and infield grading as exposed following initial A21 pool dewatering. See Drawings 14300-41D2-1021 through 1024.
39. Installation of instrumentation accessed from the toe berm.
40. Installation of the long term water handling system including DPS.
41. Commence pre-stripping of the A21 pit.

Access road and berm construction for construction of toe berm.

Access road being advanced to support toe berm construction.

Figure 6-13. Access construction for toe berm following dewatering.

 Extension of filter blanket in advance of toe berm construction.	 Compaction of Zone 1A material in the toe drain trench.
 Preparing foundations for DPS station.	 DPS station construction in progress.

Figure 6-14. Construction of toe drain and permanent dewatering system.

7.0 A21 POOL DEWATERING PLAN

7.1. General

Once the A21 dike is complete and relevant instrumentation has been installed, projected to be in October 2017, then the portion of Lac de Gras enclosed within the dike will be dewatered. Dewatering is projected to commence in the winter, therefore a layer of ice will be present on the A21 pool and Lac de Gras.

Water will be pumped directly from the A21 pool to Lac de Gras as long as water quality does not exceed the discharge limits defined in the Type A water license issued by the Mackenzie Valley Land and Water Board (License Number N7L2-1645). When the A21 pit water is no longer in compliance with the water license discharge requirements, it will be pumped to on-land storage and treatment facilities at the North Inlet Pond. The water will be held in the North Inlet Pond until it is treated in the Water Treatment Plant (NIWTP) to meet discharge water quality criteria and then released into Lac de Gras.

The activities for the A21 pool dewatering have been planned using conservative assumptions regarding the pool water quality based on experience at the A154 and A418 dikes. The baseline scenario used for planning assumes that 50% of the pool volume will meet discharge criteria and will be pumped directly to Lac de Gras. The remaining 50% of the pool volume has been assumed to not meet discharge criteria, and will be pumped to either Pond 3 or directly to the North Inlet Pond for storage and subsequent treatment prior to discharge to Lac de Gras. If operations permit, the water may be pumped directly to the NIWTP.

This baseline scenario is considered conservative for the following reasons:

- Care will be exercised throughout construction activities to avoid disturbing the water quality.
- Approximately 74% of the A418 pool was able to be discharged directly to Lac de Gras before water quality criteria dictated pumping to on-land storage.
- The lessons learned from the favourable experience with the A418 dewatering will be applied in the detailed planning and execution of the A21 pool dewatering.
- Given that the dewatering will be undertaken in the winter, the ice cover will shield the water surface from wind, and therefore reduce wave action which would otherwise generate turbidity along the shorelines as the water level drops and fine sediments are exposed.

The pool dewatering plan, and contingency measures incorporated within it, will need to interface and be consistent with overall site water management facilities, capabilities, and plans.

7.2. A21 Pool Volume and Storage-Elevation Curve

Table 7-1 presents the volumes for the A154, A418 and A21 dike pools. This table also indicates the portion of the pool volumes that was directly discharged, without treatment for suspended solids, to Lac de Gras for the A154 and A418 dewatering.

Table 7-1. Dike pool summary.

Dike Pool	Full Pool Volume (Mm^3)	Direct Discharge to Lac de Gras	
		Volume (Mm^3)	%
A154	10.1	5.3	52
A418	3.0	2.2	74
A21	6.2	n/a	n/a

The elevation versus storage volume curve for the A21 pool is given in Figure 7-1. The volume of pool water based on bathymetric data is estimated at approximately 6.1 Mm^3. Allowance for precipitation, entrapped water within the downstream rock fill of the A21 dike, potential seepage through the A21 dike during pumping and additional volume created by previous dredging work to remove sediment from the A21 pit area, have also been added to the estimated volume of pool water to define the total water to be pumped. The total estimated water within the A21 pool to be removed during the dewatering is 6.7 Mm^3. The breakdown is as given in Table 7-2.

Table 7-2. A21 pool dewatering volumes.

Water Source	Estimated Volume (Mm^3)
A21 pool	6.11
Precipitation	0.22
Entrapped water	0.13
Seepage water	0.2
Sediments removal	0.06
Total	***6.7***

7.3. A154 and A418 Dewatering Experiences

The dewatering plan for the A21 pool is based largely on the successful experiences from A154 and A418 dikes, both of which were dewatered in the fall, prior to the onset of winter and lake freeze-up. It is useful to summarize those experiences and the lessons learned, despite the planned winter pool dewatering for A21 representing very different conditions.

7.3.1. A154 Dewatering

The A154 dike pool was the first of the Diavik dike pools dewatered. This dewatering involved the removal of 10.1 Mm^3 of water. The A154 dewatering plan conservatively assumed that 100% of the dike pool water would have to be handled on-land i.e. no direct discharge to Lac de Gras. The on-land facilities identified to store A154 water were the PKC facility, the North Inlet storage facility and the sedimentation and clarification ponds located in the area of the North Waste Rock Pile. Dewatering of the A154 dike pool commenced in the beginning of August 2002 and was essentially completed before the onset of winter (end of October 2002). As noted in Table 7-1, 5.3 Mm^3 of water (52%) was pumped directly to Lac de Gras; the remaining 4.8 Mm^3 of water unsuitable for direct discharge to Lac de Gras was pumped to the sedimentation and clarification ponds.

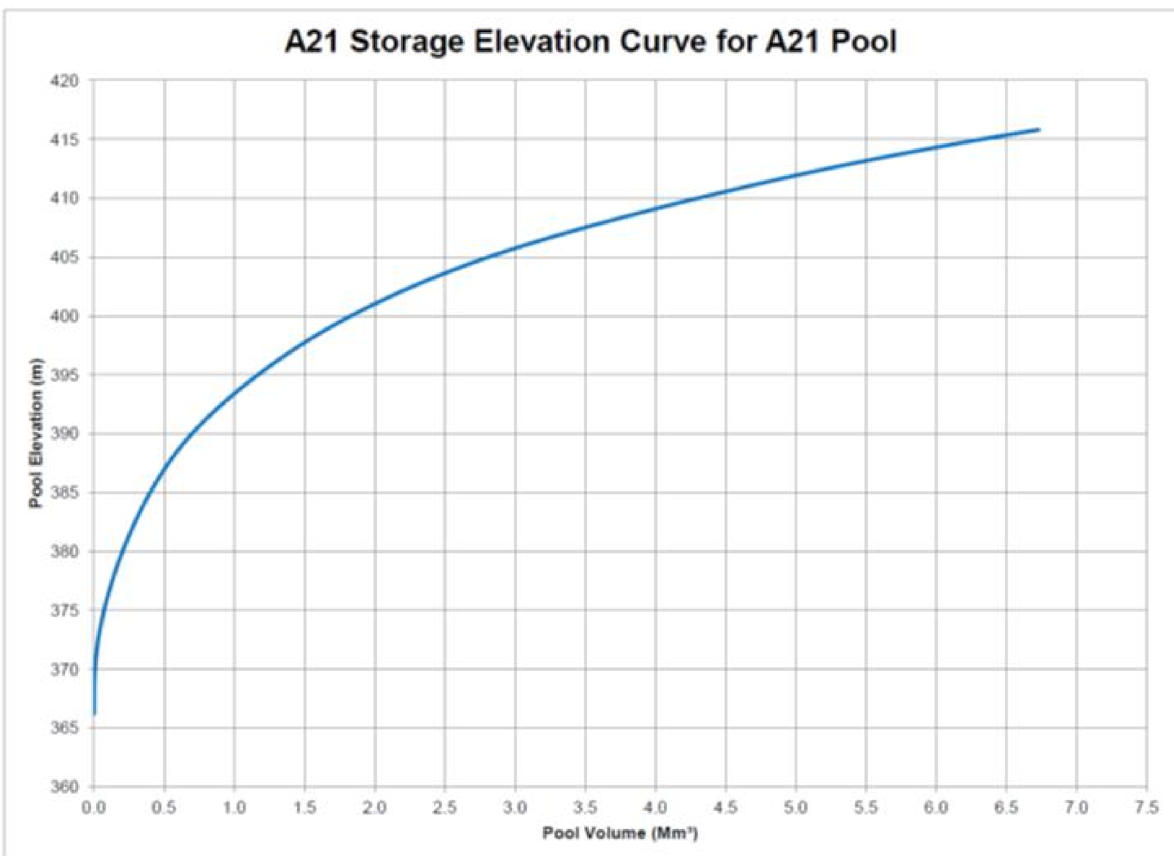


Figure 7-1. Storage elevation curve for A21 pool.

7.3.2. A418 Dewatering

Dewatering of the A418 pool was completed from late summer to early fall 2006. The total volume of the A418 pool was approximately 3 Mm^3. The A418 dewatering plan, similar to the A154 plan, conservatively assumed that all A418 pool water would require storage and or treatment through on-land facilities. The on-land facilities identified to store A418 water were the PKC facility, the North Inlet storage facility and the sedimentation and clarification ponds. As noted in Table 7-1, 2.2 Mm^3 of water (74%) was pumped directly to Lac de Gras; the remaining 0.8 Mm^3 of water unsuitable for direct discharge to Lac de Gras was pumped to the sedimentation and clarification ponds. Drawdown rates of up to 0.8 m/day were permitted on the basis of favourable piezometer readings during the drawdown period.

The sedimentation and clarification ponds will no longer be available for A21 pool dewatering, nor will the PKC facility, with the focus of developing and maintaining wide above-water beaches between the reclaim water pond and the perimeter dams. However, Pond 3 (see Drawing 14300-41D2-1002) is available, DDMI indicates it to have ample capacity, and it is therefore planned for use for A21's dredging and pool dewatering phases.

7.3.3. Lessons Learned

Lessons learned from the A154 and A418 experiences are as follows:

- It is reasonable to plan on discharge of 50% of the pool volume directly to Lac de Gras without treatment.
- The dewatering can be undertaken more efficiently if the allowable rate of drawdown is based on monitored piezometric response in the dike (as was the case for A418) and its foundation rather than a pre-set criterion based on assumed piezometric response (as was the case for A154) which out of necessity was conservative.
- Turbidity that shuts down direct discharge (once suspended solids levels reach 15 mg/litre) is generated, as the pool is lowered, largely due to wave erosion of exposed sediments on shorelines. For A418, the direct discharge phase was extended due to good weather conditions that included a lack of precipitation (which also can mobilize exposed sediments).

It is to the benefit of the A21 project to capitalize on these lessons, particularly the favourable A418 experience, to maximize the proportion of pool water that can be directly discharged. For A21, DDMI anticipates that the ice cover during winter dewatering will reduce wave action and allow for more direct discharge of the pool into Lac de Gras.

7.4. A21 Pool Dewatering Plan

It is anticipated that flexi-float barges will be used to dewater the A21 pool (see Drawing 14300-41D2-1032.2). Since dewatering will take place over the winter months, precautions will be necessary as to barge placement such that they are not adversely affected by ice. Measures to mitigate ice collapse as the A21 pool is drawn down will be required. Similarly, measures to protect walkways and pipelines need to be accounted for. Discharge into Pond 3 and to the North Inlet Pond will also require thermal protection measures to prevent ice-build up in the storage areas which would reduce holding volume. Systems will be required to prevent freeze up.

The discharge pipe lines will direct the pumped water to either Lac de Gras or to Pond 3/NI. The discharge line into Lac de Gras will be directly below the ice surface. The end of the discharge lines into both Lac de Gras and the containment facilities will be of sufficient length and positioned to avoid erosion of the dike fills. It is unknown whether heat-tracing will be required for any of the piping for direct discharge. Heat-tracing will be in place for the pipeline to Pond 3/NI.

The following construction activities are planned in advance of the A21 pool dewatering:

- Draw down of the North Inlet Pond water level (2017) in preparation for A21 pool dewatering
- Complete installation of A21 dike monitoring instrumentation (2017)
- Commission, assemble and install barges, pumps and transfer pipelines (2017)
- Discharge destination:
  - Water meeting environmental requirements: to Lac de Gras
  - Water not meeting environmental requirements: to North Inlet Pond (via Pond 3), and/or directly to North Inlet Water Treatment Plant.

The dewatering of the A21 pool is planned to proceed in the following sequence:

- A21 pool water quality will be tested for all parameters as required by the water license, to ensure that initial volumes to be pumped will meet discharge criteria
- Pumping of the A21 pool to Lac de Gras will commence
- Ongoing testing will be conducted as required in the water license to monitor water quality during pumping
- When testing of pool water shows water quality has reached the maximum discharge criteria, discharge from the pumps will be re-routed from Lac des Gras to Pond 3/NI
- The remaining pool water will pumped to the North Inlet Pond (see Drawing 14300-41D2-1032.1) and subsequently treated by the North Inlet Water Treatment Plant
- The total amount of water that can be managed this way is a function of the active storage volume in the North Inlet Pond, the treatment plant capacity available for treatment of A21 water (in addition to other site demands on the treatment plant) and the rate at which the turbid pool water is to be pumped
- As the pool is drawn down the pumping rate will be determined by the pore water pressure response within and below the dike as identified by the instrumentation, most specifically the piezometers installed within the dike foundation. Threshold levels in terms of monitored pore pressures, established in advance on the basis of stability analyses of the dike, will be established relative to draw down of the pool such that the rate of drawdown can be increased, decreased, or temporarily shut down as dictated by the Dike Engineer.
- If there is insufficient time or capacity to route all of the turbid water to the North Inlet Pond then the next option will be to reduce the rate of dewatering to one that can be tolerated given the site water management system constraints.

Based on the A154 and A418 experience, and conservatively neglecting the more favourable conditions for prolonged direct discharge, it is assumed that 50 percent of the pool water will be pumped directly over the dike to Lac de Gras. The balance of the pool water will be pumped to on-land storage facilities. The assumed distribution of the pool water is as follows:

Direct to Lac de Gras	3.35 Mm^3 = (50%)
NI/NIWTP	3.35 Mm^3 = (50%)
TOTAL	6.7 Mm^3

7.5. A21 Pool Dewatering Schedule

The maximum pump-out rate for the A21 pool is governed by the drawdown rate established by on the basis of assumed lakebed till pore pressure equilibration rates (NKSL, 1999). The initial rate has been set at 0.4 m/day. This rate is a target to allow for safe pore water pressure dissipation in the dike's foundation. Piezometers will be installed in the A21 dike to monitor actual pore pressure dissipation, and the drawdown rate will be adjusted in the field accordingly. The A418 experience indicated that more rapid drawdown rates (up to 0.8 m/day) were safely achieved on the basis of monitored instruments.

Based on a planned conservative 0.4 m/day drawdown rate, and a more optimistic 0.8 m drawdown rate, the maximum pump out rates would occur at the onset of initial dewatering. Due to the shape of the elevation versus storage volume curve, the daily volume of pump-out would thereafter decrease.

The pumps will be designed to operate at or near full capacity for the first several days, when the water in the pool is deep. The initial maximum drawdown rate will be set at 0.4 m. Depending on piezometer response, the drawdown rate may be increased as per the A418 experience. The pumps will be progressively throttled back to lower rates as the pool gets shallower to maintain the safe draw down rate as determined by the dike designers on the basis of continual evaluation of the piezometer data. The range of pump-out rates, based on the 0.4 m/day and 0.8 m/day drawdown are given in Figure 7-2 below.

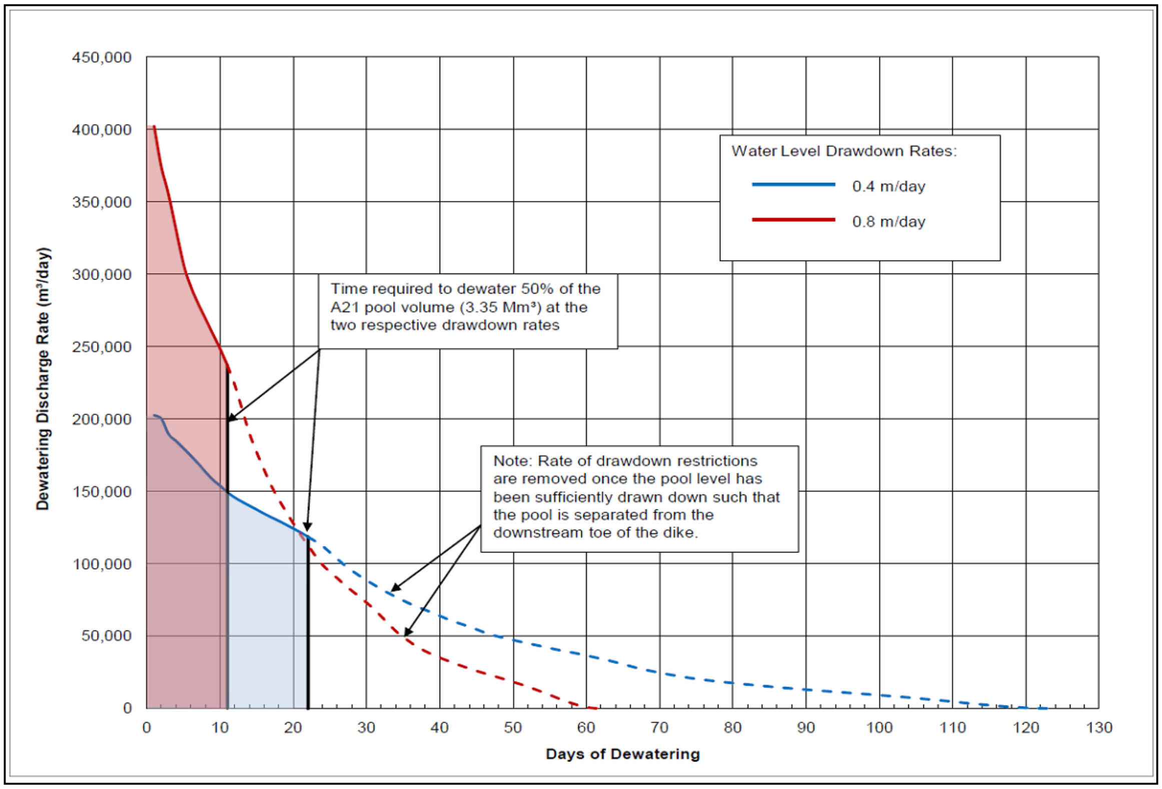


Figure 7-2. Range of pump-out rates during direct discharge phase to Lac de Gras.

The dewatering schedule for the A21 pool would be approximately 120 days based on a drawdown rate of the water's surface elevation of 0.4 m/day. The direct discharge phase (assuming 50% direct discharge) would be completed (at a drawdown rate limitation of 0.4 m/day) in about 22 days.

7.6. Dewatering Risks and Contingencies

Risks in terms of the A21 pool dewatering are outlined in Table 7-3.

Table 7-3. Dewatering risks and contingencies.

Risk	Description	Mitigating Factors and Contingencies
Unable to achieve 50% of pool dewatering via direct discharge to Lac de Gras	Suspended solids levels reach 15 mg/litre prior to removal of 3.35 Mm^3.	1. Winter dewatering expected to be more advantageous for prolonged direct discharge phase. 2. Provide for added on-land storage capacity, possibly including within the PKC facility. 3. Extend period of dewatering. 4. Consider baffles/curtains within the A21 pool to maximize likelihood of extending period of direct discharge.
Inadequate on-land storage capacity relative to base case assumptions due to other site water management constraints.	Dewatering following completion of direct discharge phase becomes constrained by capacity of WTP.	Same as 1, 2 and 3 above.
Slides in lakebed sediments/till in response to dewatering.	Slides in relatively steep areas of the lakebed, particularly in Deep Blue area generate significant turbidity that causes premature shutdown of direct discharge of water to Lac de Gras.	1. Limit rate of drawdown. 2. Be prepared to discharge to North Inlet via Pond 3.
Pumping or piping issues related to winter dewatering	Freeze-up of pumping equipment, pipes, or inlet/outlet	1. Hoarding on pumps. 2. Utilize “bubbler” or alternate method to keep water ice-free at barge/inlet area and North Inlet Pond. 3. Heat trace pipes.

8.0 DESIGN ANALYSES

8.1. General

The following sections summarize the geotechnical and thermal analyses that have been carried out in support of the Diavik dikes. The analyses undertaken to date have included the following.

- Two-dimensional limit equilibrium stability analyses have been carried to evaluate the slope stability of the dikes during construction, pool dewatering, and long term, steady state loading conditions.
- Two-dimensional finite element seepage analyses were carried to guide cut-off wall design, estimate seepage rates for a range of dike and foundation conditions, and evaluate seepage gradients and internal erosion potential.
- Stress-deformation analyses were undertaken to evaluate dike deformation in response to pool dewatering, and deformations associated to pit wall dilation (for the A21 dike only, given the reduced setback relative to the A154 and A418 dikes). The focus of these analyses was the stresses and strains imposed on the cut-off wall, which in turn guided the design parameters for the cut-off, specifically the plastic concrete component.
- Wave run-up and wind set up analyses were carried out to evaluate the freeboard required above the MDWL.
- Thermal analyses to evaluate thermal behavior of the dike, and to guide thermal design measures required in shallow water areas and at the dike abutments.

The sections that follow also summarize key monitoring data and trends from the A154 and A418 dikes, which provide a means of calibrating predictive modeling and better understanding the behavior of the A21 dike.

8.2. Stability Analyses

Stability analyses of the A21 dikes have been conducted previously by NKSL (1999, 2004) and AMEC (2007). Additional stability analyses have been undertaken in support of this updated design report for A21, and are summarized in Section 8.2.5, and presented in Appendix F.

The minimum factor of safety required for various phases of dike construction and operation, and the associated loading conditions, are given in Table 8-1.

Table 8-1. Minimum required factors of safety for the stages of dike construction and operation.

Phase	Loading Condition	Required Minimum Factor of Safety (FoS)
Phase 1 - embankment construction	Short-term Static Loading (e.g., end of construction)	1.3
Phase 2 - dewatering		
Phase 3 - steady state	Steady-State Long-term Static Loading	1.5
Phase 4 - earthquake loading	Seismic loading (pseudo-static analysis)	1.1*

* CDA (2007) guidelines specify a minimum allowable factor of safety of 1.0 for pseudo-static analysis.

The designs of the A154 and A418 dikes, and the proposed A21 dike, are essentially identical from the standpoint of slope stability. The dikes are also founded on essentially the same lakebed soils - a thin zone of lakebed sediments unable to be removed via dredging, underlain by glacial till. Only the combination of dike height and lakebed bathymetry distinguishes the dikes from the slope stability standpoint. It is therefore useful to review the previous analyses carried out for the various dikes to characterize dike stability issues and conditions.

8.2.1. NKSL (1999) Analyses for A154 Dike

NKSL (1999) undertook two-dimensional limit equilibrium slope stability analyses in support of the design of the A154 dike. Those analyses considered the maximum height section of that dike, on essentially flat lakebed bathymetry. The analyses for that single cross-section, representing a lake depth of 25 m, considered stability for the following loading conditions:

- Short term conditions for the first stage of embankment placement, to crest El. 417 m. These analyses assumed complete removal of the lakebed sediments, and excess construction induced pore pressures in the lakebed till corresponding to 36%[3] of the imposed embankment loading (change in total stress, considering buoyant weight of rockfill below lake level, and total unit weight above lake level). That analysis considered stability before and after vibro-densification of the Zone 1 core, and yielded factors of safety against upstream and downstream failure of 1.36 and 1.35 respectively, irrespective of the before and after vibro-densification cases.

[3] The excess pore pressure ratio (typically referred to as the $\bar{B}$ parameter) of 0.36 was based on isotropically-consolidated triaxial compression (CIU) shear strength tests with pore pressure measurements, on remolded lakebed till samples (NKSL, 1999). To approximate this result, the excess pore pressure in the foundation till was calculated as 36% of the change in vertical stress due to the weight (submerged or total as appropriate) of the embankment fill over any given point within the lakebed till foundation.

- Short term analysis for the 2nd phase of embankment construction after the dike was raised from El. 417 m to El. 421 m. Construction-induced excess pore pressures were again considered, but corresponded to 36% of the applied loading only for the dike raise from El. 417 m to 421 m. The implicit assumption in this analysis was full dissipation of excess pore pressures from the first stage of embankment construction to El. 417 m. Factors of safety against upstream and downstream failure were 1.49 and 1.59 respectively.
- Short term analyses for downstream slope stability during pool dewatering assumed full dissipation of embankment construction-induced pore pressures, but excess pore pressures in the lakebed till on the basis of pool draw-down occurring more rapidly than equilibration of pore pressures into the lakebed till to the drawn-down lake level. A pore pressure ratio (R_u, defined as ratio of pore pressure to total vertical stress) of 0.5 was assumed in these analyses. These analyses yielded the following factors of safety:
    - 50% pool dewatering stage - factor of safety = 1.55
    - 100% pool dewatering - factor of safety = 1.3.
- The steady state case for upstream and downstream stability yielded factors of safety of 1.51 and 1.59 respectively. The downstream stability included consideration of an ice thrust on the crest of 150 kN/m, although this was inconsequential as the critical slip surfaces for downstream stability did not extend upstream of the cut-off wall.
- Seismic stability was evaluated for the steady state case via pseudo-static analysis, with the seismic coefficient conservatively assumed equal to the MDE PGA of 0.023 g. Upstream and downstream slope factors of safety of 1.42 and 1.51 were obtained.

8.2.2. NKSL (2004) Analyses for A418 Dike

The NKSL (2004) stability analyses in support of the A418 design were essentially the same as those carried out by NKSL (1999) for the A154 dike. The material parameters, and the loading stages considered, were the same as described in Section 8.2.1. The A418 stability analyses were different from the A154 analyses in the following respects:

- The dike section analyzed was for a lake depth of about 30 m, and the slightly upstream sloping lakebed bathymetry at that section was accounted for, as opposed to the flat bathymetric profile used in the A154 dike section stability analyses.
- The lower horizon of firmer sediments (referred to by NKSL as “sensitive till”) was included within the model, above the lakebed till. This unit, typically no more than 0.5 m in thickness, was assigned by NKSL an effective cohesion (c’) of 0, and an effective friction angle (ϕ’) of 30°. The inclusion of this zone in the foundation was in recognition of the inability of the dredging to fully remove the lower horizon of firmer lakebed sediments for the A154 dike construction (see discussion in Sections 2.5.2 and 4.2.2).

As described above for the A154 dike analyses, all of the loading conditions and stages considered yielded acceptable factors of safety. The slightly weaker strength (ϕ' = 30°, as compared to 35° for the lakebed till) assigned to the lower horizon of firmer sediments assumed to remain after dredging did not affect stability given the minimal thickness of this unit, and the very gentle lakebed slopes for the section analyzed.

For the short-term cases with construction-induced pore pressures within the residual lakebed sediments (sensitive till as designated by NKSL) and the underlying lakebed till, the same excess pore pressure ratio ($\bar{B}$) of 0.36, as established by laboratory testing in support of the A154 dike design, was used. Actual foundation pore pressure response to application of dike embankment loading could not be measured as the piezometers could not be installed in the foundation until after the dike was constructed.

8.2.3. AMEC (2007) Analyses for A21 Dike

AMEC (2007) undertook limit equilibrium stability analyses for four two-dimensional sections along the 2007 A21 dike alignment identified as being both critical and representative. The AMEC (2007) analyses are provided in Appendix E.

The A21 dike is similar in geometry to the A154 and A418 dikes, therefore the stability for A21, as expressed by a limit equilibrium factor of safety, would be expected to be similar to that of the existing dikes. Nonetheless, A21-specific stability analyses were undertaken for the following reasons:

- In general, the lakebed till thickness along much of the A21 dike alignment appears greater for the highest section of the A21 dike than for the previous two dikes.
- Given the inability to fully remove the lakebed sediments by dredging, as experienced for both the A154 and A418 dikes, the effect of the lower, firmer horizon of sediment, not removable by dredging, on the dike stability required assessment.
- The A21 dike was aligned to stay within shallow water to the extent possible given the constraints imposed by the open pit shell and setback between the pit rim and the downstream toe of the dike. For much of the dike alignment, this means that the lakebed falls off to either side of the dike at grades sufficiently steep to potentially affect the overall embankment stability. This is particularly the case in the northeast portion of the dike, between about Sta. 0+600 m to 0+800 m.

The material parameters used by AMEC (2007) were the same as those used by NKSL (1999, 2004), with the exception of the residual lakebed sediments assumed to remain after dredging. For those sediments, NKSL assigned ϕ' = 30°, while AMEC (2007) judged ϕ' = 32° (10% lower shear strength than assigned the lakebed till) to be appropriate, on the basis of the laboratory test data (see Section 2.5.1) and observations of this material during the pre-stripping of the A418 open pit.

The AMEC (2007) analyses considered the same phases as the previous dikes, as listed in Table 3-1 and 8-1. The stability analysis results confirmed the critical periods in terms of the stability of the dike would be during Phases 1 (embankment construction) and 2 (pool dewatering). Dike stability during Phase 2 can be controlled via monitoring of foundation piezometers, setting threshold levels for the piezometers based on stability analyses, and adjusting the rate of pool dewatering as required to maintain a minimum factor of safety of 1.3. No such control is available during embankment construction, however, which can only be guided by stability analyses, the assumed $\bar{B}$ = 0.36 response of the foundation soils to rapid load application, and the successful precedent of stable conditions established by the construction of the A154 and A418 dikes. However, areas of adversely sloping lakebed bathymetry along the A21 dike alignment go beyond the precedent established by the previous two dikes, and therefore warranted specific analysis.

The AMEC (2007) analyses evaluated short-term construction stability for a range of $\bar{B}$ values, rather than just the single value of 0.36 considered in the NKSL (1999, 2004) analyses. The A21 analysis results indicate that, for the adverse lakebed geometries between Sta. 0+600 m and 0+800 m, where the lakebed slopes away from the dike, upstream stability would be marginal unless very low excess pore pressures existed within the foundation soils, lower than would be the case for the assumed $\bar{B}$ = 0.36 used within the NKSL (1999, 2004) analyses. Specifically, to obtain the minimum required factor of safety of 1.3 for that area of the dike, the $\bar{B}$ values would have to be no higher than:

- 0.04 for a slip surface following the residual lakebed sediments not removed via dredging
- 0.15 for a slip surface through the underlying lakebed till.

The mitigation for this condition is clamshell placement of a Zone 3 buttress, below lake level, in the toe area (i.e. the bathymetric low) along this portion of the dike, in advance of placement of the main dike embankment, to achieve an acceptable factor of safety (1.3) assuming $\bar{B}$ = 0.36.

A similar situation was indicated by the analyses for the dike section at about Sta. 0+300 m. For this section the AMEC end-of-construction analyses indicated that, to obtain the minimum required factor of safety of 1.3, the $\bar{B}$ values would have to be no higher than:

- 0.26 for slip surface within the residual sediments not removed via dredging
- 0.38 for slip surface within the underlying lakebed till.

Monitoring of the foundation pore pressure response to the loading imposed during the A154 and A418 dikes' construction could not be carried out, making comparisons between the $\bar{B}$ values back-analyzed from the A21 analyses (to obtain the minimum required factor of safety of 1.3) and that assumed by NKSL (1999, 2004) on the basis of laboratory testing on remolded till samples a largely theoretical exercise. If actual pore pressure ratios were significantly higher, the actual factors of safety for the previous dikes' construction could have been well below 1.3 (but still greater than 1 given the lack of substantial deformation noted during embankment advance). However, the analyses are useful in indicating to what degree the A21 dike end-of-construction condition as expressed by factor of safety could be beyond established and successful precedent for the previous two dikes, and for guiding the design of mitigation measures, which are discussed further in Section 8.2.5.

Prior to the dewatering phase, piezometers are to be installed within the foundation soils below the proposed dike. Stability analyses specific to each instrumentation section will be carried out in advance of pool dewatering, to establish threshold levels for lakebed till pore pressure equilibration levels (relative to the pool draw-down stage) corresponding to an acceptable condition (factor of safety > 1.3). Those threshold levels will be used to monitor and control the pool draw-down rate, as was done for the A418 dike pool dewatering.

8.2.4. Implications of the 2012 Dike Alignment Shift

As discussed in Section 1.2.4, in 2012 the A21 dike was modified slightly relative to the alignment that was the basis of the AMEC (2007) stability analyses. The difference between the two alignments is shown on Figure 1-4. The main areas of alignment shift from 2007 to 2012 of significance, from the perspective of slope stability, are as follows.

- Along the southeast, between about Sta. 1+000 m and 1+200 m, the dike alignment was shifted further out into the lake, into slightly deeper water, but still within an area of favorable (i.e. minimal slope) lakebed bathymetry.
- Along the northeast perimeter, between about Sta. 0+650 m to 0+900 m, the alignment was shifted outwards, in an area of already adverse lakebed bathymetry for upstream slope stability.

8.2.5. A21 Pit Slope Stability Analyses

The A21 pit slope stability analyses undertaken by Golder (2006b, 2012) included consideration of slip surfaces extending as far back from the pit rim as the downstream toe of the A21 dike. These analyses informed pit slope designs, and pit slope depressurization requirements, to achieve a minimum limit equilibrium factor of safety of 1.5 for pit slope failure geometries that could affect the dike. Golder (2012) presents the stability analyses for the current pit design and A21 dike alignment.

8.2.6. Updated Analyses for Current Dike Alignment

To address the stability conditions associated with the 2012 dike alignment shift, and to determine mitigation measures required to achieve acceptable end-of-construction stability in areas of adverse lakebed bathymetry, BGC undertook stability analyses for the A21 dike sections listed in Table 8-2.

Table 8-2. Sections analyzed and loading cases considered.

Analysis Section	Reason for Analysis	Loading Case(s) Considered
A - Sta. 0+745 m	Upstream slope stability evaluated due to adverse bathymetry. Determine dimensions of required toe buttress to achieve acceptable end-of-construction stability.	End-of-construction, with $\bar{B}$ = 0.36, for two stages of embankment construction: • Stage 1 - to crest El. 417 m • Stage 2 - to crest El. 421 m. Long term, steady state.
B - Sta. 1+053 m	Upstream slope only evaluated. This section has favorable bathymetry, but represents the maximum (highest) dike section. Bathymetry slopes very slightly away from the pit, so the upstream factor of safety will be lower than for downstream (apart from the pool dewatering phase), so the latter was not analyzed.	End-of-construction, with $\bar{B}$ = 0.36, for two stages of embankment construction: • Stage 1 - to crest El. 417 m • Stage 2 - to crest El. 421 m. Long term, steady state.
C - Sta. 1+500 m	Downstream - the dike section here is low, but this portion of the dike is adjacent to the A5 kimberlite bathymetric low, where stability of the natural lakebed till might affect the dike during pool dewatering.	Pool dewatering, considering a range of pore pressure equilibration rates in the lakebed sediments and till. Long term, steady state conditions.
D - Sta. 0+310 m	Upstream slope stability evaluated due to adverse bathymetry. Determine dimensions of required toe buttress to achieve acceptable end-of-construction stability.	End-of-construction, with $\bar{B}$ = 0.36, for two stages of embankment construction: • Stage 1 - to crest El. 417 m • Stage 2 - to crest El. 421 m. Long term, steady state.

The stability analyses for these sections are presented in Appendix F.

The results of the stability analysis for the four sections presented above are summarized in Table 8-3.

Table 8-3. Stability modeling results summary.

Cross Section	Loading Case	Required FoS	FoS Without Upstream Toe Buttress	FoS With Upstream Toe Buttress
A (0+745)	Steady State	1.5	1.4	1.5
	End of Construction El. 417 m	1.3	1.1	1.3
	End of Construction El. 421 m	1.3	1.2	1.3
B (1+053)	Steady State	1.5	1.5	No buttress required
	End of Construction El. 417 m	1.3	1.4	
	End of Construction El. 421 m	1.3	1.5	
C (1+500)	Pool Dewatering	1.3	See Table 8-5.	
	Steady State	1.5	1.5	No buttress required
D (0+310)	Steady State	1.5	1.2	1.5
	End of Construction El. 417 m	1.3	0.9	1.4
	End of Construction El. 421 m	1.3	1.0	1.4

For the two sections (A and D, at Sta. 0+745 m and 0+310 m respectively) with adversely sloping bathymetry below the upstream slope of the dam, slope flattening is required to achieve the minimum required factor of safety for short term construction conditions, and for long term, steady state conditions. As outlined in Appendix F, it is assumed that this flattening would be achieved by clamshell placement of the toe berms, prior to advance of the main embankment. The buttress dimensions and extents are given in

Table 8-4. The buttress volumes upstream of the Zone 3 1.6H:1V slope design line are also provided. The actual buttress volumes considering the portion within the Zone 3 slope design line would be higher. The stability buttresses constructed in the vicinity of Sta. 0+745 m and 0+310 m should be constructed so that they extend from the toe of the buttress to the reference line of the dike to prevent any stability issues on the natural slopes above the toe region.

Table 8-4. Configurations for upstream toe buttressing.

Buttress Extent		Crest Elev.	Crest Width (m)	Estimated Buttress Volume (m^3) Beyond Zone 3 Limits	Estimated Buttress Volume (m^3) Placed Prior to Dike Advance
From Sta.	To Sta.				
0+285	0+340	405 410	6 5	5,000	7,200
0+550	0+800	400	5.5	7,000	17,000

The results for Section C at Sta. 1+500 m, where the dike height is low but the bathymetry downstream of the dike's downstream toe is adverse, are summarized in Table 8-5 for the pool dewatering phase. The analysis indicates pore pressure equilibration within the lakebed till, relative to the drawn-down pool level, must be near 70% to achieve the minimum required factor of safety of 1.3. Similar analyses will be undertaken for all instrumented dike sections for incorporation into the pool dewatering plan. Threshold piezometer levels will be established for various stages of draw-down to identify safe conditions (factor of safety of at least 1.3) versus unsafe conditions that would require slowdown or even temporary cessation of drawdown.

Table 8-5. Cross Section C dewatering equilibration factor of safety.

Till Equilibration Level (% change over total dewatering)	FoS of Section
0%	0.7
25%	0.7
50%	1.0
75%	1.4
100%	1.5

The stability of the dike at Section B, the highest section of the A21 dike at Sta. 1+053 m, met or exceeded the required factor of safety for all loading cases, consistent with the results for the previous analysis of that portion of the dike conducted by AMEC (2007) for the 2007 alignment. This result indicates that in areas with favourable (i.e. relatively flat) bathymetry, stability is adequate, a result borne out by the experience with the A154 and A418 dikes.

8.2.7. Stability Parallel to Dike Axis during Embankment Advance

Analyses undertaken to date for the Diavik dikes have not considered the stability of the slopes in the direction of embankment advance, where adverse lakebed bathymetry (lakebed sloping in the direction of embankment advance) is present. Besides the potential stability concern, significant lakebed slopes in the direction of embankment advance could also affect the ability to achieve the "theoretical" boundaries between Zones 1, 2 and 3. To date, neither the stability nor the zone boundary problem have occurred for the construction of the A154 and A418 dikes. This warranted a comparison of lakebed slopes along the A21 alignment with those below the A154 and A418 dike alignments, to evaluate if there are portions along the A21 alignment where these slopes are steeper than the precedent established for the previous two dikes. As indicated in Section 8.2.5, there are areas of adverse bathymetry transverse to the axis of the A21 dike that require upstream toe buttressing in two areas.

Figure 8-1 through Figure 8-3 indicate lakebed bathymetry slopes below each of the A154, A418 and A21 dike alignments respectively. Slopes above 40% are shown in red, below 10 % in purple. For the A154 and A418 dike alignments, the figures indicate significant slopes (>40%) paralleling the dike axes, and sloping in the direction of embankment advance, near the shores at Sta. 0+330 m for the A154 dike, and at Sta. 0+940 m for the A418 dike. For the latter case, the steep lakebed slope extended into the deepest water section of either of the A154 and A418 dikes. In both cases, the lakebed slopes are steeper than those indicated on Figure 8-3 for the A21 dike, where the maximum slope along the dike axis is about 30%, except for a portion of the upstream shell of the dike near Sta. 0+550 m, which will be supported by the toe buttress required in that area. The as-built reports for the A154 and A418 dikes (NKSL, 2003 and 2007a, respectively) make no mention of stability issues when constructing the dikes along the relatively steeply sloping lakebed sections. Based on that experience, no measures are proposed for A21 construction over and above the monitoring during embankment advance similar to what was undertaken during the A154 and A417 dikes' construction.

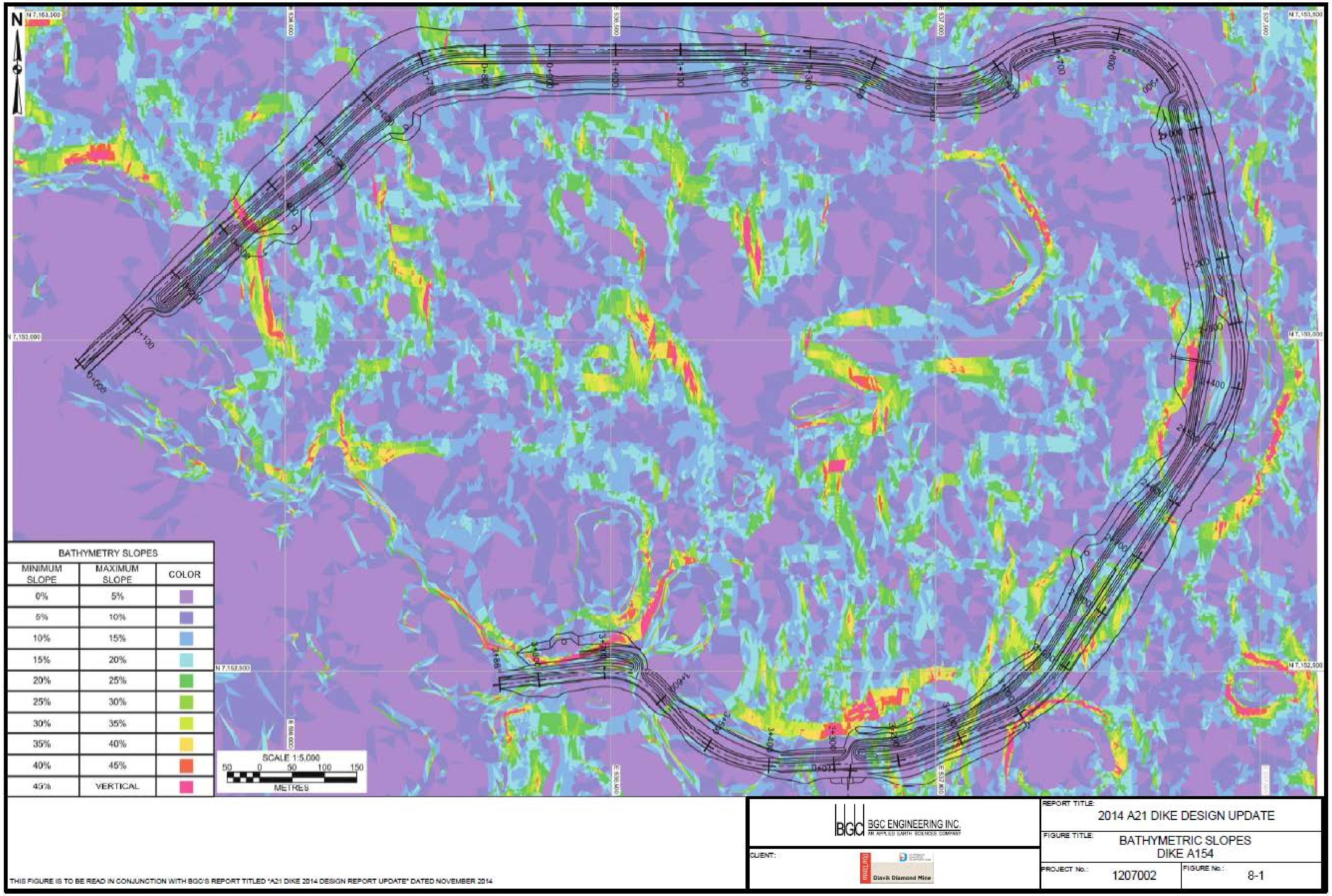


Figure 8-1. Bathymetry slopes dike A154.

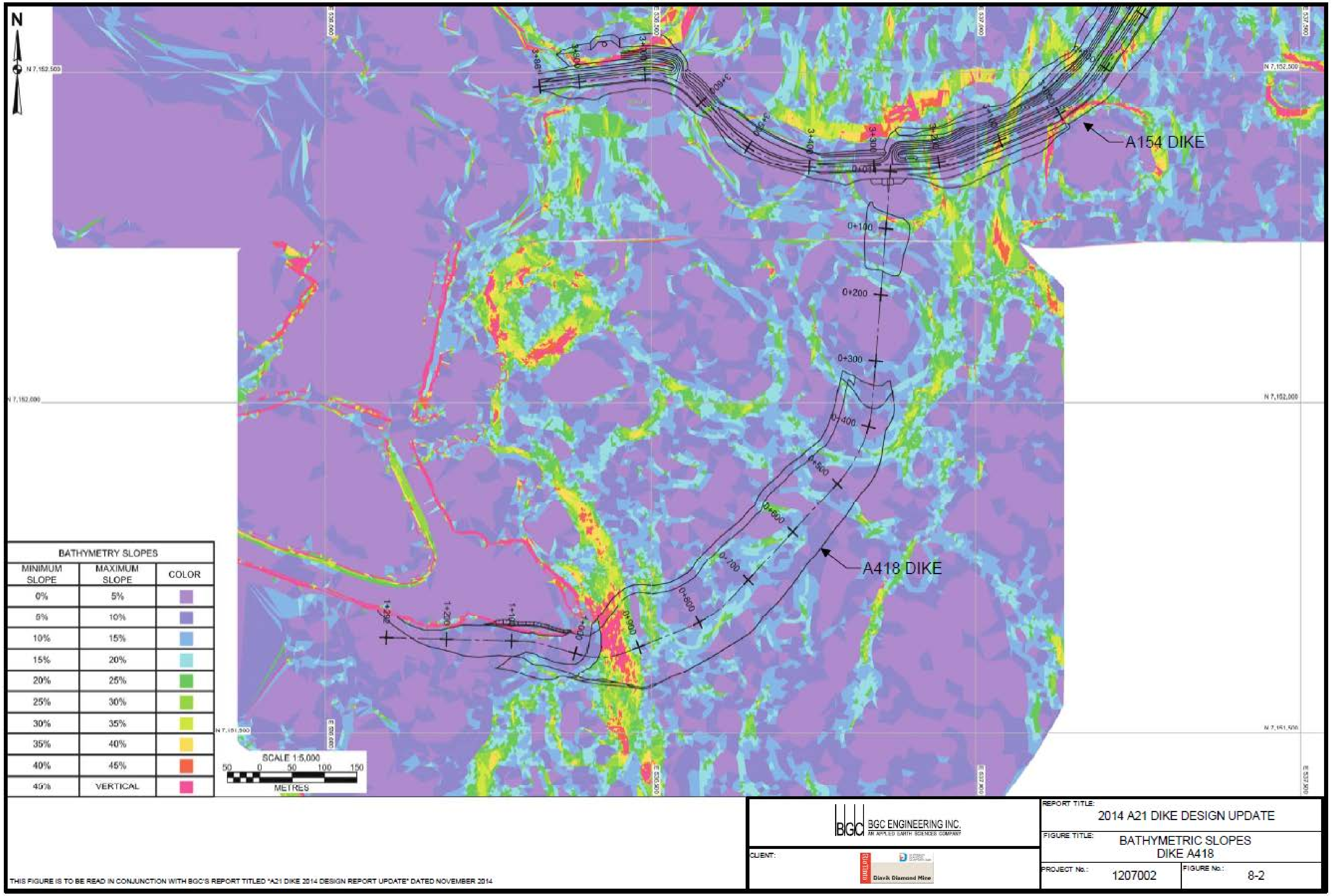


Figure 8-2. Bathymetry slopes dike A418.

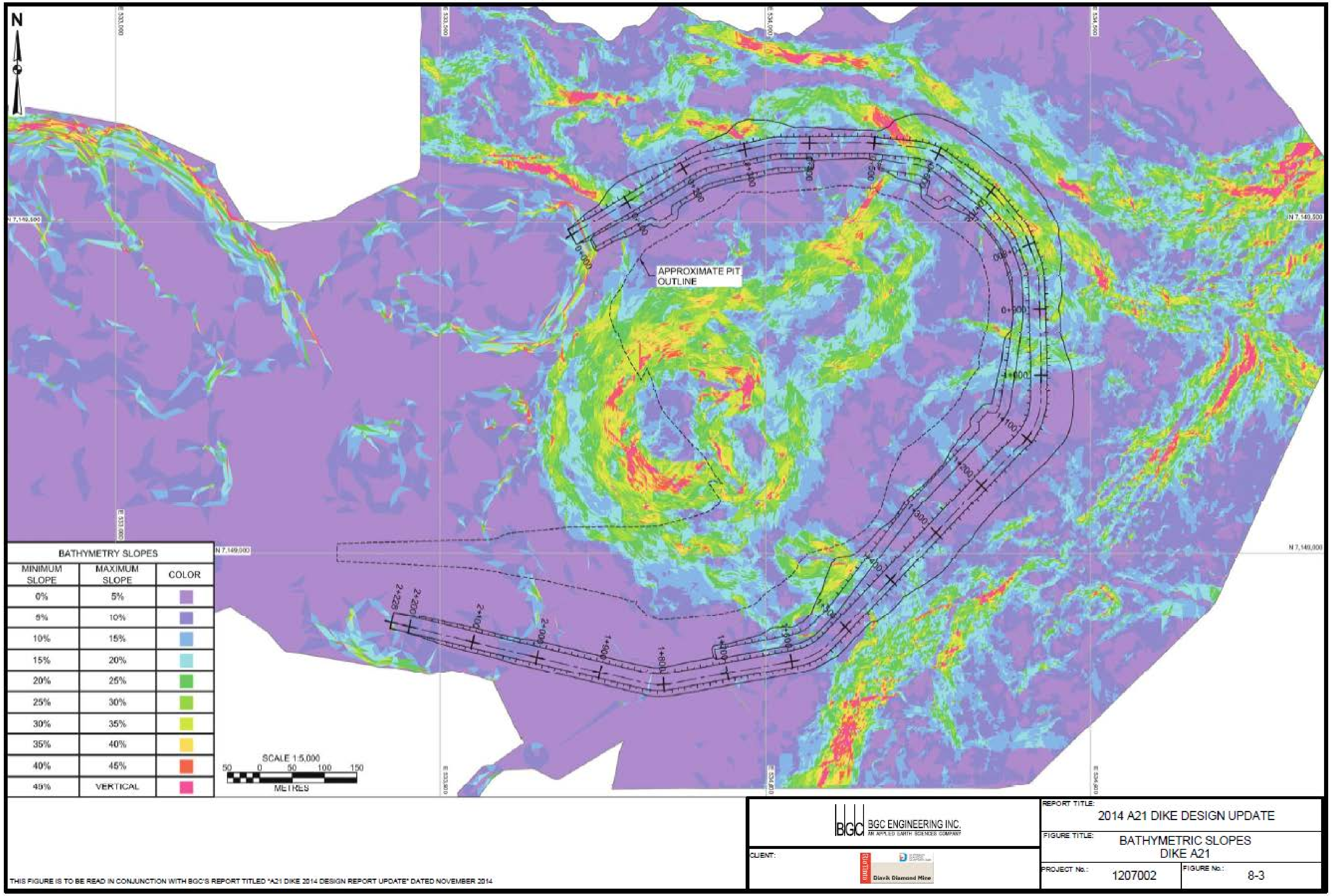


Figure 8-3. Bathymetry slopes dike A21.

8.3. Seepage Analyses

8.3.1. General

This section summarizes the seepage analyses undertaken in support of the A21 dike design. These include analyses undertaken by NKSL (1999) for the A154 and by AMEC (2007) for the 2007 alignment of the A21 dike. Golder (2012) presents three-dimensional groundwater modeling carried out in support of pit slope design, and the design of a pit slope depressurization system for the 2012 alignment of the A21 dike. This section includes a discussion of piezometric trends for the A154 and A418 dikes, which provide the best analogue for the piezometric and seepage performance that can be anticipated for the A21 dike.

8.3.2. NKSL Analyses for A154 Dikes

8.3.2.1. General

NKSL (1999) presents two-dimensional, finite element seepage modeling carried out in support of the A154 dike design. For the A418 dike, NKSL (2004) performed a review of the geological conditions along the A418 dike alignment to select the appropriate grout curtain depth in order to achieve similar control over pressures, gradients and inflow rates. Beyond that review, no additional seepage analyses were performed in support of the A418 design.

8.3.2.2. Seepage Rates

The seepage rate design criteria established by NKSL (1999) for the A154 dike design was a rate not exceeding 1.8 l/min/m of dike. Seepage rates per unit length were calculated for various dike heights in order to estimate the inflow to the pump stations. The analyses also served to demonstrate that the maximum allowable exit gradient (0.3 for till) was not exceeded.

The seepage rates predicted by NKSL, as a function of lakebed elevation (i.e. lake depth) are as shown in Figure 8-4. As noted by NKSL (2004), these predicted seepage rates proved to be conservative. The minimum lakebed elevation along the alignment of the A21 dike cut-off wall is about 395 m (post-dredging). The average lakebed elevation for the in-lake portion of the A21 dike is about 408.5 m

NKSL also modeled the effect of a window in the cut-off wall, both within the lakebed till, and in the Zone 1 core of the dike. The increased seepage rate associated with a cut-off window within the lakebed till at the bedrock interface was, for a two-dimensional analysis section, about 3 to 4 times higher than the unit seepage rate without a window. For a window within the Zone 1 dike fill, however, the unit seepage rate was predicted to increase by nearly three orders of magnitude.

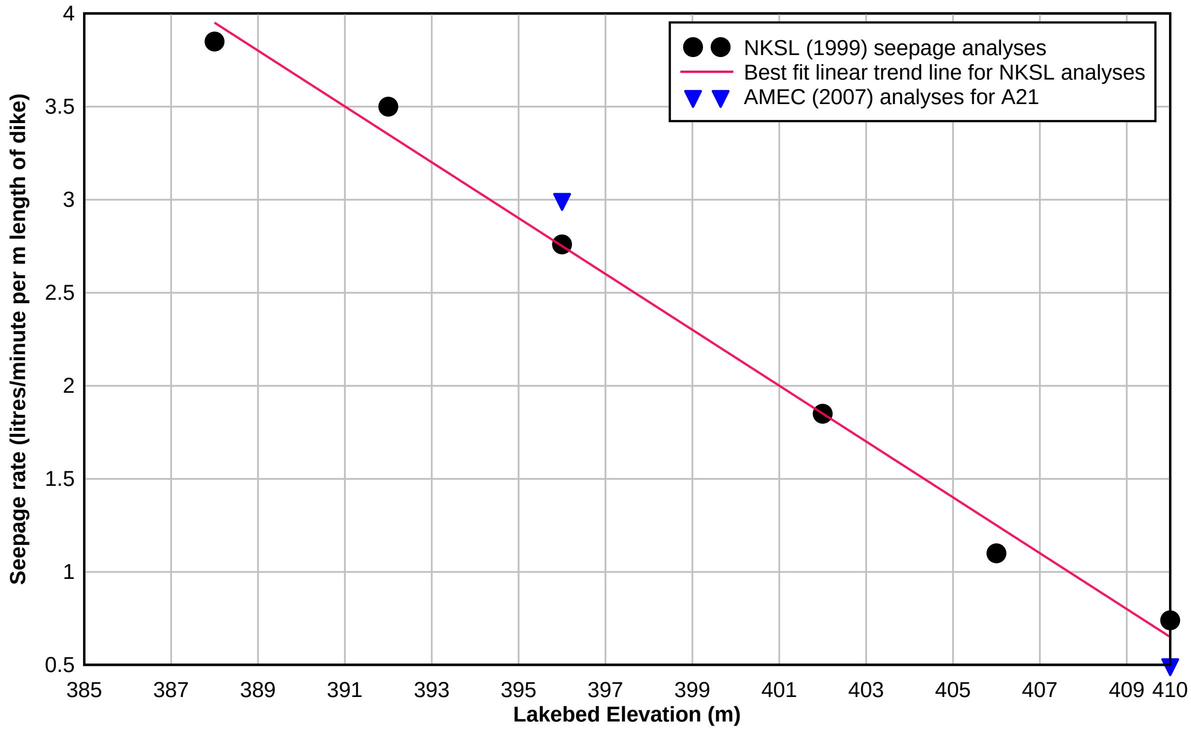


Figure 8-4. Predicted seepage rates as a function of lakebed elevation for A154 dike (NKSL, 1999) and the A21 Dike (AMEC, 2007).

8.3.2.3. Exit Gradients from Lakebed Till

NKSL (1999) specified, as a design criterion considering potential internal erosion, exit gradients near the till surface less than 0.3. This was based on an assumed tolerable exit gradient of 0.9, with a factor of safety of 3. The filter blanket is included within the design as an essential defensive measure against internal erosion, in addition to designing the cut-off to reduce seepage gradients.

The seepage analyses indicated that when the foundation level is less than 402, higher exit gradients could develop if the horizontal hydraulic conductivity of the lakebed till is lower than the assumed value of 2 x 10^{-5} m/sec and/or if the anisotropy ratio was increased to 25 or even 100. Under such conditions, predicted exit gradients of up to 0.47 were obtained. Similarly, higher exit gradients were obtained if the till thickness was reduced.

Based on these results, NKSL (1999) specified relief wells be installed within the lakebed till below the downstream shell of the A154 dike, in areas where the dike foundation elevation is less than 402 m. This same measure was incorporated into the designs of the A418 dike (NKSL, 2004) and the A21 dike (AMEC, 2007).

8.3.3. AMEC Analyses for 2007 A21 Dike Alignment

AMEC (2007) undertook finite element seepage analyses of the 2007 A21 dike alignment. Those analyses are provided in Appendix G. AMEC (2007) stated the objectives of the two-dimensional, finite element seepage analyses to be as follows:

- Check that the basic design criteria (from A154 dike, and carried over for A418) of seepage not exceeding 1.8 l/min/m of dike could be shown analytically to be met.
- Estimate the quantity of lake water leakages through and under the dike section that will eventually report to the downstream seepage collection system and to the A21 open pit.
- Evaluate potential seepage exit gradients within the till foundation soils to the downstream of the cut-off walls to determine if internal erosion concerns exist.

The AMEC analyses considered three sections for the 2007 alignment (see Figure 1-3 and Figure 1-4 in Appendix G):

- Sta. 0+450 m - representing a typical shallow dike section without a filter blanket, in which no bedrock grouting was to be performed.
- Sta. 1+050 m - representing the deepest water section of the dike (lakebed El. 396 m at the cut-off wall location). This section was analyzed both without and with the open pit, so that the effects of the open pit drawdown on dike seepage could be evaluated.
- Sta. 1+450 m - in an area of relatively steeply sloping lakebed till to the downstream side of the dike, where potential concerns over high seepage exit gradients were identified in a till cut required for the open pit access ramp.

AMEC (2007) concluded the following from the A21 dike seepage analyses:

- The basic design criteria (from A154) of seepage not exceeding 1.8 l/min/m of dike was analytically satisfied for both of the shallower sections and exceeded for the deepest section, based on the hydraulic conductivities assumed.
    - At the deepest section (Sta. 1+050 m) a seepage rate of approximately 3 l/min/m was predicted through the dike and foundation, and indicated to report to the toe drain trench (for the no open pit case).
    - Following pit development, this rate was predicted to increase to approximately 5 l/min/m (due to increased overall hydraulic gradient associated with the open pit groundwater sink) with the amount of flow reporting to the toe drain reduced to roughly 1 l/min/m.
    - The A21 dike design includes installation of pressure relief wells in the areas where the dike foundation is less than El. 402 m, as was done for the A154 and A418 dikes.

- Hydraulic gradients at the interface between the lakebed sediments (assuming they are left in place below the dike) and the filter blanket were shown to be in excess of the specified critical gradient of 0.3 for Sta. 1+050 m. This analytically demonstrates the criticality of the filter blanket in the deeper sections of the dike. Exit gradients at Sta. 0+450 m (no filter blanket as lakebed is higher than El. 414 m downstream of the cut-off wall) do not exceed the critical gradient.
- At Stn. 1+450 m (filter blanket at el. 410 m) gradients are predicted to be above critical in the lakebed sediments and tills, again confirming the need for the filter blanket for dike sections subject to significant hydraulic head.
- The excavation of the open pit was predicted, consistent with the A154 and A418 dikes experience (see Section 8.3.5), to result in a cone of groundwater depression that will eliminate upward seepage gradients downstream of the cut-off wall, to the point that eventually there will be no upward seepage into the filter blanket and shallow foundation piezometers will dry up. This analytical result simply confirms expectations and the monitored experience at the A154 dike.
- There exists the potential for a seepage face to develop on the large till cut at Sta. 1+450 which will exhibit critical exit gradients on the till surface. This may contribute to significant erosion and risk of slope instability of the till face. Armoring of the lower till slope would be required to mitigate this risk.

8.3.4. Validity of the AMEC (2007) Analysis for the Dike as Realigned in 2012

The shift to the 2012 alignment carries no significant implications in terms of the seepage analyses undertaken by AMEC (2007) for the 2007 dike alignment. The maximum height of the dike increased by only 1 m, and remains well below the maximum heights of the A154 and A418 dikes. The results of the analyses undertaken in 2007 remain fully representative, such that no additional analyses are warranted. That conclusion is supported by the monitored seepage and piezometric performance of the A154 and A418 dikes, which provides a better indicator of likely A21 performance, and is briefly summarized below.

8.3.5. Piezometric Response to Sinking of A154 and A418 Open Pits

NKSL (2007b, 2010, 2011) and DDMI (2011) provide time-history plots and summaries for the A154 and A418 dikes' foundation piezometers. Representative examples of the A154 and A418 data are provided in Figure 8-5 through Figure 8-7. For the post-dewatering period, the observed trends can be summarized as follows (DDMI, 2011):

- Gradients for foundation piezometers closest to the cut-off wall on its downstream side tended to indicate slight upward gradients in the immediate post-dewatering period. However, in most cases, gradients gradually transitioned to downward gradients thereafter, in response to the drawdown effect of the open pits.
- The more shallow of the piezometers in the lakebed till downstream of the cut-off wall tended to become dry, or mostly dry with only seasonal response, within two to three years of the completion of the dikes. An exception to this is the portion of the A418 dike between Sta. 0+600 m to 0+925 m where upward seepage gradients still

existed (see Figure 8-6) as of July 2012, as indicated by nested piezometers near the top of bedrock and the base of the lakebed till. This represents the deepest water section of either of the two dikes.

- Piezometers installed in till and bedrock upstream of the cut-off walls for both the A154 and A418 dikes indicated downward seepage gradients, likely in response to the drawdown effect of the open pits, the A154 north wall pit slope depressurization system, and the underground development below the mined-out open pits.
- 80% of the A154 dike's foundation piezometers were dry as of 2011, with temporary recharge during freshet in response to infiltration of snowmelt in the in-field area.
- Seepage stopped reporting to A154 dike's pumping stations' wells shortly after underground dewatering galleries went on line.

In terms of monitored seepage rates, these can only be inferred from pumping records from the dike pumping stations (DPS). Flows to the DPS locations, combined with operational issues, such as the frequent need to operate them manually, have made it difficult to quantify seepage rates for both dikes.

- NKSL (2007b) estimated the total seepage rate for the A154 dike to be less than 10 litres/sec at that time, which translates to about 0.24 litres/minute per meter of dam length (well below the design criterion of 1.8). Since then, with about 80% of the A154 dike's piezometers dry, the seepage rate has likely dropped significantly, in response to the pit sinking, the pit slope depressurization system, and the underground development.
- NKSL (2011) estimated the seepage rate for the A418 dike (as of 2011) to be less than 3.4 litres/sec. For an in-lake dike perimeter of about 800 m, that translates to a unit seepage rate of less than 0.26 litres/minute per meter of dam length.

In summary, the seepage performance of the A154 and A418 dikes has been satisfactory with seepage rates less than designed for, and upward gradients during pool dewatering transitioning to downward seepage gradients in response to open pit excavation. No areas of high concentrated seepage, and no areas of boiling or unstable/soft ground due to seepage, have been noted at or beyond the downstream dike toes. Similar acceptable seepage performance is anticipated for the A21 dike.

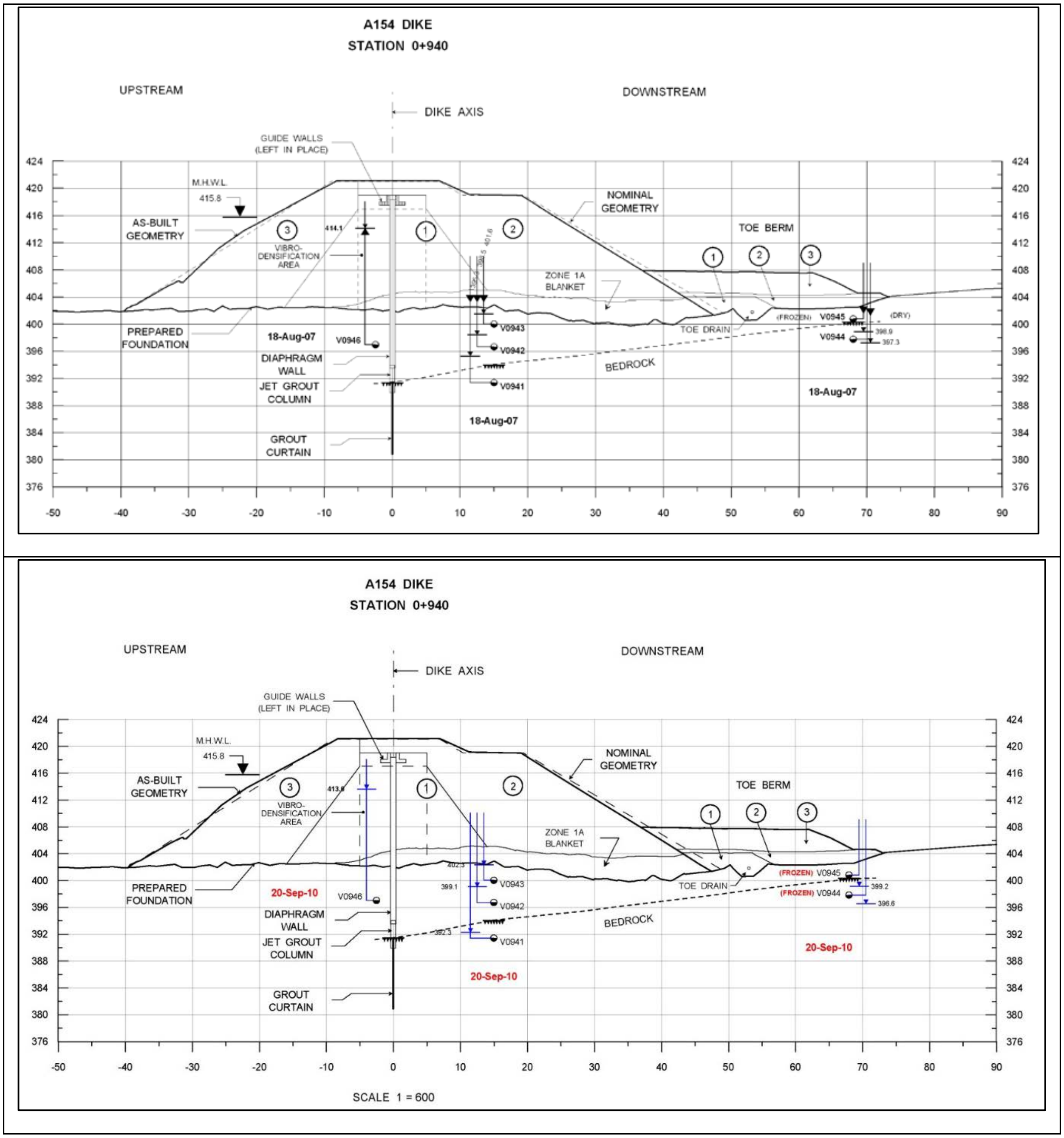


Figure 8-5. A154 Dike piezometric heads at Sta. 0+940 m: August 2007 (top) and September 2010 (bottom), indicating change in downstream piezometers from near hydrostatic to strong downward gradients from 2007 to 2010.

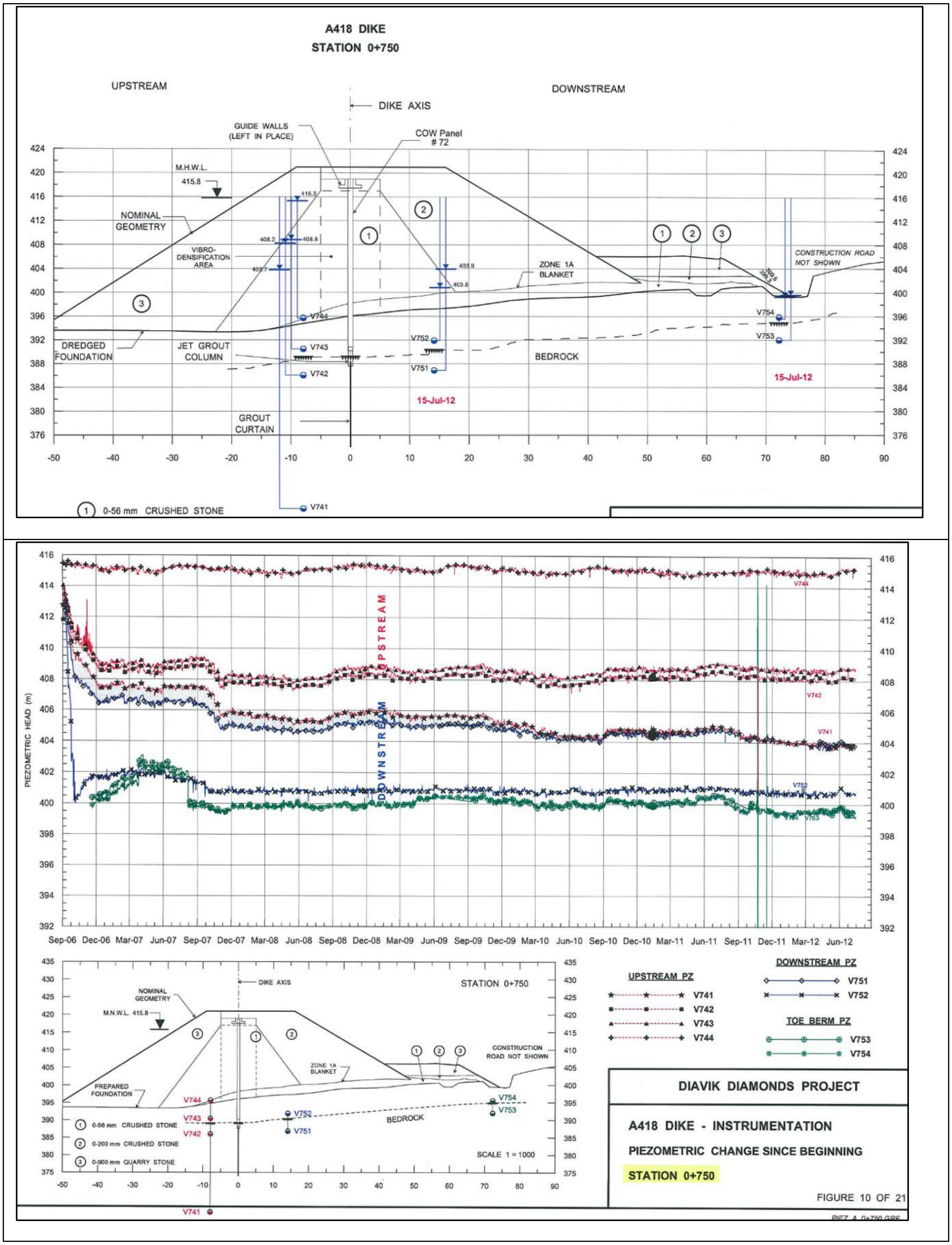


Figure 8-6. Typical piezometer section and time-history Data - A418 Dike (Sta. 0+750 m), in portion of A418 dike where upward gradients persist in the piezometers downstream of the cut-off wall.

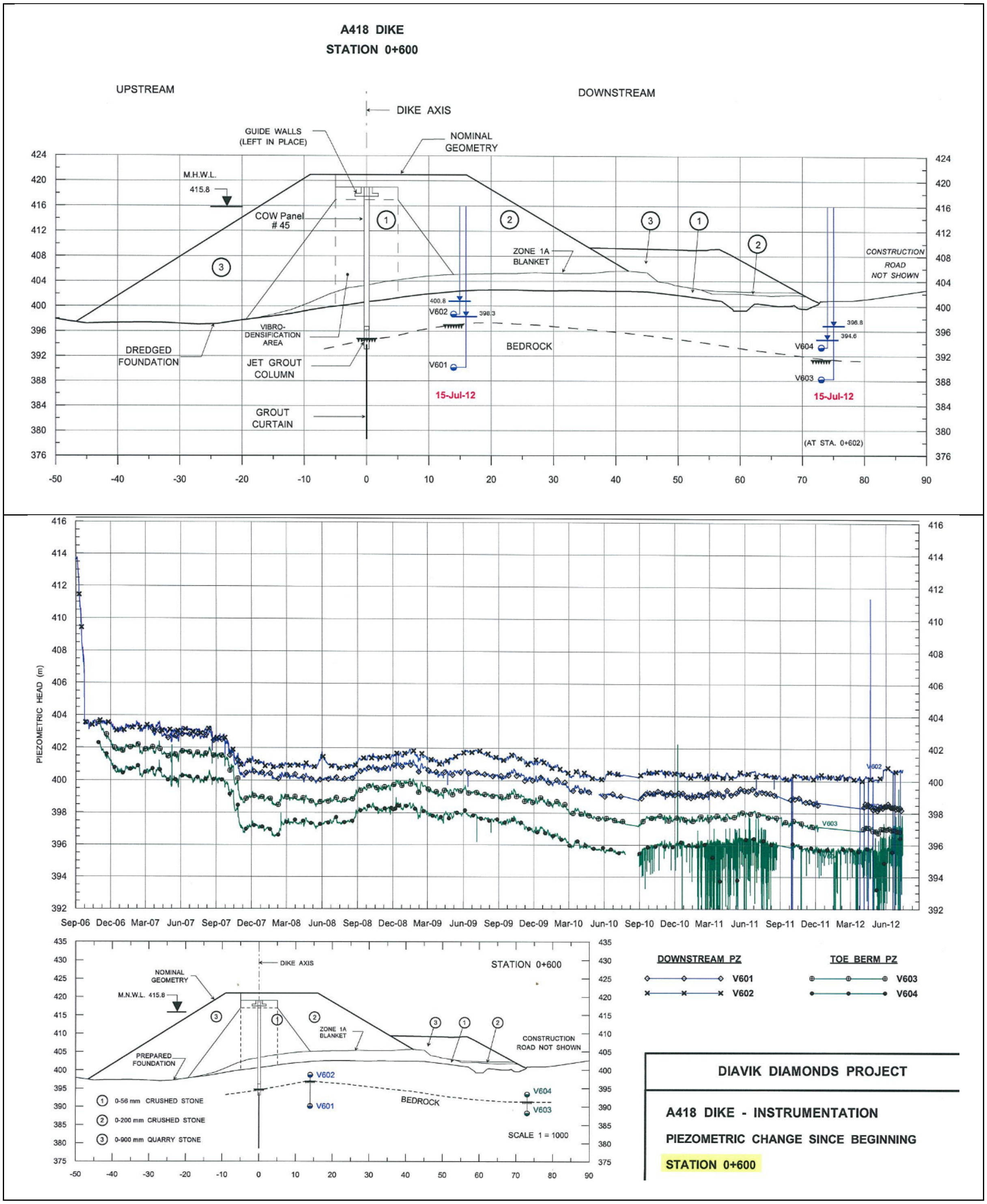


Figure 8-7. Typical piezometer section and time-history data - A418 dike (Sta. 0+600 m), where vertical gradients are variable downstream of the cut-off.

8.4. Stress Deformation Analyses

8.4.1. General

Stress-deformation analyses for the Diavik dikes have been carried out by NKSL (1999) for the A154 dike, NKSL (2004) for the A418 dike, AMEC (2007) for the A21 dike, and BGC for the A21 dike as part of this design report update. The A154 dike stress-deformation analyses were also described by Zhou et al. (2004).

The NKSL analyses focused on the deformation of the dike in response to pool dewatering, and the resultant imposed stresses and strains on the cut-off wall. The results of those analyses informed the design of the plastic concrete portion of the cut-off wall in terms of compressive strength, elastic modulus, and ductility (strain to failure).

The AMEC (2007) analyses built upon those carried out by NKSL, specifically modifying the strength and deformation characteristics of the lakebed till on the basis of laboratory testing in order to obtain a better match between predicted and monitored deformations for the A154 and A418 dikes. The AMEC (2007) analyses also considered the effects of pit wall dilation given the reduced setback criterion (see Section 3.8) adopted for A21.

The following sections summarize the analyses completed to date for the A21 dikes, provide a comparison between predicted and monitored cut-off wall deformations for the A154 and A418 dikes, and describe additional analyses undertaken by BGC in support of this design update.

8.4.2. NKSL Analyses for A154 and A418 Dikes

A series of deformation and stress analyses were carried out to predict the behavior of the A154 and A418 dike embankments particularly during the period of initial dewatering. This period is critical because the balanced dike loading is changed when one side of the dike is dewatered. The total weight of the fill downstream of the cut-off wall starts to act on the dike rather than just the buoyant weight, and hydrostatic pressure from the lake acts against the upstream side of the cut-off wall as the water level is drawn down. The resultant deformation of the embankment exerts stresses on the cut-off wall, which the wall must accommodate and still achieve the specified performance criteria.

The NKSL analyses were carried out with the finite element software SIGMA/W to estimate displacement of A154/A418 cut-off walls in response to pool dewatering. However, the program does not permit the introduction of structural elements that would be required to simulate the stresses in the cut-off wall. Therefore, NKSL conducted non-coupled analyses using the deformations obtained from SIGMA/W as inputs to the structural analysis software P-FRAME and S-FRAME for both the A154 and A418 dikes. The cross sections analyzed were selected to be representative of the alignment of the A154/A418 dikes at the location where deeper bedrock surfaces existed. Table 8-6 gives the lake depth and lakebed till thickness at the modelled sections. The strength and stiffness parameters for the embankment and foundation materials were chosen based on field and laboratory testing results and the Duncan and Chang hyperbolic constitutive model (Duncan et al., 1980; Duncan and Chang, 1970). Upper bound and lower bound hyperbolic parameters were used for the till materials to account for limited testing data and till variability.

Table 8-6. Lake depth and lakebed till thickness at modelled sections.

	A154 Dike	A418 Dike
Lake Depth	18 m (lake surface modeled at El. 415.8 m)	27 m (lake surface modeled at El. 417 m)
Till Thickness	8 m	12 m

The deformations predicted by SIGMA/W were the input to a structural analysis of the cut-off wall using the software P-FRAME/S-FRAME to provide extreme fiber stresses due to bending moments and axial compression. A minimum factor of safety of 1.5 for compressive strength over extreme fiber stress was adopted for stress analysis. The details of the model, modeling stages and the findings of the analyses are presented in A154 and A418 final design reports (NKSL, 1999 and 2004, respectively). Zhou et al. (2004) describe the analyses carried out for the A154 dike.

8.4.3. Comparison of Predicted and Monitored Deformations

8.4.3.1. General

The deformation monitoring data from the A154 and A418 dikes allow comparisons between the predicted and monitored deformations for those dikes. The following sections describe the monitoring facilities for those dikes, present the pertinent monitoring data, and provide the comparison with the analysis predictions.

8.4.3.2. A154 Dike Monitoring

The A154 dike was constructed in 2001 and 2002. The initial dewatering took place from late July to mid-September of 2002.

A comprehensive set of instruments was installed in the dike and the foundation to monitor the behavior of the dike. The instruments included:

- Eight inclinometers, six of which were installed in the cut-off wall, and two located in the embankment downstream from the cut-off wall. Table 8-7 shows the lake depth and lakebed till thickness at the location of the inclinometers installed in the A154 dike cut-off wall
- Survey markers along the upstream and downstream dike crest
- Survey pins on the top of the cut-off wall to monitor horizontal and vertical deformations.

Table 8-7. Lake depth and lakebed till thickness at A154 inclinometer locations.

	IN044	IN064	IN094	IN219	IN270	IN390
Lake Depth (m)	16	21	14	8	13	10
Till Thickness (m)	6	8	10	13	9	11

The instrumentation system also included piezometers, extensometers (which were removed later), thermistor strings, flow meters installed in the dike pump stations and portable seismographs. The instrument locations on the dike layout and profile, the inclinometer displacement profiles and survey records are included in the A154 dike annual inspection report (NKSL, 2007b).

8.4.3.3. A418 Dike Monitoring

The A418 dike was constructed in 2005 and 2006. Initial dewatering took place from late August to late September of 2006.

As with the A154 dike, a comprehensive set of instruments was installed in the dike and the foundation to monitor the behavior of the dike, especially after initial dewatering. For deformation monitoring, this included:

- Seven inclinometers, six of which were installed in the cut-off wall at selected locations and one located in the toe berm at the deepest part. Table 8-8 shows the lake depth and lakebed till thickness at the location of the inclinometers installed in the cut-off wall
- Survey markers (upstream and downstream dike crest)
- Survey pins on the top of the cut-off wall to monitor horizontal and vertical deformations.

Table 8-8. Lake depth and lakebed till thickness at A418 inclinometer locations.

	IN560	IN650	IN720	IN820	IN890	IN930
Lake Depth (m)	16	18	21	18	25	18
Till Thickness (m)	10	6	7	8	8	10

The instrument locations on the dike layout and profile, inclinometer displacement profiles, and survey records are provided in the A418 dike annual inspection reports (NKSL 2007b, 2010 and 2011).

8.4.3.4. Predicted Cut-Off Wall Deformations in Response to Pool Dewatering

The range of displacements of the cut-off wall in A154, A418 and A21 as predicted in the NKSL and AMEC deformation/stress analyses are shown in Figure 8-8. The results are presented with the top of bedrock as the base level. The top of the bedrock and lakebed till are represented by two bands to account for the variation in dike height and lakebed till thickness used in the respective sets of analyses.

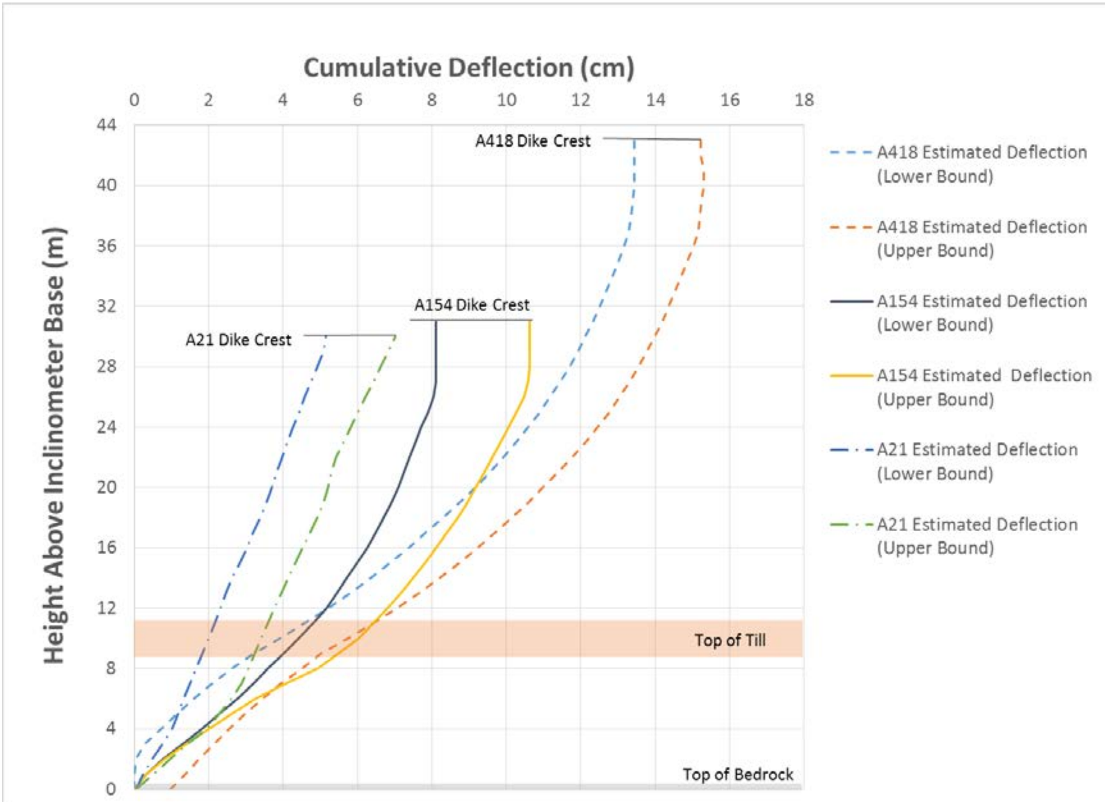


Figure 8-8. Range of predicted cut-off wall deformations in response to pool dewatering by NKSL (1999, 2004) and AMEC (2007).

The range between the lower and upper bound deflection values shown in Figure 8-8 resulted from the range of stiffnesses and thicknesses of foundation material:

- For A154, the lower bound values correspond to the cut-off wall deflection profile assuming homogenous stiffer lakebed till and the upper bound values correspond to softer lakebed till.
- For A418, the upper bound values represent the cut-off wall deflection profile assuming the dike rests on a 12 m thick till foundation and the lower bound values represent the deflection profile accounting for a thinner till foundation layer. NKSL applied a correction offset to the upper bound values to obtain the lower bound values, therefore the upper bound deflection profile shows defection values > 0 in the bedrock.
- As with A154, the lower bound deflection values of A21 dike correspond to homogenous stiffer lakebed till and the upper bound values correspond softer lakebed till. The till properties were derived from drained and undrained, isotropically-consolidated (CIU) triaxial compression tests. Those tests yielded higher stiffnesses for the lakebed till than were assumed for the A154 and A418 analyses, hence the reduced predicted deformations within the lakebed till for A21.

The deformation and stress analyses of the cut-off wall in the A154, A418 and A21 dikes predicted maximum extreme fiber stresses of about 780 kPa, 900 kPa and 890 kPa, respectively. The predicted stresses for A21 dike are quite high considering it has a shallower depth than A154 and A418 dikes; this is attributed to the different modelling approach and software used for A21 dike analysis. This is described further in Section 8.4.4 below. Comparing the predicted extreme fiber stresses with the available compressive strength of the plastic concrete (based on tests on samples cured for 28 days) yielded factors of safety ranging from 1.35 to 1.5 for the A154, A418 and A21 dikes. This factor of safety would increase as the plastic concrete ages and gains strength after 28 days.

The predicted deformation profiles in Figure 8-8 show a generally smooth cumulative displacement pattern of the cut-off wall, with no sharp bends at the interface between the dike fill and the foundation till. There is a small bend near the bedrock to lakebed till interface.

The deformation and stress analyses also predicted that the maximum extreme fiber stresses would occur at the base of the lakebed till (i.e. bedrock interface), below the 3 m penetration depth of the plastic concrete cut-off wall in the till foundation, and within the jet-grouted portion of the cut-off wall. The maximum extreme fiber stresses are imposed on the jet grout column and have no interaction with embedded portion of the cut-off wall in the lakebed till, except in areas where the thickness of the lakebed till is minimal, and the plastic concrete wall extends to or very close to the bedrock surface. In such areas, however, the water is shallower and deformations will be reduced due to typically lesser dike height, which will yield reduced extreme fiber stresses, lower than those the cut-off wall is designed to sustain.

8.4.3.5. Inclinometer Data

Selected inclinometer deformation profiles for the A154 and A418 dikes are presented in Figure 8-9 and Figure 8-10, respectively.

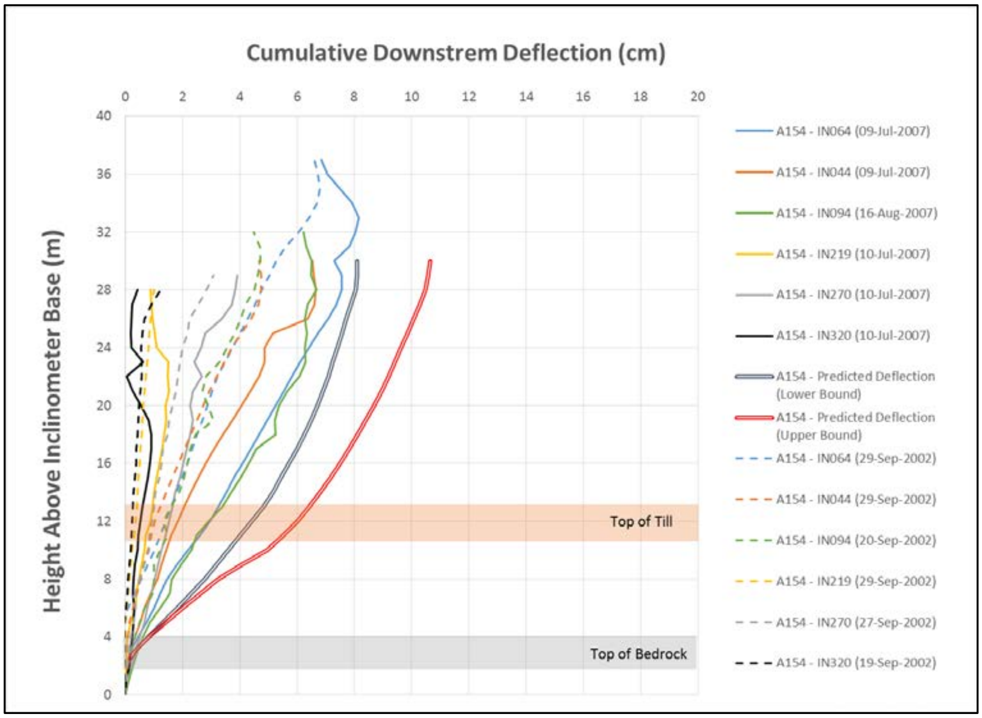


Figure 8-9. Deflection profiles of A154 dike: predicted and monitored.

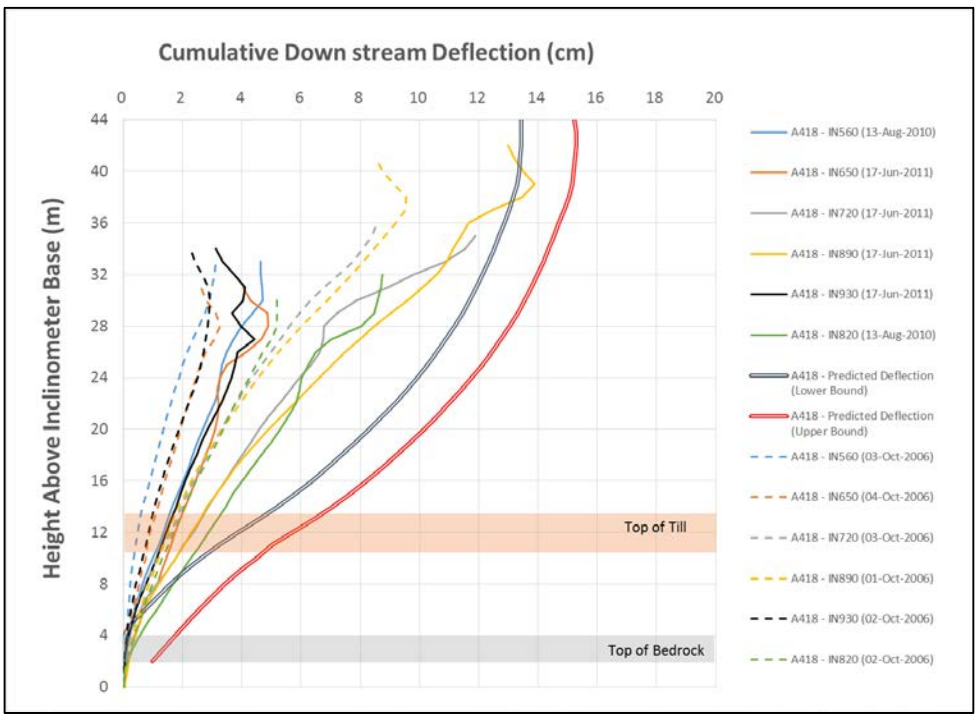


Figure 8-10. Deflection profiles of A418 dike: predicted and monitored.

The profiles presented in these two figures represent the cumulative deformation (deflection) of the cut-off wall in the downstream direction (perpendicular to the dike axis) during the period of:

- Initial dewatering that took place from late July to mid-September of 2002 for A154 dike and late August to late September of 2006 for A418 dike (dashed lines)
- Continuing embankment movement subsequent to the initial dewatering until August 2007 for A154 dike and August 2011 for A418 dike (solid lines).

Note that the dike heights vary at the inclinometer locations, so the results are presented with the bottom of the inclinometers as the base level. The top of the lakebed till and bedrock are represented by two bands to account for the ranges corresponding to the inclinometer locations and the dike sections that were modeled.

The records of other inclinometers that were installed outside the cut-off wall to address specific questions such as deep foundation movement associated with mining were reviewed but are not presented herein.

Review of the inclinometer deflection profiles in Figure 8-9 and Figure 8-10 indicates the following key points:

- The higher dike sections yielded the largest cumulative deformations at the top of the cut-off wall, as would be expected. As the height of the dike increases, the anticipated cumulative movement at the top of the inclinometer increases approximately by the square of the height (based on calculations).
- The most pronounced deformation occurred during and immediately after initial dewatering, with the cut-off wall tilting toward the downstream. After dewatering, there was some continuation of deformation but to a lesser degree. The same movement pattern during and after dewatering was also observed at the survey markers installed near the upstream and downstream crests of the dike.
- In spite of the continuing embankment deformation after dewatering, the cumulative deflections remained below those predicted in the deformation analyses for A154 and A418. The cut-off wall was designed to accommodate the stresses induced from those predicted strains with a safety factor of 1.5. The post-dewatering inclinometer and survey records show that the embankment deformation has decelerated significantly, and the long-term deflection of the cut-off walls is expected to be well less than predicted for the pool dewatering phase.
- The inclinometer deflection profiles appear to be somewhat more linear overall than the predicted deflection profiles, which limits reduced extreme fibre stresses.
- The irregularities in the deflection in the top portion of the profiles are mainly due to the spreading of the dike crest as indicated by survey monument records.
- The inclinometer plots indicate that there was no change in the deflection pattern nor significant localized displacements at the lakebed till to dike fill interface. In the NKSL analyses there was a contrast in stiffness, and therefore concentration of stresses at this interface, and a predicted non-linear cumulative displacement pattern.

To better illustrate the last observation, strain and incremental strain profiles were generated for A154 and A418 dikes based on the inclinometer displacement data presented in Figure 8-9 and Figure 8-10, and the NKSL model predictions. The strains were calculated by dividing the inclinometer cumulative displacement at successive heights above the inclinometer base by the relevant inclinometer height above the base. The incremental strains were calculated by dividing the inclinometer incremental displacement at successive inclinometer data points along the profile by the elevation difference between the two measurement points. The strain and incremental strain profiles are a means of normalizing the predicted and monitored deflections (strains), and highlighting the zones with significant localized displacements (stress concentrations) along the cut-off wall (incremental strains). The strain and incremental strain profiles for A154 and A418 dikes based on deformation data obtained from analyses and inclinometers records are presented in Figure 8-11 through Figure 8-14.

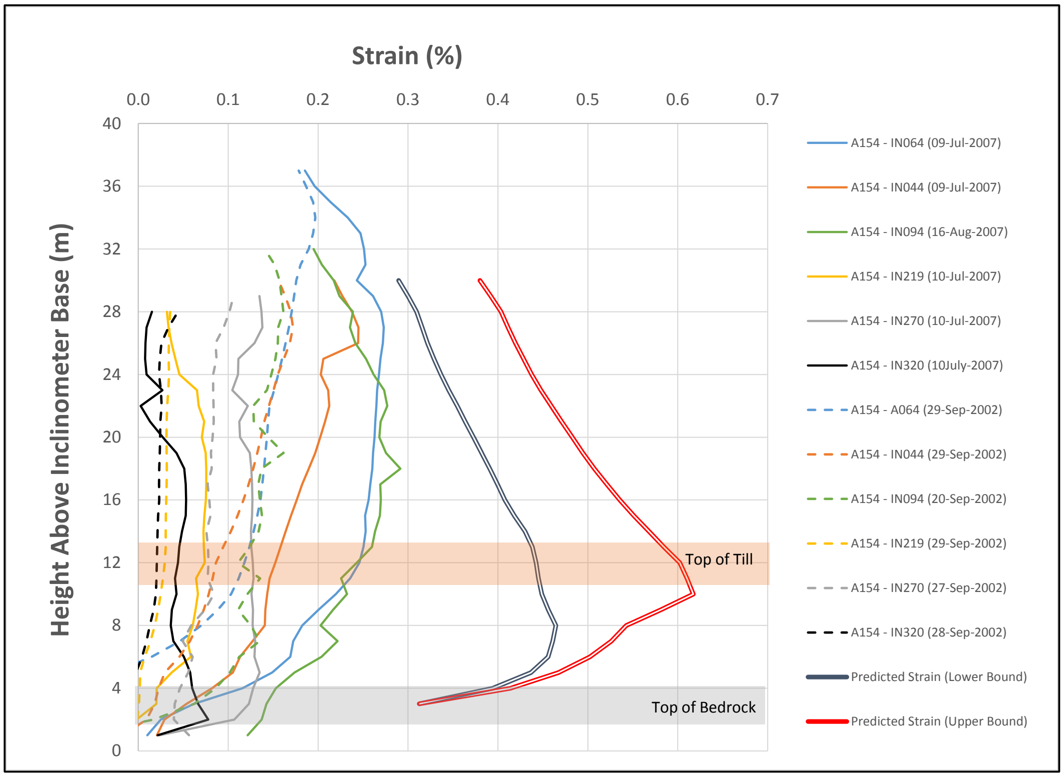


Figure 8-11. Strain profiles for A154 dike.

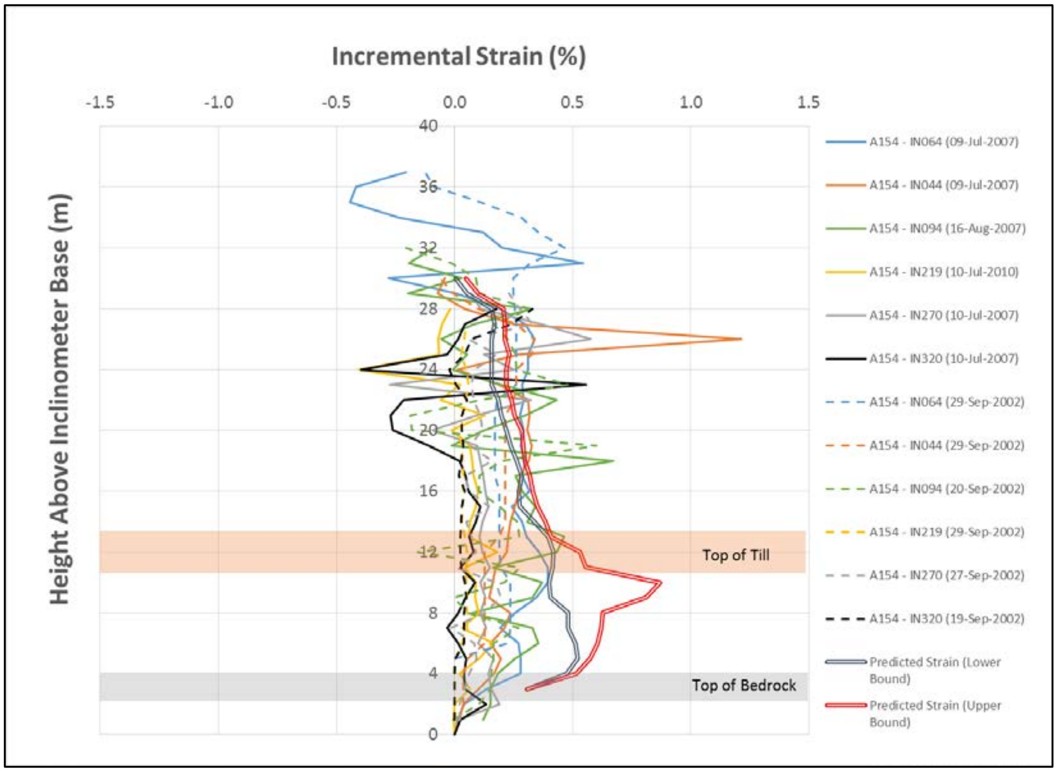


Figure 8-12. Incremental strain profiles for A154 dike.

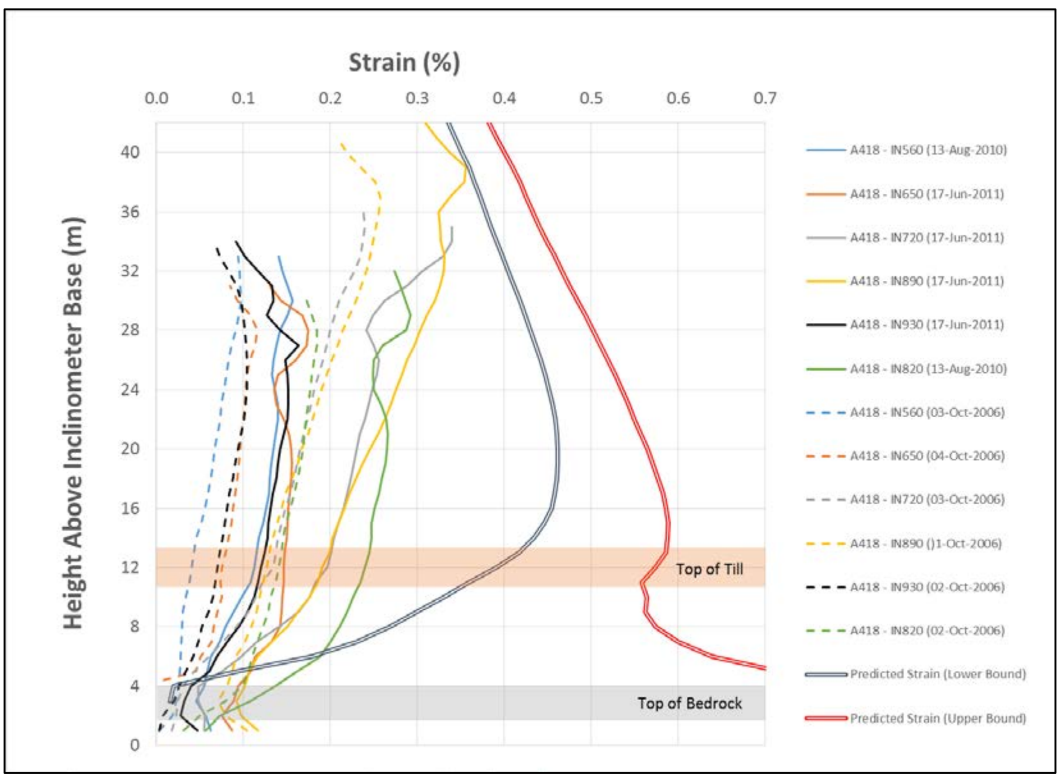


Figure 8-13. Strain profiles for A418 dike.

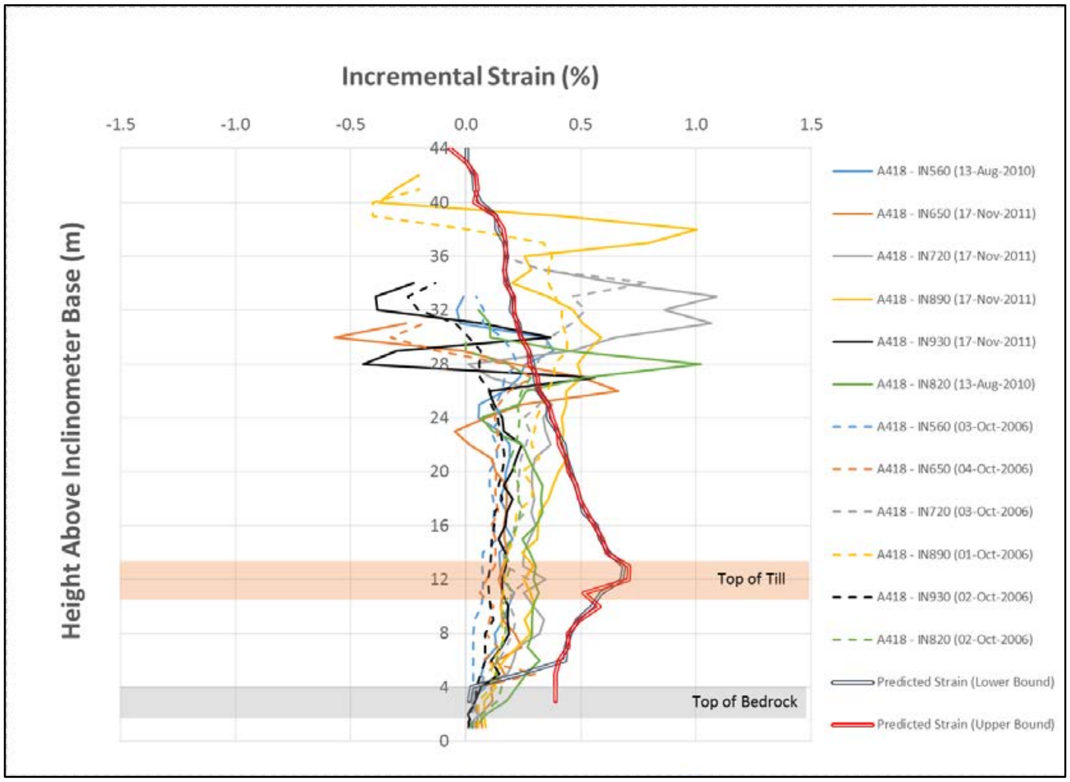


Figure 8-14. Incremental strain profiles for A418 dike.

The predicted upper bound strain value at the base of the cut-off wall in A418 dike appears anomalously high on Figure 8-13. This higher strain is an artifact of using an offset to obtain the upper bound displacement value at the base of the cut-off wall (as explained earlier), which resulted in predicting a displacement higher than zero at the cut-off wall base. The upper and lower bound predicted incremental strain profiles for the cut-off wall in A418 dike overlap because of using a constant offset to obtain the upper bound displacements based on the lower bound ones.

The strain profiles in Figure 8-11 through Figure 8-14 indicate the following key points:

- The strain and incremental profiles for the inclinometers do not exhibit any pattern signifying the presence of zones with substantial localized displacements (stress concentration) along the lower part of the cut-off wall including, of particular interest for this review, at or near the lakebed till to dike fill contact. It should be noted that there are some zones well above the till to dike fill interface where more substantial, but still acceptable, localized strains are observed. However, this is attributed to the distortion of inclinometer readings due to the spreading of the dike crest, as discussed above.
- The maximum strain indicated by the inclinometers is < 0.3% for A154 dike and <0.4% for A418 dike. This is below the maximum value of about 0.6% predicted by the deformation/stress analysis for both dikes.

- The maximum incremental strain observed in the lower part of the cut-off wall is <0.5% for A154 and A418 dikes during the dewatering phase, which is below the maximum value predicted by the deformation/stress analysis for both dikes. Some of the incremental strains observed at the top region of the dike are higher than 0.5% and exceed the values predicted by the deformation/stress analyses. However, this is attributed to the distortion of inclinometer readings due to the spreading of the dike crest, as discussed above.

8.4.3.6. Survey Monument Data

Survey markers were installed at the upstream and downstream crests for both the A154 and A418 dikes. The A154 monument survey methods achieved an accuracy of limited utility relative to the magnitude of movements. However, improvements in pit slope monitoring and survey methods resulted in a much-improved survey monument data set for the A418 dike. Therefore, only survey monument data from the A418 dike were reviewed and analyzed by BGC. The locations of survey markers along A418 dike and vectors of their cumulative horizontal displacement are shown in Figure 8-15, adapted from the 2011 A418 dike annual inspection report (NKSL, 2011).

<u>Dike Crest Settlement</u>

The survey markers provided vertical as well as horizontal movement data, whereas the inclinometers provided horizontal displacements only. Surveys of the survey markers indicate the following with regards to the monitored vertical displacements (NKSL, 2011):

- On the downstream side of the crest, settlement occurred primarily during the dewatering phase, likely as a result of the change in stress within the embankment fill as the unit weight increased from buoyant to total. The maximum cumulative values of about 155 mm measured in 2011 were as anticipated and little changed from the previous year.
- The settlement of the crest on the upstream side is much less as no such change in stress regime took place - the upstream crest movements are associated with long term creep (and associated spreading) of the embankment fill.

<u>Dike Crest Horizontal Displacements</u>

The survey markers near the downstream crest of the A418 dike indicated maximum downstream displacement of 195 mm through 2011. This is greater than the movements recorded at the top of the inclinometer casings. The increased downstream displacement near the edges of the crest is likely due to increased settlement in these areas due to spreading of the fill. Figure 8-15 shows lake depths and till thicknesses for various segments along the A418 dike alignment, and the locations of the inclinometers and survey markers.

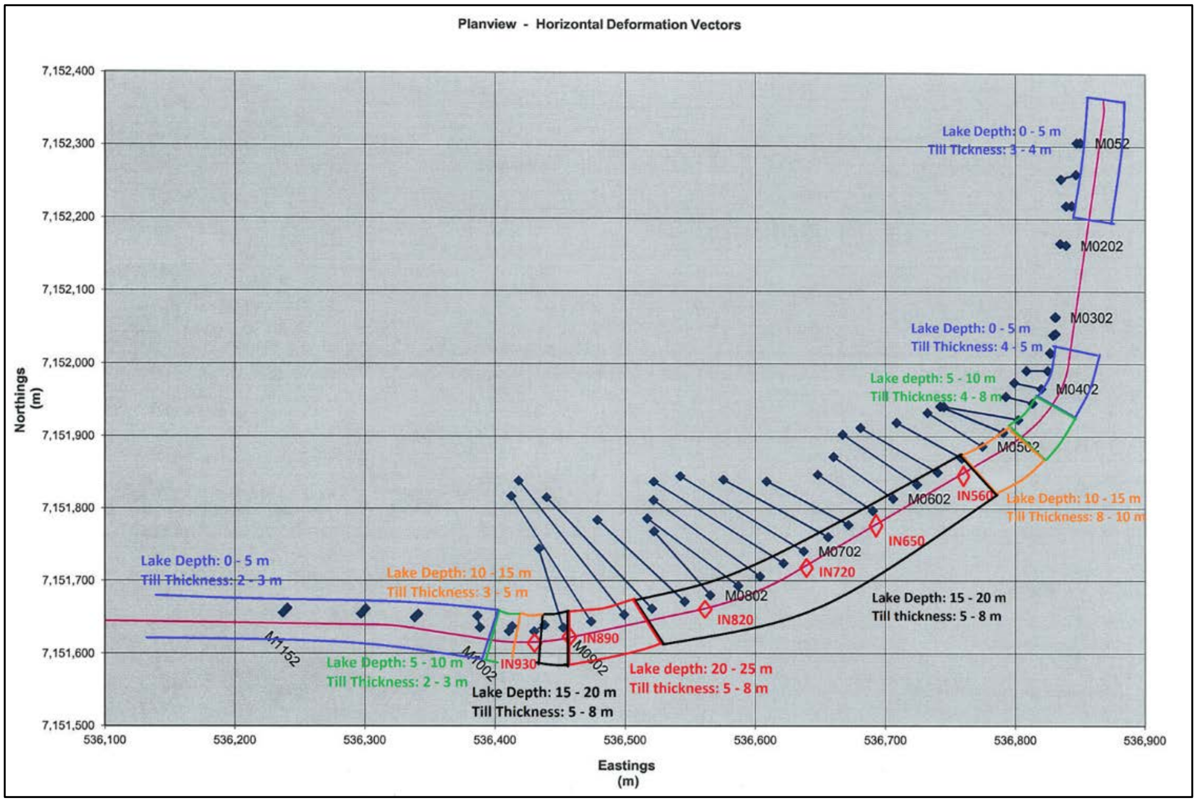


Figure 8-15. Downstream crest survey monument movement vectors for A418 dike.

As is evident on Figure 8-15, the displacement vectors show downstream movements in the shallow water sections (lake depth 5 m and less) substantially less than those in the deeper water areas. There is also some correlation to lakebed till thickness, as for a given range of lake depth, displacements are larger for increased till thickness. In areas where the lake depth is in the range of 5 m to 10 m, monitored displacements are also significantly lower (by at least 50%) than in the deeper water portion of the A418 dike.

These observations are of significance given that the A21 dike is to be constructed in relatively shallow water for a significant portion of its alignment, as summarized Table 8-9. In general, lakebed till thickness along the A21 dike alignment correlates reasonably well with lake depth.

Table 8-9. A21 dike alignment - lake depth ranges.

Lake Depth Along A21 Dike Alignment (m)	Length of Dike (m)	Percentage of the In-lake Portion of the Dike
Depth < 5	820	45
5 ≤ Depth < 10	260	15
10 ≤ Depth < 15	490	28
Depth ≥ 15	210	12

It is therefore unlikely that the degree of ductility incorporated into the cut-off wall design will be mobilized given lower the deformations expected for a substantial portion of the A21 dike alignment. This then offers some flexibility in the embedment depth of the plastic concrete CSM portion of the cut-off into the lakebed till in the shallow water areas. It also provides an additional factor of safety in terms of the structural design criteria for the cut-off wall (see Section 3.6.1.4).

8.4.3.7. Predictions vs. Monitored Data: Conclusions

The review indicated the monitored deformation behaviour of the A154 and A418 cut-off walls for the A154 and A418 dikes to be well within design analysis predictions, in terms of both absolute displacements, and concentration of displacements that would have increased the potential for cracking. Concentrated displacements predicted in design analyses were not observed in the inclinometer profiles at or near the lakebed till to dike fill interface. Furthermore, the maximum bending stresses were predicted by deformation/stress analysis to occur at the base of the till foundation layer (where through-going cracks are of lesser consequence), which is below the embedded depth of plastic concrete portion of the cut-off wall and in the jet-grouted portion.

Although no inclinometer data is available from the shallower water portions of the A154 and A418 dike alignments, survey monument data for the A418 dike confirm that displacements (and strains) within the cut-off wall in water less than 5 m in depth are about an order of magnitude less than those monitored in the deep water sections of both dikes.

8.4.4. AMEC (2007) Stress-Deformation Analyses

8.4.4.1. General

The AMEC (2007) stress-deformation analyses for the 2007 alignment of the A21 dike considered deformation of the dike and the cut-off wall for two driving mechanisms:

- Stress changes imposed during pool dewatering
- Effects of A21 open pit wall dilation.

The AMEC (2007) stress-deformation analyses are provided in Appendix H.

8.4.4.2. Modeled Response to Pool Dewatering

Results from the AMEC (2007) FLAC deformation modeling for the pool dewatering phase are summarized, along with the NKSL (1999, 2004) results for the A154 and A418 dikes in Figure 8-8. As outlined in AMEC (2007), the A21 dike analyses predicted less deformation of the cut-off wall during pool dewatering owing to modification of the NKSL stiffness parameters of the lakebed till based on laboratory test data.

The conclusions of the AMEC (2007) analyses for the pool dewatering phase are summarized as follows:

- As observed in the previous analyses of the A154 and A418 dikes, the maximum vertical displacement occurs close to the edge of the downstream crest. The predicted maximum vertical displacement in the dike was in the order of 50 mm. The estimated maximum horizontal displacement in the proposed A21 dike is about 50 mm.
- The maximum horizontal and vertical displacements of the top of cut-off wall for the proposed A21 Dike were estimated to be about 45 mm and 25 mm, respectively.
- The predicted deformation patterns and amounts along the cut-off wall reasonably agreed with monitored displacements of the A154 Dike where it is approximately the same height as the modeled A21 dike section.
- The maximum bending moment in the cut-off wall was predicted to occur at the bottom of the till foundation level. The estimated maximum bending moment and axial force were predicted to be about 12 kN m and 350 kN, respectively.
- A range of maximum bending moments was estimated based on sensitivity analyses. The possible range of maximum bending moments was 10 kN m to 30 kN m and the corresponding range of axial forces was 220 kN to 500 kN.
- Based on the different cases analyzed, the maximum extreme fibre stress in the cut-off wall was about 890 kPa. The available strength of the plastic concrete at 28 days is 1,220 kPa. Therefore, the factor of safety against maximum stresses imposed in cut-off wall exceeding the available strength is 1.37. This factor of safety is a bit lower than 1.5 as specified in the earlier analysis but it will increase with the age of the plastic concrete to a value of 2.09 at 180 days.
- The FLAC analysis was undertaken for the highest section of the proposed A21 Dike, which would be constructed where the lake is 19 m in depth. Lesser deformations, and stresses, will develop in the remainder of the proposed dike where the depths of water (and dike height) are less than for the section modeled.

These conclusions remain valid despite being based on the 2007 alignment of the A21 dike. The updated alignment results in the lake depth are only about 1 m deeper for the modeled section.

8.4.4.3. Modeled Effects of Pit Wall Dilation

As the A21 open pit mine is excavated and deepened, inward dilation of the pit walls in response to stress relief can be anticipated. The displacements associated with this dilation will be greatest at the pit walls, and will decrease with increasing distance from the pit walls.

Golder (2006b) modelled pit dilation with two-dimensional analysis using distinct element modeling, with the UDEC modeling package (Universal Distinct Element Code). The modeling assumed a ratio of horizontal stress to vertical stress of 2. The deformations under the A21 dike predicted in Golder's UDEC modeling, for the 2007 configurations of the dike and the open pit, are as summarized in Table 8-10 for the final pit shell. The degree of lateral strain, associated with pit dilation, between the downstream dike toe and dike centerline, are also given. Note that the same differential movement (2 cm) between toe and centerline yields different lateral strains for different sections along the dike. This is the result of differing dike heights, and therefore different distances from toe to centerline.

Table 8-10. Summary of Golder (2006b) UDEC model predictions for displacements below A21 dike.

Pit Sector	Joint Sets: Base Dip/Back Dip	Predicted Horizontal Displacements Below A21 Dike (cm)	Lateral Strain Below Dike Between Downstream Toe and Centerline (%)
Southeast 1	18/83	8 (toe) to 6 (centerline)	0.04
	33/83	14 (toe) to 8 (centerline)	0.04
Southeast 2	17/84	4 (toe) to 2 (centerline)	0.01
	30/84	2 (toe) to 0 (centerline)	0.01
North	21/82	6 (toe) to 4 (centerline)	0.04
	29/81	8 (toe) to 6 (centerline)	0.04
Northwest	21/81	< 8	0.01

The two-dimensional UDEC modeling cannot account for potential major structures (such as faults) across which the potential for significant differential straining exists. Such differential straining, particularly along steeply dipping features cutting across the axis of the dike, creates the potential for concentrated deformation with the cut-off wall in excess of the averaged strain fields projected via the UDEC modeling.

The dike-pit interaction modeling was performed by imposing the displacements predicted by the UDEC modelling onto the AMEC FLAC model boundary in the bedrock layer. The AMEC and Golder modelers met in 2007 with ITASCA Consulting Group experts in ITASCA's Minneapolis offices to review the UDEC and FLAC models, and develop a program to create the required interface between them (AMEC, 2007).

The dike-pit interaction modelling was carried out in the following sequence (AMEC, 2007):

- The pit wall dilation displacements just below the lake bottom till were extracted from the results of UDEC analyses performed by Golder (2006b). This procedure was performed for various stages through the pit development.
- The displacements corresponding to the first stage of pit development were first imposed on the boundary of the FLAC model which was in equilibrium under initial dewatered conditions. The pore pressures in the FLAC model was also updated for that stage with an approximate phreatic surface.
- Following the application of displacements, the FLAC model was stepped to equilibrium conditions.
- The predicted additional displacements for all subsequent pit stages were then applied in sequence.

The UDEC displacements incorporated into AMEC's FLAC modeling were based on conservative scenarios, rather than expected conditions. The findings of the integrated pit dilation-dike deformation modeling were as follows (AMEC, 2007):

- The pit wall dilations predicted by UDEC analyses were fairly linear and would not have concentrated displacements. Analyses indicated that these linearly varying pit dilations would not cause significant increase in maximum bending stresses in the cut-off wall.
- The cut-off wall and dike would experience a considerable amount of translation, compared to differential deformation, as a result of pit wall dilation. As expected, these translations would not cause much increase in bending moments and axial forces.
- The maximum bending moment in the cut-off wall was predicted to occur at the bottom of the foundation till. For the case of UDEC predicted pit dilation, the estimated maximum bending moment and axial force were predicted to be about 47 kN m and 430 kN, respectively, compared to values of 12 kN m and 350 kN from the analysis without pit dilation.
- Based on the analyses performed, maximum axial and shear strains in the cut-off wall would be in the order of 0.5%. Triaxial tests on plastic concrete indicate axial strains at compressive failure of 3% to 5% which correspond to failure shear strains of 4.5% to 7.5%. The estimated maximum shear and axial strains (~ 0.5%) in cut-off wall are much smaller than the corresponding strains (axial compressive strain 3%, shear strain 4.5%), at which plastic concrete would reach its maximum stresses.
- Analyses with cases of pit dilations inducing concentrated deformations, as might result from differential movement via geologic structure, produced considerable increase in maximum moment and axial force in the cut-off wall. Based on the analyses performed, the change/increase in the maximum bending moment and axial force also depend on the location of concentrated deformation. A concentrated deformation just downstream of the cut-off wall produced the largest maximum bending moment.
- A range of maximum bending movements and axial forces were estimated based on sensitivity analyses. The predicted range of maximum bending moments was 37 kN m to 71 kN m and the corresponding range of axial force was 430 kN to 610 kN. With no pit dilation, the ranges of calculated values were 10 kN m to 30 kN m and 220 kN to 500 kN.
- Based on the different cases analysed, the maximum extreme fibre stress in the cut-off wall is about 1,100 kPa compared to about 890 kPa without pit dilation.

8.4.4.4. Implications of 2012 Pit Design Update and A21 Dike Realignment

In 2012, the open pit was re-designed and the A21 dike realigned to suit the new pit design. The 50 m setback criterion between dike toe and the pit rim (see Section 3.8) was unchanged. However, the reconfiguration of the dike-pit system raised the question as to whether the integrated dike-pit deformation analyses as carried out in 2007 and described above remain valid.

Golder (2012) updated the pit wall dilation modeling for the revised pit design and A21 dike alignment. Their results are summarized in Table 8-11, and indicate the following:

- The average strains predicted below the realigned A21 dike are essentially the same as the Golder (2006b) analysis predictions, indicating that the FLAC analyses for the A21 dike need not be re-run
- Pit dilation strains that extend as far as the dike are predicted to occur primarily for the final stage of pit sinking - of the total pit depth of about 170 m at El. 230 m, the final 50 m of depth is achieved in the final year of mining.

Table 8-11. Summary of Golder (2012) UDEC model predictions for displacements below A21 dike.

Pit Sector	Pit Bottom Elev. (m)	Max. Pit Crest Horiz. Displacement (cm)	Max. Pit Slope Horiz. Displacement (cm)	Downstream Dike Toe Horiz. Displacement (cm)	Upstream Dike Toe Horiz. Displacement (cm)	Average Strain Below Dike (%)
North	350	1	2	1	1	0
	320	2	3	2	2	0
	230	7	9	6	6	0.02
Northeast	350	1	2	1	0.8	0
	320	2	4	2	2	0.01
	230	6	11	6	2	0.05
Southeast	350	1	2	1	0.8	0
	320	2	4	2	2	0
	230	8	14	7	6	0.03

Notwithstanding these modeling predictions, operation of the dike-pit system must include a comprehensive monitoring system tied into appropriate trigger action response planning (TARP) for the integrated open pit and dike system. A listing of potential pit wall dilation effects on the dike, and mitigating factors, is provided in Table 8-12. This is discussed further in Section 9.0. As an example, the TARP developed by DDMI for the A154 and A418 integrated dike-pit systems is provided in Appendix P. Golder (2012) provides additional guidance and details for the A21 TARP, including quantified triggers, actions, and responses related to pit wall deformations and pore pressure conditions.

Table 8-12. Potential effects of pit dilation on A21 dike (AMEC, 2007).

<table>
<tr><th>Effect Of Pit Wall Dilation</th><th>Potential Effect On Dike</th><th>Mitigating Factors</th></tr>
<tr><td>Uniform translation of dike towards pit</td><td>Negligible in terms of cut-off wall rotation. Potential for cracking of wall considering 3-D effects.
Increase of secondary hydraulic conductivity of bedrock due to dilation of joints and fractures, resulting in increased seepage below the dike, negating to some degree the effectiveness of the grout curtain.</td><td>Plastic concrete cut-off wall is designed with erosion resistance such that a crack is unlikely to progress into a failure. Consequence is increased seepage through the wall.
Similarly, jet-grouted portion of wall has erosion resistance. Effect of seepage through jet-grouted portion of the wall also mitigated by the downstream filter blanket against internal erosion.
As pit deepens, groundwater depression will reduce seepage reporting to seepage collection system. Could however see increased seepage into the open pit but this is to be handled by the pit dewatering system.</td></tr>
<tr><td rowspan="3">Extension of bedrock below dike due to differential straining perpendicular to the axis of the dike</td><td>Deformation of dike and rotation of cut-off wall</td><td>Wall can accommodate additional straining beyond that induced during dewatering phase.</td></tr>
<tr><td>Disruption of the single row grout curtain.</td><td>As above, by the time dilation potentially affects grout curtain, drawdown due to open pit and active dewatering system will limit increased seepage reporting to dike toe drain system.</td></tr>
<tr><td>Dilation extending upstream of the cut-off wall creating high permeability paths in bedrock that circumvent the grout curtain, and/or induce cracking in lakebed till.</td><td>Bedrock joints typically are relatively tight or infilled, though dilation might eliminate or reduce that natural defence. Erosion of till fines, if till does crack, likely to fill and plug open bedrock fractures. Internal erosion failure due to piping of till fines through bedrock joints seems highly improbable.</td></tr>
<tr><td>Shearing of cut-off wall due to differential straining along the axis of the dike due to cross-cutting faults or other major structure</td><td>Shearing of the cut-off in plan view. Sub-vertical cracking within plastic concrete diaphragm and/or jet grouted section.
Disruption of the continuity of the grout curtain, leading to increased seepage through bedrock.</td><td>The dike (and pit) will be closely monitored. Threshold levels of varying degrees of concern will be identified in advance, along with appropriate TARPs (trigger action response plans).</td></tr>
</table>

8.4.5. 2014 Deformation Analyses

To embed the CSM cut-off wall 3 m into the till will require pre-drilling and backfilling large diameter holes through the Zone 1 dike fill and into the till. As described in Section 5.3 and illustrated on Drawing 14300-41D2-1020.2, the pre-drilled holes will have a diameter of about 1180 mm and the CSM wall will have a thickness of 800 mm. The 12.5 mm crush that will be used to backfill the pre-drilled holes will not be densified; thus, zone of loose 12.5 m minus crush will remain between the completed CSM wall and the vibro-compacted Zone 1 material.

Based on discussions with specialty contractors familiar with the CSM method, the mixing energy applied by the equipment, together with the cohesionless nature of the 12.5 mm minus backfill within the pre-drill holes, is expected to result in over-building of the CSM cut-off wall width to include most if not all of the backfill. However, as this cannot be conclusively demonstrated in advance, BGC undertook additional deformation analyses for the A21 Dike to evaluate the potential effects on cut-off wall stresses and deformations of a potential loosened zone of pre-drill holes backfill around the upper portion of the cut-off wall installed via the CSM method. The purpose of the deformation analyses was to address two questions:

1. What will be the effect of this loose zone on stresses and deformations imposed on the cut-off wall, particularly during the pool dewatering phase when the majority of deformations in the dike fill and cut-off wall occur? How will these stresses and deformations compare to those for the original dike design wherein the cut-off wall excavation was undertaken by hydraulic grabs and trench cutters within vibro-densified dike core fill, and there is no such loose zone to either side?
2. Could the loose zone to either side of the cut-off wall result in stresses and/or deformations that are unacceptable in terms of the compressive and tensile strengths of the CSM wall, and/or the wall’s ductility?

The analyses addressing these questions are presented in Appendix I. The approach taken was to model cut-off wall stresses and strains both with and without the loose zone of 12.5 mm minus backfill. The analysis results indicate that the cut-off wall deformations, bending moments and shear forces would not be significantly affected by the presence of the loose zone, and the performance of the cut-off would not be compromised.

8.5. Wave Run Up and Wind Set Up Analyses

The MNWL and the MDWL derived by NKSL (1999) for the A154 and A418 dikes are held to also be valid for the design of the A21 dike. Those analyses were repeated by AMEC (2007), and are provided in Appendix J.

AMEC (2007) completed wave run-up and wind set-up analyses, specific to the location of the A21 dike within Lac de Gras, to confirm the required dike crest elevation and requirements for riprap on the upstream slope of the dike. Repeating of the NKSL analyses for these parameters was judged appropriate given the somewhat more exposed position of the A21 dike alignment within Lac de Gras relative to the A154 and A418 dikes. The analyses indicate that:

- A dike crest elevation of 421 m, as was used for the A154 and A418 dikes, is appropriate, and slightly more conservative, for use on the A21 dike
- The riprap design determined for the A154 dike is appropriate for the A21 dike.

The 2012 alignment shift is immaterial to these analyses, which remain valid as the design basis for the A21 dike.

8.6. Thermal Analyses

8.6.1. NKSL Analyses

Thermal analyses were undertaken by NKSL (1999) in support of the design of the A154 dike. No additional analyses were undertaken in support of the A418 dike design, as NKSL (2004) considered the A154 analyses to be sufficiently representative of the A418 dike design. The objectives of the NKSL (1999) analyses were as follows:

- Evaluate the risk of frost heave in till foundation
- Evaluate the freeze-thaw depth
- Determine the thermal protection needed for till foundation soils
- Determine the pattern of thermosyphon groups at the dike abutments
- Evaluate the impact of dike construction and seepage on permafrost.

8.6.2. AMEC (2007) Analyses

Two-dimensional thermal analyses were carried out for the A21 Dike abutment and on-land sections by AMEC (2007), to confirm the designs for:

- The abutment thermosyphon groups (see Drawings 14300-41D2-1015, 1017 and 1018), the functions of which are to cool warm permafrost at the permafrost-talik contact, and to extend permafrost into the talik to obtain an overlap with the cut-off wall
- The on-land and shallow water portions of the dike where the seepage cutoff is to be formed by the CSM (see Drawing 14300-41D2-1008) embedded into permafrost to a depth that freezes and will remain frozen.

The thermysphon analyses carried out by AMEC (2007) are provided in Appendix K, and confirm that the proposed pattern for thermosyphons at the abutment sections is viable to form a continuous solid frozen wall within five months of active freezing. The thermosyphon banks should therefore be installed and activated in August or September 2015 to allow dewatering to commence in January or February. For the A154 and A418 dikes, for which pool dewatering commenced in the early fall, the thermosyphon groups had to be activated in the early spring. Several months of active operation is required to achieve target abutment temperatures before pool dewatering commences.

The pattern proposed by AMEC (2007) consists of two rows of thermosyphons that are 5 m apart. Spacing between the thermosyphons are 4 m on the cold side and 3 m on the warm (talik) side of the inferred permafrost boundary. Seven pairs of thermosyphons (with an additional outside pair at the permafrost-talik contact, see Drawings referenced above) are installed to create the artificially frozen transition between the natural permafrost and the talik.

8.6.3. Updated Thermal Analyses

8.6.3.1. General

Additional thermal modeling was undertaken by BGC in support of this design update for the A21 dike. The design aspects for which thermal modeling was updated, and the reasons for the updates, are as listed in Table 8-13. In all three aspects, the analyses were undertaken to optimize thermal designs for the A21 dike.

In addition, thermal modeling of the south abutment thermosyphon group was carried out to assess the shortened active freezing time of 2 months that is available on this abutment based on the construction schedule shown in Figure 6-1. A review of the monitored performance of past thermosyphon installations at Diavik found the north abutment thermosyphon group to be in accordance with design requirements.

Table 8-13. Summary of thermal modeling undertaken in support of design update.

Thermal Design Aspect Modeled	Reason(s) for Updating the Modeling
Thermal protection requirements for the toe berm and the buried toe drain.	Evaluate rate of permafrost aggradation below the toe berm, and measures that could be taken to protect the seepage collection toe drain from freezing for the initial years of dike operation, prior to the pit drawdown effect depressurizing the lakebed till. The alternative is a more substantial toe berm with greater insulative capacity (and at greater cost), as constructed for the A154 and A418 dikes.
Thermal protection requirements for shallow water dike sections	Determine measures necessary in shallow water dike sections where frost heave in previously unfrozen lakebed till could induce dislocations within the cut-off wall. Such areas are extensive for the A21 dike and adopting the same approach for the A154 and A418 dikes would involve a large cost associated with extruded polystyrene insulation.
Thermal protection and cut-off wall depth in abutment (on-land) sections	As above, such areas are extensive along the A21 alignment, and adopting the same approach for the A154 and A418 dikes to achieve a frozen cut-off, with no repeated freeze-thaw cycling that could damage the cut-off, would involve a significant cost associated with extruded polystyrene insulation.

The updated thermal analyses are provided in Appendix L. All of the analyses considered the thermal performance data for the A154 and A418 dikes, which is briefly summarized below.

8.6.3.2. Summary of Thermal Data from A154 and A418 Dikes

Frost Penetration of the Dike

Thermistors installed from the crests of the A154 and A418 dikes confirmed rapid cooling of the dike and the underlying foundation, as outlined below. At the abutments, where the dikes were constructed on permafrost, the ground at El. 418 m stayed frozen until after the second winter (e.g., A418 dike, thermistors T1075 or T1838), and the dike foundation was part of the permafrost. At the location of thermistor T100, the dike was built on the lake talik with initial ground temperatures of about 4°C at El. 410 m. Thermistor T100 indicates the ground to be slowly cooling, and permafrost conditions were reached during the winter of 2011/2012, i.e., six years after construction of the A418 dike (Figure 8-16).

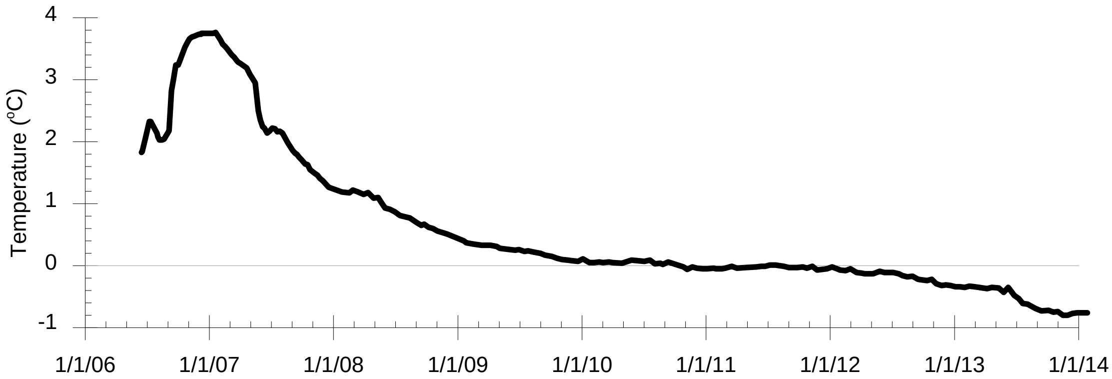
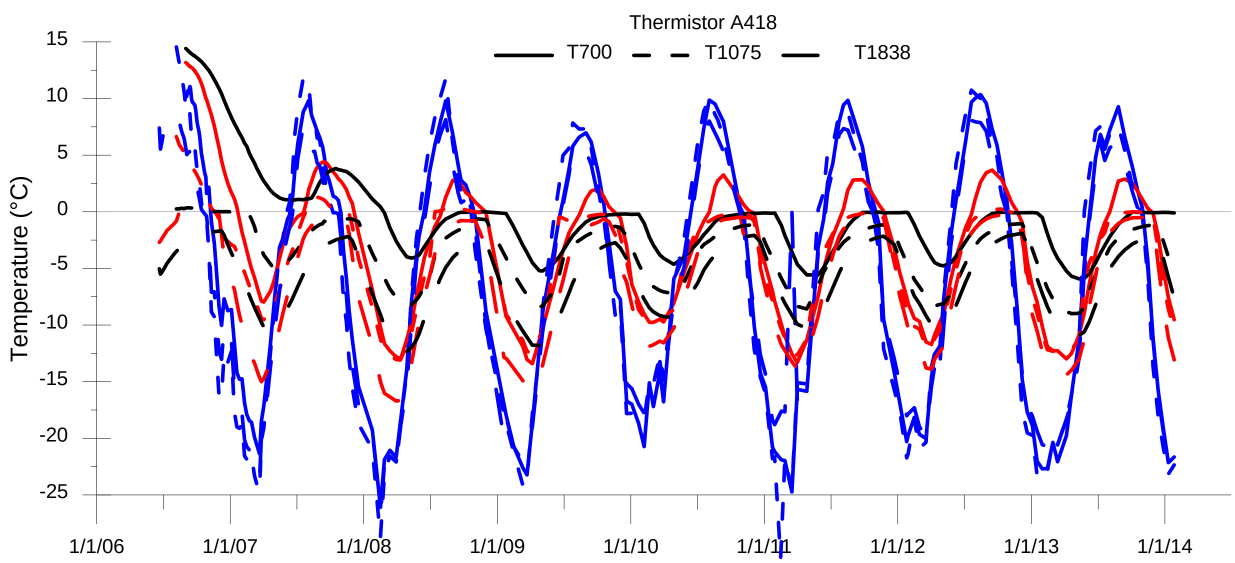


Figure 8-16. Temperature profile of thermistor T100 (A418).

Active layer thicknesses within the A418 dike are between 3 to 5 m based on the thermistor data available. The temperature records presented in Figure 8-17 show that only for thermistor T700 a zero curtain effect (temperatures stay at 0°C for an extensive period of time) exists at El. 416 m, whereas the other two locations indicate the zero curtain effect at El. 418 m.



Note: Native ground elevation at ~396 m for T700, ~419 m for T1075 and ~420 for T1838.

Figure 8-17. Shallow temperatures recorded at three locations in A418 Dike at elevations 420 m (blue), 418 (red) and 416 (black). Dike crest elevation: 421 m.

Toe Drain System

The toe berms of the A154 and A418 dikes and underlying lakebed till were instrumented with thermistors and piezometers to monitor the changes in the temperatures as well as pore water pressures with time. A detailed discussion on these data is provided in Appendix L.

Two typical temperature trends are presented in Figure 8-18. It is important to note that the thermistors are not located at the location of the toe drain, but at the bottom of the dike fill.

Therefore, the initial temperatures recorded at the beginning of the time series are likely not indicating the temperature of the toe berm material, but temperatures of the lower parts of the dike fill. This is important because winter construction of the toe berm likely resulted in much colder temperatures of the actual toe berm than the dike bottom temperatures imply. The temperature data available further indicate that despite significant toe berm fill thickness, sub-zero temperatures were reached at the level of the toe drain within three years. All thermistor data available further show a continuous cooling of the ground over time and permafrost aggradation in the former talik under the toe berm. At thermistor T084 (A154 dike), for example, the initial till temperature of the talik was 3°C when the berm was placed in November 2002. The top of the toe berm is at El. 408.9 m and the till surface at about 405.4 m. At level 404.9 m, i.e. 4 m below ground surface, positive temperatures were only recorded until the beginning of February 2003. Because of the thermistor location, it is likely that these temperatures are not representative for the toe berm, but reflect the cooling of the lower part of the actual dike, which had been in the water until the end of pool dewatering in late 2001. In March 2005, the permafrost base reached El. 402.9 m and in April 2009, it reached El. 400.9 m.

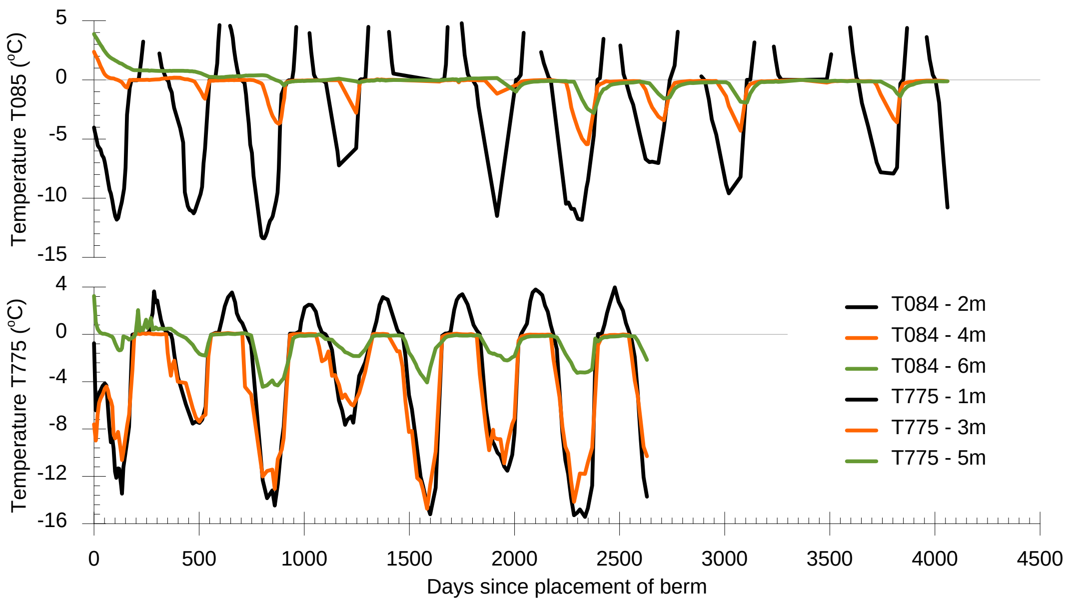


Figure 8-18 Shallow ground temperatures for T775 (A418; toe berm crest at 405.6 m) and T084 (A154; toe berm crest at 408.9 m).

Based on the toe berm thermistor data available for the A154 and A418 dikes, it is very likely that the toe drain system was only above freezing for a very short period of time, probably less than a year considering winter construction and the thick toe berm (relative to plans for the A21 dike toe berm). This configuration locked the cold into the drain system. The active layer could not penetrate to the elevation of the toe drain and the heat from the talik only marginally slowed down the advance of permafrost into the toe berm and the native ground underneath.

A representative set of piezometer readings for one of the A154 dike instrumentation study sections is presented in Figure 8-19. The readings confirm rapid depressurization of the lakebed till and bedrock downstream of the cut-off wall, in response to pool dewatering, followed by a continued, more gradual decline. The water level in the till under the toe drain immediately dropped to El. 402 m, which is below the toe drain at that location. In October 2004, the level dropped further to a level of 400 m.

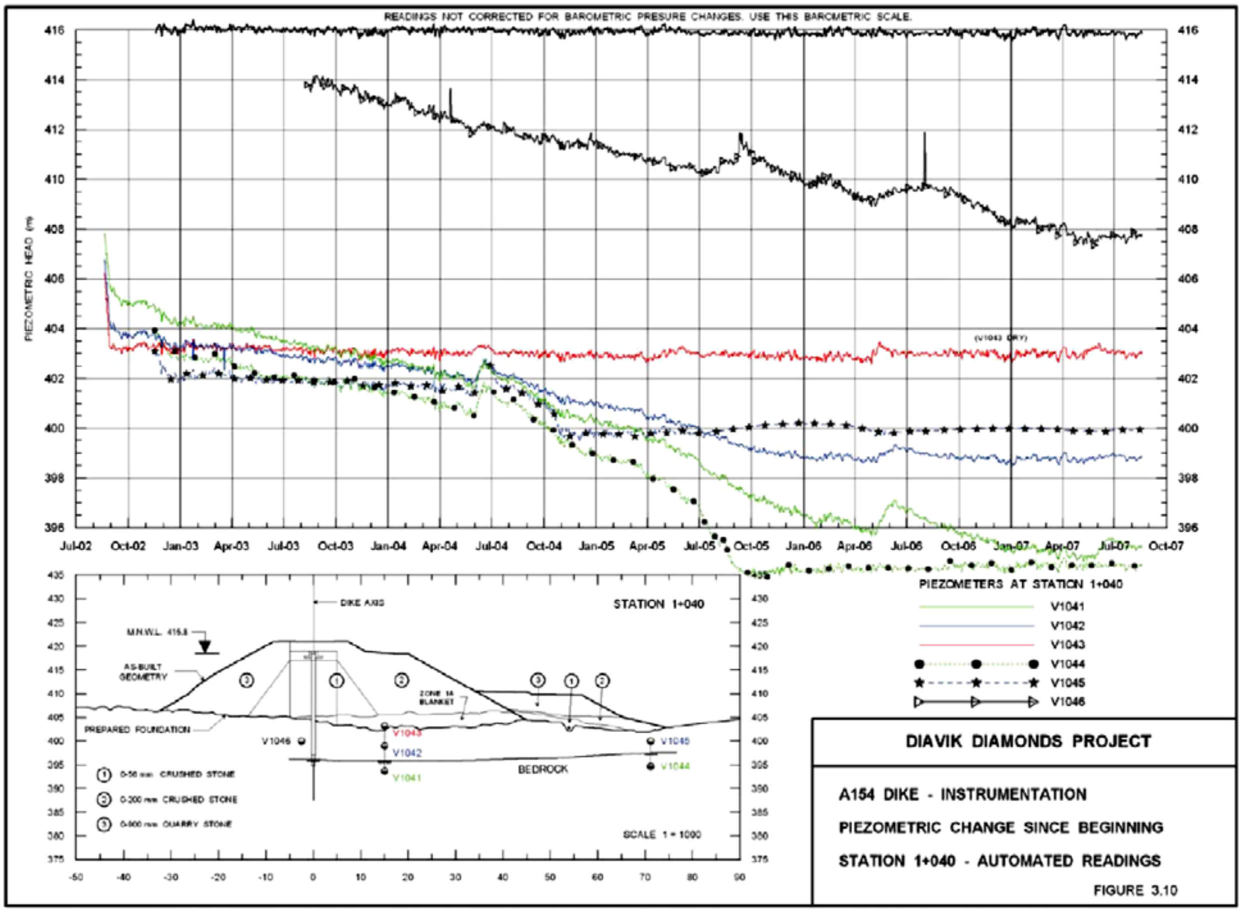


Figure 8-19. Piezometer readings at cross section 1+040 (A154).

The piezometer readings for this instrumentation study section confirmed the effectiveness of the cut-off wall and the quick response of the groundwater to the dewatering and pumping of the in-dike area. The dewatering was so efficient, that the toe drain system was not needed and its performance could not truly be evaluated in terms of quantifying seepage conveyance through the dike and the till. As noted in Section 8.3.5, 80% of the A154 dike foundation piezometers (downstream of the cut-off wall) are now dry. For both of the dikes, downward gradients exist where the piezometers are not dry, with the exception of the deepest water section of the A418 dike. There upward seepage gradients (e.g. see Figure 8-6) persist. It therefore appears that in shallower water sections of the dikes, where the need for the toe drain to limit foundation seepage pressures is questionable in any case (due to low heads), the dike foundation depressurizes relatively quickly, and the toe drain becomes redundant from the perspective of foundation seepage. In deeper water dike sections, as evidenced by the A418 dike between Sta. 600 m to 0+925 m (see Section 8.3.5), the higher toe berm and the seepage flux are sufficient to prevent freezing of the toe drain.

8.6.3.3. Modeling Methodology and Parameters

Geothermal modelling was carried out using the finite element program Temp/W, which is the heat flow module of the GeoStudio 2012 package (Geo-Slope International Ltd., Calgary, AB), widely used for modelling ground thermal regimes. To better model the surface energy balance, the "Add-In" model was used, which is based on the well-established approach presented by Hwang (1976) and was calibrated and tested. The surface energy balance was modelled using air temperature, surface albedo, solar radiation, snow depth, and evapotranspiration, and is therefore more accurate than the n-factor approach (e.g. Andersland and Ladanyi, 2004), which simplifies the complex energy fluxes at the ground surface.

The thermal design for the toe berm and the dike frost/thaw penetration were carried out using one-dimensional soil columns for the sensitivity analysis and a limited set of two-dimensional cross sections for verification. For both cases, heat flow was dominated by one-dimensional, vertical fluxes and therefore this simplification of the geometry was considered reasonable. Potential heat transfer caused by seepage was not considered during the analyses. Seepage rates in the shallow sections where freezing could be critical are expected to be extremely low considering that the cut-off wall acts as the primary flow barrier.

An overview of the climate boundary conditions and thermal material properties is given in Table 8-14 and Table 8-15 below. The material properties were adapted from previous studies (AMEC, 2012) and adjusted after a model calibration using the thermistor data available. Additional details on the geothermal modelling are provided in Appendix L.

Table 8-14. Climate parameters used for the geothermal modelling.

Month	Air Temperature (°C)	Wind Speed (km/h)	Solar Radiation (W/m^2)	Snow Depth (cm)
Jan	-27.8	16.9	4.8	27.8
Feb	-25.9	16.2	24.7	31
Mar	-24.4	18.9	88.5	34.4
Apr	-14.0	19.6	176.5	33.2
May	-4.3	18.3	238.2	13.4
Jun	7.6	17.7	248.6	0
Jul	12.5	16.4	192.4	0
Aug	10.6	18.7	152.3	0
Sep	5.7	20.3	74.1	0
Oct	-5.2	22.2	28.0	4.9
Nov	-18.5	18.1	8.9	15.9
Dec	-25.1	16.4	1.8	22.5

For the dike model, the snow depth was reduced to 30% of the actual value to account for drifting that blows snow away from the crest. This assumption is considered conservative, as the actual snow thickness is likely minimal allowing more heat extraction during winter than modelled.

Further, the following assumptions were used for the geothermal models (values based on Hwang, 1976):

- First Snow day: October 15th
- Last Snow Day: May 15th
- Greenhouse factor: 0.8
- Surface Albedo for Gravel: 0.6 (summer)/0.8 (winter)
- Surface Albedo Crushed Processed Kimberlite: 0.1 (summer)/0.8 (winter)

Table 8-15. Thermal material properties.

Material	Dry Density (kg/m^3)	Moisture (%)	Thermal Conductivity (W/m·K)		Vol. Heat Capacity ($kJ/m^3 \cdot K$)	
			Frozen	Unfrozen	Frozen	Unfrozen
Till	1620	20	1.81	1.57	2410	3160
Fill 1 (regular rock fill)	1750	7	1.37	1.25	1740	1990
Fill 2 (fine rock fill)	2200	8	1.85	1.69	1843	2194
Bedrock	2560	2	2.91		2540	2660
Crushed Processed Kimberlite	1550	7	1.04	1.27	1450	1809
Insulation (Styrofoam™ SM)	35	-	0.027		40	

Note: only frozen and unfrozen properties are provided in the table. However, the model uses transitional functions for most parameters, which are provided in Appendix L.

Additional parameters used were:

- Density of Water: 999.8 kg/m^3
- Vol. heat capacity of water: 4187 $kJ/m^3 \cdot K$
- Geothermal Gradient: 1.2°C/100 m (Golder, 2007)
- Snow thermal conductivity: 0.2 W/m·K

Initial temperature conditions were varied depending on the location of the element assessed (e.g., within permafrost on land, or on the talik). In general, two start dates were selected for the geothermal investigations:

1. The toe berms were analyzed using a start date of November 1st, assuming that dike construction and in-dike pond pumping was complete. The toe drainage system and the berm were completed during the winter. This is conservative as cold temperatures would tend to be "locked in" to the foundation as a result of that timing. The more likely case, however, is for winter dewatering, and toe berm construction in the spring, which would be a more favourable situation.
2. Dike construction in the shallows and abutments was assumed to be carried out during July. In the first year, the dike was constructed to crest El. 418 m and in July of the second year, the dike was raised to El. 421 m. Frost penetration was allowed into the dike during winter between construction years 1 and 2.

8.6.3.4. Thermal Modeling of A21 Dike Abutments

Details on the geothermal modeling and design for the A21 dike abutments are presented in Appendix L and only a summary and results from selected cases are provided here. For the thermal analysis, an initial dike crest elevation of 418 m was assumed, which was elevated to 421 m during the second summer of construction. The temperatures at the top and bottom of a 2 m thick till layer at the bottom of the dike are presented in Figure 8-20 for two initial foundation and two lakebed conditions:

A. Top of till at 413 m, permafrost foundation
B. Top of till at 416 m; permafrost condition
C. Top of till at 413 m, talik
D. Top of till at 416 m, talik

In general, it took longer for the till to freeze under talik conditions compared to permafrost conditions. However, the top of the bedrock (i.e., the bottom of the till), was predicted to remain frozen after the third winter following the dike completion to crest El. 421 m under all conditions simulated. It typically took longer for a deeper till layer (Cases *A* and *C*) and the talik situation.

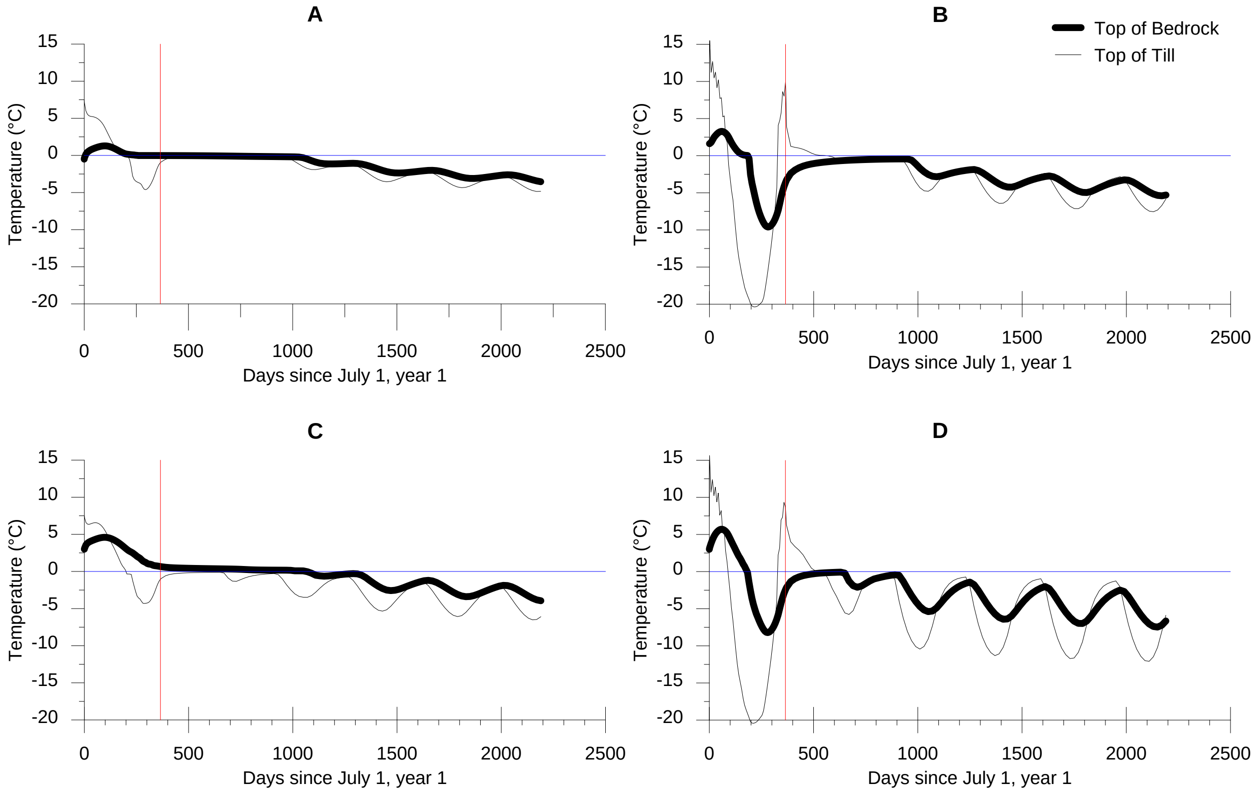


Figure 8-20. Results of geothermal model, temperatures at the top of a 2 m thick till layer and top of bedrock under the dike without polystyrene thermal insulation. On day 0, the dike is constructed to El. 418 m and after 365 days (red line), the dike is at El. 421 m.

The geothermal modelling indicated that regardless of the dike thickness and foundation conditions, permafrost is predicted to aggrade into the till layer under the abutments within three years. The modelling (Appendix L) further indicated that the addition of thermal insulation (e.g. polystyrene as done for the previous two dikes) only extended the duration of the till remaining unfrozen. Essentially, insulation would be comparable to a change from situations *B* and *D* into *A* and *C*. Because a permanently frozen till layer provides the best protection against frost heave and seepage, no thermal protection is proposed so that permafrost can form in the till as quickly as possible. Convective cooling will further be promoted by coarse, air permeable rock fill along the dike shoulders, although that effect has not been included in the modeling. Thermistors installed in the A21 dike and the foundation will allow real-time monitoring of the permafrost aggradation.

Figure 8-21 shows a cross section through the dike from the two-dimensional geothermal model, approximately at Sta. 1+550, where the dike is built on the talik. For the case shown, the talik was assumed below El. 413 m, whereas frost can form under shallow waters at the upstream toe where the lake freezes to its bottom during winter. After about 4.5 years, the talik under the upstream slope of the dike is predicted to be completely frozen, whereas freezing of the till at the location of the cut-off wall is not predicted at that time. As permafrost aggradation under the dike continues, the till section near the toe berm is expected to freeze to bedrock. The risk of generating substantial deformations due to ice lensing near the cut-off wall is therefore reduced because there is no open access to water to the cut-off wall.

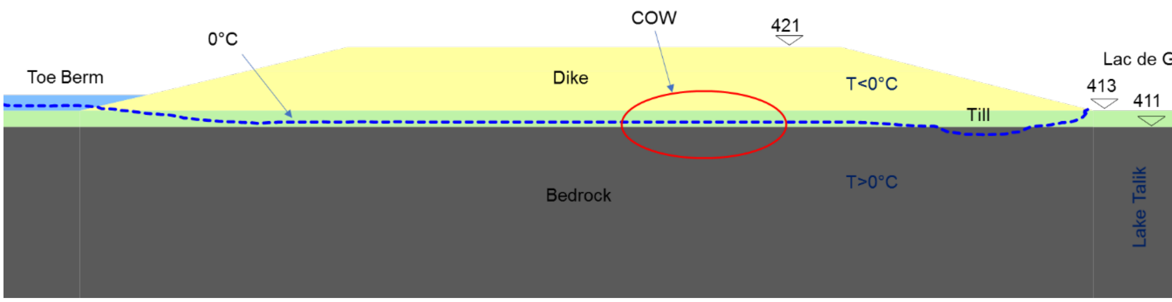


Figure 8-21. Cross section at ~ Sta. 1+550, 4.5 years after dike level at 421.

8.6.3.5. Thermal Modeling of Toe Berm and Toe Drain

The data available from the A154 and A418 dikes, as presented in Sections 8.3.5 and 8.6.3.2, reveal:

- The impracticality of keeping the toe drain unfrozen for an extended period of time
- That keeping the toe drain unfrozen is not essential for shallower water sections of the dike, given the downward seepage gradients and/or dry piezometers indicating depressurization of the dike foundation.

The major challenge associated with maintaining the toe drain in an unfrozen condition is the construction schedule, i.e. the point in time when the toe drain is installed and the berm constructed. For the A154 and A418 dikes, the toe berm was constructed in the winter months, with the pool dewatering completed in the late fall. For A21, however, with winter pool dewatering planned, the toe drain may remain unfrozen for a longer period because spring construction of the toe term could be undertaken.

As the toe berm acts as a thermal insulator and the frost penetration exceeds the thaw penetration, sub-zero ground temperatures locked around the toe drain will not thaw under regular conditions. The heat from the talik under the toe drain is not capable of extracting sufficient heat from a frozen embankment to thaw it.

AMEC (2007) proposed to use thermal insulation to compensate for the reduced toe berm thickness relative to the A154 and A418 dikes. However, the observations made at the two existing dikes, as summarized in Section 8.6.3.2 and verified using geothermal modelling, indicated that addition of insulation would only reduce the active layer thickness and further the freezing of the toe drain. The updated toe berm design therefore facilitates ground surface warming during the summer, and reduces heat extraction during winter. This can be achieved by using a dark surface (low albedo) in summer and a snow cover as a thermal insulation with a high albedo during winter. The geothermal modelling results predict that, even though winter construction traps some freezing temperatures into the toe berm during the first winter, sufficient thaw penetration can be achieved with a dark, coarse processed kimberlite (CPK) surface and a minimum 2 m thick toe berm, to keep the toe drain from freezing (Figure 8-22). During winter, the modeling indicated snow cover of 1.5 m to be required between November and March to reduce heat extraction and prevent the toe drain from freezing. The model results shown in Figure 8-22 assume unfrozen till below the toe drain (talik), which is representative for the majority of the toe drain. However, the modelling confirms that sufficient heat can penetrate during the summer months using the proposed configuration in order for the toe drain to also remain unfrozen after the first summer if the toe drain is built on permafrost. If as mentioned above the pool dewatering schedule is such that construction of the toe drain can occur in the spring, the situation would be more favourable and the measures discussed above could be reduced.

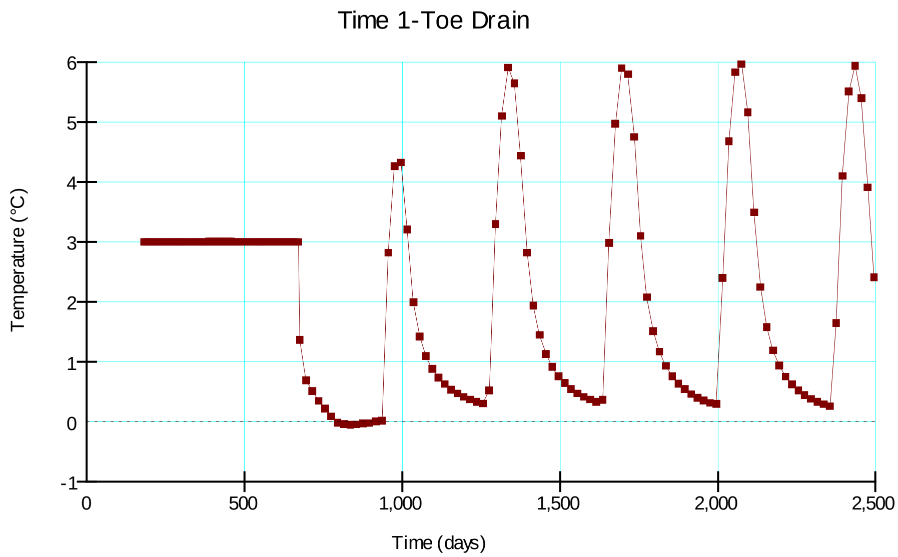


Figure 8-22. Temperatures in the toe drain based on geothermal modelling. The model assumes dark crushed processed kimberlite at the surface of a 2 m toe berm and 1.5 m snow cover during winter. Note: Day 670 represents the placement of the toe berm on November 1.

It is unknown whether the amount of snow cover, indicated to be required on the basis of the modeling, can be generated naturally using snow fences along the downstream side of the dike crest. Wind data from the weather station at Diavik (see Section 2.2 and Figure 2-2) do not indicate a single dominant wind direction. Monitoring of the snow distribution and accumulation, together with detailed ground temperature measurements in the toe berms at the location of the drains is required to help optimize the maintenance and operation procedures in the future. Since the required design life of the toe drain is, based on the depressurization of the dike foundation observed for most of the A154 and A418 dikes, only about three years, any required effort to dump snow over the dike crest to encourage drifting on the toe berm would be short-term, and targeted at those areas with minimal toe berm thickness.

8.6.4. Risk Assessment

BGC carried out a qualitative risk assessment on the changes in the dike thermal design and toe drain protection compared to the A154 and A418 dikes. Similar to operations of those existing dikes, a monitoring program, a detailed TARP, and an operation protocol will be available to identify any potential hazards before unsafe conditions arise and adequate mitigation measures can be implemented. A summary of the risks assessed and mitigation measures as presented in Appendix L is provided below:

8.6.4.1. Freezing of the Toe Drain

A completely frozen toe drain may result in elevated pore pressures that affect dike stability. High foundation pore pressures are of more concern near deeper lake sections along the dike where the toe berm is generally higher than 5 m. If pore pressures exceed safe conditions, depressurization (via the relief wells in the deeper water areas of the dike) can be initiated. However, initial ground temperatures are expected to be warmer at deeper lake bottoms as a result of the Lac de Gras talik, which reduces the likelihood of locking in cold temperatures during spring toe berm construction and freezing the toe drain. Further, failure of the toe drain to convey water does not necessarily translate into problematic pore water pressure increases in the dike foundation.

Limits for pore pressures, seepage collection and temperature will be defined and a TARP developed, similar to the existing one. Mitigation measures that can be used in the unlikely event that unacceptable conditions form are, as follows:

- Adding weight (fill) to the toe berm
- Drilling additional depressurization wells through the frozen toe berm
- Enhance pit dewatering
- Steaming of the toe drain system.

The risk of dike stability reduction due to the freezing of the toe drain is considered low.

8.6.4.2. Spring Fill Placement on Toe Berm

If toe berm construction in the spring of 2018 results in cold fill placement, there is a low potential for locking in cold temperatures such that the toe drain does not thaw during the initial summer months. The dark surface of the toe berm will enhance ground warming during summer and snow accumulation in winter will reduce heat extraction. In addition, the drain systems for the A154 and A418 dikes indicated that even if subjected to seasonal freezing in some areas, the drainage performed in an acceptable manner, despite late fall and early winter construction in both cases.

The risk of toe drain freeze-up due to potential cold fill placement for toe berm construction in the spring of 2018 is considered low.

8.6.4.3. Toe Drain Freezing in Permafrost

The sections where the toe drain is placed in permafrost are at an elevated risk for initial freezing because of the colder initial temperatures. However, these sections are the ones with the lowest hydraulic heads and if frozen, do not affect the performance of the toe drain as much as where the hydraulic head is higher.

The risk of toe drain freeze-up in the permafrost sections is considered low.

8.6.4.4. Inadequate Snow Fence Performance

The snow fences may not capture sufficient snow, resulting in colder than anticipated toe berm temperatures in the winter months, and more rapid aggradation of permafrost. If the monitored ground temperatures indicate an elevated risk of toe berm freeze-up, extra snow could be hauled from other locations and strategically placed in areas of concern, and/or additional snow fencing could be constructed.

The risk of toe drain freezing due to inadequate snow fence performance is considered low.

8.6.4.5. Inadequate CPK Performance

Dust, weathering and/or mechanical breakdown may reduce the warming capacity of the dark CPK surface with time. Mitigation measures may include the placement of additional CPK should monitoring data indicate this to be required. Lack of summer ground warming can further be compensated by an increase in snow cover the following winter. Temperature monitoring will provide information on the effectiveness of the system.

The risk of the toe berm reaching colder than anticipated temperatures during the summer due to inadequate CPK performance is considered low.

8.6.4.6. Thermosyphon Performance

The two thermosyphon groups on the north and south portions of the dike are installed to provide a cold, frozen interface between the permafrost sections and the unfrozen side and complement the jet grouting and the grout curtain, providing a barrier against seepage. Ground temperature and seepage measurements are an integral part of the performance monitoring. As the thermosyphons can be run in active mode, active ground cooling can be initiated if required should heat extraction from the passive mode prove insufficient. Further, additional thermosyphons can be installed at any time to increase the heat extraction if required.

The risk of inadequate thermosyphon performance is considered low.

8.6.4.7. Frost Heave

Frost heave is expected to occur near the thermosyphon groups on the north and south abutments similar to observations at the existing dikes. The heave is indicative of the cooling from the thermospyhons and will be monitored to identify any potential hazards.

The risk of frost heave jeopardizing the integrity of the dike is considered low.

9.0 MATERIAL QUANTITY TAKEOFFS: A21 DIKE

Material quantities were estimated for the various components of the A21 Dike. The embankment quantities where calculated based on the digital terrain models provided by DDMI and a digital dike model created using Civil3D. Hand calculation checks were used as necessary. Some quantities were generated from drawings or cross sections. The quantities listed in Table 9-1 are based on neat line quantities and do not include contingencies, or other allowances for factors such as crushing or waste/stockpile loss factors. Detailed material quantities are provided in Appendix O which includes waste and overbuild factors for the DDMI crushed material.

Material quantity estimates were not completed for water management systems, electrical or mechanical components and will be completed by others under separate cover.

Table 9-1. Material quantity estimates for A21 dike construction.

Item		Estimated Quantity
Turbidity Barrier, Foundation Dredging and Excavations		
Turbidity barrier		24,000 m^2
Dredging of lakebed sediments		116,600 m^2
Boulder removal	Via clam shell	27,300 m^2
	Via excavator	30,400 m^2
	Total	*57,700 m^3*
Toe drain excavation		18,300 m^3
Pre-drilling through Zone 1 material and removal in support of CSM (diameter of 1.18 m)		28,400 m^3
Infield Excavations	Lakebed sediments	34,800 m^3
	Lakebed Till	900 m^3
	Drainage Ditch	800 m^3
	Total	*36,500 m^3*
Embankment Fills		
Zone 1 (wet, end dump into lake)		306,600 m^3
Zone 1A (clamshell)		42,200 m^3
Zone 1B (Dry)	Placed between El. 417 m - 418 m	20,200 m^3
	Vibro-densification backfill to fill depressions	22,400 m^3
	Dike crest road surfacing	5,000 m^3
	Toe berm	26,400 m^3
	Total	*74,000 m^3*
Zone 1C (Clamshell)		18,200 m^3
Zone 2	Wet, end dump into lake	310,600 m^3
	Dry, placed between El. 417 m - 418 m	33,000 m^3
	Dry, placed above El. 418 m	150,700 m^3
	Dry, toe berm	79,400 m^3
	DPS containment berm surfacing	600 m^3
	Total	*574,300 m^3*
Zone 3	Wet, end dump into lake	328,800 m^3
	Dry, placed between El. 417 m - 418 m	18,200 m^3
	Dry, placed above El. 418 m	48,000 m^3
	Upstream buttress @ 0+310 m (Clamshell)	7,200 m^3
	Upstream buttress @ 0+745 m (Clamshell)	17,000 m^3
	Total	*419,200 m^3*
Zone 4 (Till)	Infield grading backfill	5,600 m^3
	DPS containment berm	1,700 m^3
	Total	*7,300 m^3*
Zone 6 (filter material surrounding perforated pipe in toe drain)		800 m^3
Run of Mine (ROM)	Toe area access road during toe berm construction phase	40,500 m^3
	Infield grading backfill	13,000 m^3
	Total	*53,500 m^3*
Coarse Processed Kimberlite (toe berm surfacing)		13,000 m^3
50 mm minus (Toe berm road surfacing)		3,500 m^3
12.5 mm minus crush (Pre-drilling backfill)		28,400 m^3
Cut off Wall Components		
Cut-off wall (CSM method from Sta. 0+025 m to 1+823 m, 3 m into lakebed till)		21,600 m^2
Cut-off wall (Trench method from Sta. 1+823 m to 2+050)		890 m^2
Concrete for guide wall		450 m^3
Jet Grouting (Sta. 0+025 m to 1+823 m, overlapping CSM wall by 1 m and grout curtain by 1.5 m)		7,100 m^2
Grout Curtain (Sta. 0+040 m to 1+640 m, 15 m into bedrock + 0+525 m to 0+850 m and 1+325 m to 1+500 m, 25 m into bedrock)		29,000 m^2

10.0 INSTRUMENTATION AND MONITORING

10.1. General

Geotechnical instrumentation is required to monitor the behaviour of the dike and its foundation during the construction and dewatering phases as well as during subsequent normal operation. It is intended to provide the following vital information:

- Early confirmation that the performance of the fill embankment and foundation during the various phases of interest is consistent with the predictions made during the design studies, notably in stability, deformation, seepage and thermal analyses.
- Early warning of the development of potentially adverse trends such as excessive pore water pressure, seepage and/or deformation.

Collected data can be used to identify conditions and areas requiring remedial measures and to determine the nature and extent of corrective work. Several types of instruments have been selected to collect the required information, consistent with the monitoring installed for the A154 and A418 dikes.

- Vibrating wire piezometers
- Thermistors
- Inclinometers
- Survey markers
- Flowmeters.

An automatic field data acquisition system will allow the collection of the data from the vibrating wire piezometers and possibly some thermistors. Flow meter readings will be transmitted to the Control Room along with other pump station operating data using radio modem.

Instrumentation drawings for the 2007 A21 dike were prepared by AMEC (2007), and are as listed below:

- Drawings 14300-41D2-1026.1 through 1026.3: plan locations of proposed instrumentation
- Drawing 14300-41D2-1027: list of instrumentation
- Drawings 14300-41D2-1028.1 through 1028.7: dike cross sections showing instrumentation locations.

10.2. Piezometers

Piezometers will be installed for the following purposes:

- Prior to dewatering: measurement of excess pore water pressures which may have developed in foundation soils following fill placement.
- During dewatering: measurement of pore water pressure changes in the dike foundation material while drainage takes place simultaneously with water level

drawdown on the pool side. This will provide invaluable information as to the ongoing consolidation process and the possible need to control the dewatering rate, as well as the occurrence of local seepage concentrations through the dike fill or foundation.

- Immediately after dewatering: evaluation of the efficiency of the cut-off by measurement of seepage uplift and gradients in the downstream part of the foundation as well as evaluation of the efficiency of the drainage system installed at the downstream toe of the dike.
- Monitoring response to pit blasting operations: piezometers installed in the lakebed till, primarily downstream of the cut-off wall, and within Zone 1 fill upstream of the cut-off wall, will monitor transient pore pressure response to blasting which, along with monitoring of peak particle velocity (PPV) on the dike, will be used to adjust blasting practices as needed to maintain the safety of the dike.
- In the long term: monitoring of variations in pore water pressure and groundwater gradients under the dike and near its downstream toe, simultaneously with permafrost penetration into the dike fill and foundation and open pit deepening. Piezometers installed within lakebed till and bedrock upstream of the cut-off wall will be used to evaluate the development of downward gradients associated with the pit-drawdown effect and the pit slope depressurization system.

After the dike crest has reached the level 421 m, and before pool dewatering commences, vibrating wire piezometers will be installed from the dike fill at its upstream and downstream limits of the dike crest. These piezometers are to be spaced at intervals of approximately 100 to 200 m along the dike alignment. After substantial completion of pool dewatering and construction of the toe berm, another set of vibrating wire piezometers will be installed near the toe of the toe berm. The toe berm piezometers will be spaced with an interval of approximately 50 to 100 m. The piezometers will be located in the dike filter blanket, the overburden foundation and the upper bedrock zone. When the thickness of the lakebed till is greater than 6 m, two piezometers will be installed in the till. Up to four vibrating wire piezometers will be installed per borehole. Each piezometer will comprise a single cable and tip.

10.3. Thermistor Cables

Thermistors will be installed in the following areas:

- Dike abutment areas and on-land dike segments: to measure ground temperature in the frozen foundation soils and in the upper bedrock zone after dewatering has been carried out in order to monitor the long term aggradation/degradation of permafrost.
- Within the abutment thermosyphon groups: to evaluate the efficiency and capability of thermosyphons to maintain stable conditions over the life of the A21 dike, and to guide thermosyphon operation in active versus passive mode.
- In the toe berm: to monitor the frost penetration in the toe berm and provide a warning of potential freeze-up of the toe drain, or if concentrated seepage flow, as

evidenced by temperature anomalies indicative of convective heat flux, through and/or below the dike, is present.

- In the cut-off wall: appended to inclinometer installations, these thermistors are to monitor temperature profiles within the higher portions of the cut-off wall, for correlation of any potentially anomalous inclinometer deflections, which could indicate an increased likelihood of cracking, to temperature anomalies that could be indicative of convective heat flux associated with seepage through the cut-off.
- Lakebed dike segments: to monitor long term permafrost ingress in the dike fill and foundation and provide supporting information to the vibrating wire piezometers in determining the effectiveness of the cut-off wall at limiting flow through and below the dike.

Thermistors in the abutment areas will be installed to a depth of 26 m from the dike surface and will have bead spacing configuration D, as listed in Table 10-1.

Eight thermistors will be installed within each of the abutment thermosyphon groups as shown on Drawings 14300-41D2-1026.1 and 14300-41D2-1026.3. Single thermistors will be placed on the dike reference line (cut-off wall alignment) while pairs will each be offset 1.5 m from the reference line. These thermosyphons will use bead configuration A, as outlined in Table 10-1.

The toe berm will be monitored by fourteen thermistor stings using the bead configuration B (Table 10-1). Each string will be installed to a depth of 30 m below the toe berm surface.

Thermistor strings will be installed to a depth of 45 m from the dike surface where the dike is in the lake. The thermistors will have bead configuration C as shown in Table 10-1.

In some cases thermistors will be installed immediately adjacent to inclinometers. Where thermistor strings are paired with inclinometers installed within the cut-off wall, these will extend 3 m into bedrock. Because the depth from the top of the dike to the bedrock varies, each thermistor will have a custom bead spacing to make full use of the available sixteen beads. The custom bead spacing as well as tabulated installation details for all thermistors can be found on Drawing 14300-41D2-1027.2.

Thermistor readings will also be used to validate or calibrate the thermal analyses done for various parts of the dike. This will allow more accurate projection of future dike thermal behaviour thus allowing better operation and longer advance warning of potential problems that might warrant proactive remediation.

Thermistor cables installed during previous and future (see Section 14.4) site investigations, conducted prior to dike construction, will be extended through dike fill to make full use of monitoring instruments.

Table 10-1. Standard thermistor bead configurations.

Beading Spacing Configurations																
Bead Configuration	Bead Depth Below Dike Surface															
	Bead 1	Bead 2	Bead 3	Bead 4	Bead 5	Bead 6	Bead 7	Bead 8	Bead 9	Bead 10	Bead 11	Bead 12	Bead 13	Bead 14	Bead 15	Bead 16
A	0.8	1.3	2	4	7	10	13	16	19	22	25	28	31	35	38	41
B	0.8	1.1	1.5	2	3	4	5	6	8	10	12.5	15	17.5	20	25	30
C	0.8	1.3	2	4	6	8	10	12	14	16	18	20	25	30	35	40
D	0.8	1.3	2	3	4	5	6	7	8	9	10	12	14	18	22	26

10.4. Inclinometers

Inclinometers will be used to measure the magnitude and rate of horizontal deformations during dewatering and over the life span of the dike. They will be drilled from the dike crest in the center of the cut-off wall at about 100 to 200 m intervals, in relatively higher sections of the dike. All inclinometers will be installed to a depth of 3 m into bedrock.

10.5. Survey Markers

Survey markers will be installed at about 25 m intervals on the dike crest, at the upstream and downstream limits, and on the toe berm, to measure the vertical and lateral movements associated with fill settlement and lakebed foundation soils consolidation during dewatering and in the long term.

At three locations, one on Island A and two in the on-land portion of the south abutment, survey markers will be placed directly on top of the cut-off wall. These markers will be installed after the dike construction is complete. A hole will be drill from the dike surface to a depth of 0.5 m into the cut-off wall. A metal bar, with a 100 mm diameter plate welded to the base of the metal bar to prevent pullout, will be placed in the hole and grouted so that the top of the pin is just below the dike surface. A cover will then be placed over the marker, flush with the road surface to protect the pin.

10.6. Measurement of Flows

As part of the water handling system, flows will be measured in each pumping station to monitor the total volume of flows associated with seepage across the dike and its foundation, and with runoff from the dike and between the dike toe and the open pit perimeter.

Flows into and out of each of the three pumping stations will be monitored. As shown on Drawing 14300-41D2-1022, the pump stations are located along the toe drainage trench of the dike, allowing the perforated pipe of the toe drainage trench to discharge directly into the sumps of the pumping stations. A small water intake, located at the downstream toe of the dike, will collect the runoff which will be discharged into each pumping station sump through an HDPE pipe embedded in the dike fill. Each pipe from the toe drain and downstream toe will discharge into a small weir equipped with an ultrasonic water level measuring probe in the pumping station sumps, by which flow rates will be monitored and continuously transmitted to the Control Room. This is the same system implemented for the A154 and A418 dikes. The design of the pump stations and associated monitoring is being undertaken by AECOM.

The total flow from the outlet pipe in each pumping station will be measured by means of a meter installed on the circuit breaker of each pump that will record the operation hours of the pumps, coupled with a gauge to record the pressure in the outlet pipe. The combination of the hour meter and pressure gauge will permit calculation of the total volume of flow from each pumping station, including the seepage and runoff water components.

10.7. Seismograph

Blast monitoring will be carried out using portable seismographs that will measure blasting induced velocities and accelerations at the crest and toe of the dike. Monitoring locations will be determined by pit development and where blasting will be taking place.

10.8. Automated Data Acquisition System

The vibrating wire piezometers, thermistor cables adjacent to thermosyphons, and flowmeters will be connected to the automatic data acquisition system. The acquisition system will collect the data and transmit it to the Control Room where it will be available at any time.

A complete system will require the installation of data loggers along the length of the dike, each data logger being connected with buried cables to a maximum of 80 sensors or instruments. The data collected by each of the terminals will be transmitted to the monitoring station, via the fibre optic cable installed along the crest of the dike. Each data logger terminal will require electrical and heating supply to keep the installation at a temperature greater than - 20°C.

11.0 MONITORING OF A21 DIKE-PIT INTERACTION

11.1. General

Development of the A21 ore body, and optimization of both the tonnage of ore recoverable via open pit mining methods and dike construction costs, has necessitated two significant changes relative to the A154 and A418 open pits:

- Steeper pit slopes
- Reduced setback between the A21 dike toe and the ultimate pit rim.

Monitoring of pit-dike interaction, particularly the effects of dilation of the open pit, will be an important component of dike monitoring. This will likely involve survey markers between the dike and the open pit. The system to be implemented will be developed in conjunction with the open pit engineers prior to completion of the dike. This will also include development of an Operations, Maintenance and Surveillance (OMS) manual specific to the A21 dike. The manual will be based largely on those developed by DDMI for the A154 and A418 dikes.

As a result of the reduced dike-pit setback (see Section 3.8), the issue of dike-pit interaction, especially the potential deformation to which the dike and the seepage cut-off could be subjected, will warrant a greater of degree of scrutiny than was the case for the A418/A154 pits and dikes. That scrutiny included the joint UDEC-FLAC modeling carried out by Golder and AMEC (see Sections 8.4.4.3 and 8.4.4.4) to predict pit dilation effects on the A21 dike.

Golder (2012) describes the planned monitoring system for the A21 pit:

- Surface and boring extensometers, inclinometers and time domain reflectometry (TDR) cables
- Vibrating wire piezometers
- Fracture shear displacement meters
- GeoMos automated prism monitoring system
- Prismless laser scanners (Riegl LPM-2K) or LIDAR with reflectors
- Slope stability radar.

11.2. Time Available for Monitoring and Evaluation

There are a number of factors that serve to mitigate dike-pit interaction risks related to time, as outlined below.

1. Monitoring of pit dilations from the start of excavation to compare against model predictions, and identify more “active” areas along the pit walls. The use of robotic total station surveys and radar will greatly facilitate this.
2. Mapping of geologic structure on a bench by bench basis in the pit to evaluate deviations from design analyses, and to correlate against monitored pit wall performance.

3. The minimum required factor of safety against pit wall failure geometries extending to the dike, 1.5, is not approached until the final year or two of A21 pit excavation.
4. Enhancement of the pit walls depressurization system if and as required prior to approaching the final pit shell.
5. Correlation of pit monitoring data and dike monitoring data, and review and adjustment of the monitoring programs as required based on the observed performances.
6. Pit groundwater drawdown effects due to the deepening pit, and the pit slopes depressurization system (groundwater well fields) become more pronounced when dilation effects become more marked, thereby reducing potential for upwelling of seepage below the downstream shell of the dike. As evidenced by the performance of the A154 pit-dike system, the groundwater drawdown effect caused by the pit excavation essentially "drains" the lakebed foundation of the dike to the downstream of the cut-off wall. A portion of the lakebed foundation for the A418 dike has not been similarly drained, although seepage gradients are sufficiently low as to be of no concern.

In summary, there will be ample time to better understand dike-pit interaction issues prior to the pit approaching its final configuration at which time the factor of safety criteria are being approached. In the preceding years, the factors of safety as determined from the pit slope stability analyses undertaken by Golder (2012) are predicted to be in excess of design requirements.

11.3. Monitoring of Dilation from Pit Walls to Dike

Direct and indirect means available for monitoring of dilation between the pit walls and the dike include:

- Survey pins on the dike crest (top of cutoff wall, at 30 m intervals)
- Inclinometers installed within the cutoff wall
- Deep inclinometers installed at dike toe, and between dike toe and open pit rim
- Survey markers (anchored to bedrock) installed between dike toe and pit rim
- Horizontal extensometers installed from successive pit benches as the pit is deepened (once the open pit reaches final wall)
- Survey prisms within the open pit (likely to be installed at 30 m intervals along the crest and on all benches).

Detailed geologic mapping of the pit walls as they are exposed will be important, and this is described by Golder (2012) in some detail. Any significant features identified that could represent areas of concentrated differential movement cutting across the dike axis would trigger installation of additional monitoring capability, and/or more frequent monitoring, in that particular area.

11.4. Monitoring of Changes in Seepage Conditions

The potential consequences of excessive pit dilation leading to damage to the cut-off wall, and/or to significant dilation of the fractured bedrock, would be increased seepage through the dike. It will be important to detect at an early stage any changes in piezometric and seepage conditions that could be attributable to the effects of pit walls dilation. Besides the piezometers and thermistors installed below the dike crest and toe berm, and the seepage collection system below the toe berm, the following will be implemented as necessary based on interpretation of piezometer and thermistor data:

- Additional piezometers and thermistors between the dike toe berm and pit rim, installed upon completion of dewatering, will be considered if the currently planned piezometers indicate significant heads and gradients post-dewatering.
- Installation of additional piezometers into bedrock to the upstream side of the cut-off wall may be undertaken to better evaluate the performance of the pit slope depressurization system, and/or upstream of any particularly wet areas within the pit walls. Such piezometers could be useful in detection of any changes in piezometric conditions that could potentially be attributed to dilation of the bedrock and increasing secondary hydraulic conductivity.
- Hydraulically conductive structures that are exposed in the open pit walls that may be continuous below the dike will be mapped and monitored.

The potential utility of periodic geophysical surveys of the dike and the area between the dike and the open pit rim will be evaluated. The use of geophysical methods to detect zones of concentrated leakage in water retaining structures is becoming increasingly commonplace. An increasing number of firms specialize in providing this type of service. It may be of benefit to conduct such surveys up to several times per year on the dike, along the toe berm of the dike, and possibly along the upstream (either via boat or from the ice). If there are areas of concentrated seepage, such surveys might be capable of identifying such areas, which would enable more detailed localized monitoring as a first step, followed up by remedial measures, if deemed necessary.

11.5. Pit Blasting and Peak Particle Velocity Limit Criteria

Given the 50 m setback criterion adopted for A21, controlled blasting to limit peak particle velocity (PPV) at the dike will be required. Common threshold vibration levels for damage have been established relating PPV to potential vibration damage. Table 11-1 summarizes additional threshold damage levels commonly used in urban settings in evaluation of the potential for blast-induced damage.

Table 11-1. PPV threshold damage levels.

PPV (mm/sec)	Damage
3 - 5	Vibrations perceptible
10	Approximate limit for poorly constructed and historic buildings
33 - 50	Vibrations objectionable
50	Limit below which risk of damage to structures is very low (< 5%)
125	Minor damage, cracking of plaster, serious complaints
230	Cracks in concrete blocks
300	Rock falls in unlined tunnel
380	Horizontal offset in cased drill holes
635	Onset of cracking in rock
1000	Shafts misaligned in pumps, compressors
1500	Pre-fabricated metal buildings on concrete pads, metal twisted and concrete cracked
2500	Breakage of rock

The criteria given in Table 11-2 were proposed by Charlie et al. (1987) for blasting near dams, based on the potential for liquefaction in either the dam fills or the foundation soils.

Table 11-2. PPV guidelines for blasting near dams.

Liquefaction Susceptibility	PPV Limit (mm/sec)
Dams constructed of or having foundation materials consisting of loose sand or silts that are sensitive to vibration.	25
Dams having medium dense sand or silts within the dam or foundation materials.	50
Dams having materials insensitive to vibrations in the dam or foundation materials.	100

The lakebed tills and firm sediments underlying the dike are considered to be of suitable relative density and liquefaction resistance that the 50 mm/sec PPV limit measured at the crest of the dike via portable seismometers is deemed appropriate and conservative.

Given the reduced dike-pit setback for A21, appropriate blast designs and monitoring will be required to achieve the PPV criterion, and to achieve blast frequencies that avoid dike resonance. DDMI has good experience from the A418 and A154 pit blasting to apply for A21. As has been the practice for the A418 and A154 dikes, lakebed till pore pressure response to blasting should be monitored along with PPV's as monitored on the dike crest. If pore pressure responses monitored in the A21 dike foundation are significantly higher, and more persistent, than was the case for the A418 and A154 dikes, the PPV limit should be reviewed and, if warranted, reduced.

Experience for the A418 and A154 dikes indicated transient pore pressure response in piezometers installed within the lakebed tills in response to pit blasting operations. The observed piezometric response was inconsistent, with some piezometers indicating negative pore pressure response (i.e. a decrease in pore pressure), and others a positive pore pressure response. In some instances, increased pore pressures would persist for some time following the blast. As the A21 pit is developed, presumably in a single stage (with no push-backs), blasting will become further removed from the dike due to the depth of the pit. Experience with the A154 and A418 dikes demonstrates that, due to the groundwater sink effect of the deepening open pits, the phreatic level (and thus pore pressures) within the lakebed till to the pit side of the dike cut-off wall declines, with a number of piezometers indicating de-saturation and permafrost aggradation. Therefore, concerns associated with blast vibrations below the dikes are likely to decrease with time, provided that good, controlled blasting practices are maintained.

12.0 LONG TERM WATER HANDLING

12.1. General

The long term water management system at the downstream toe of the A21 dike will comprise a toe drain (perforated, non-heat-traced pipe in a gravel drain/filter) with three pumping stations. Seepage and runoff will report to the pumping stations via existing lakebed topography, augmented by re-grading as necessary to facilitate drainage towards those stations.

Sources of water reporting to the pumping stations will comprise:

- Seepage through the dike and foundation soils, which will be collected by the toe drain and conveyed by gravity to one of the three dike pumping stations (DPS).
- Surface water runoff which will report to the pumping stations from the drainage area confined between the downstream toe of the dike and the pit perimeter.

Based on the topography of the area between the dike and proposed pit rim, three catchment areas were delineated which would direct both the toe drain water and surface water to the low point of each catchment. A DPS will be installed at the low point of each catchment area, which will pump the collected water to the main A21 pump skid (located near the access ramp where it exits the pit on the west side) and ultimately to Pond 3, at the northwest side of East Island. The pumping stations are being designed by others, but will be essentially the same as the stations installed for the A154 and A418 dikes. Photographs of the pumping stations, and associated ponds, for the previous dikes are shown in Figure 12-1.

For the A154 and A418 pits, a nominal minimum setback from the toe of the dike to the open pit rim of 100 m was established. For the A21 pit-dike system, the setback has been reduced to 50 m, driven by project economics. Section 3.8 describes the setback and is shown schematically on Figure 3-2.

The result of the reduced dike-pit setback criterion for A21 is reduced catchment for runoff flows, but also a greater constraint on grading and drainage measures to route seepage and runoff flows to the DPS locations. Moreover, the open pit geotechnical design incorporates groundwater well fields for pit slope depressurization (see Drawing 14300-41D2-1021) that must be accommodated within the infield area.

DPS on toe berm of A154 dike.

DPS in A154 infield.

Panorama of A154 dike infield area - arrow indicates DPS sump. Pond adjacent to the sump is dry. Infield area between dike and pit rim in A154 is more extensive than is the case for A21.

DPS in the deepest section of the A418 dike.

Figure 12-1. Photographs of DPS stations at the A154/A418 dikes.

The following sections present the inflow estimates for seepage and surface water.

12.2. Surface Runoff and Seepage through Dike and Foundations Soils

12.2.1. Seepage Inflow

Seepage through the dike will be collected by the toe drain and directed to the three pump stations (DPS-7, DPS-8 and DPS-9, see Drawings 14300-41D2-1021 and 1022). The toe drain consists of a drainage trench with an embedded perforated pipe in a gravel drain/filter located at the downstream toe of the main dike embankment (i.e. not at the toe of the toe berm). The toe drain system is connected to the DPS system so that seepage water can be collected, then pumped away from the toe of the dike. The dike layout has not changed from the AMEC (2012) layout, therefore no modifications of the total seepage inflow estimate have been made relative to the 2012 alignment.

The design seepage rate presented by AMEC (2007, 2012) was based on maximum seepage rates estimated from their seepage analyses for the A418 and A154 dikes. Actual seepage rates have been substantially lower on the basis of the pumping records for the DPS system.

AMEC (2007, 2012) indicated that modeled seepage rates vary as a function of the dike height with low head conditions (6 m) having a predicted seepage rate of 0.25 litres/min/m of dike length and high head conditions (28 m) having a predicted seepage rate of 1.5 litres/min/m. The maximum predicted seepage is 3.85 litres/min/m for a head of 28 m (foundation at elevation 388 m), which includes a factor of safety of about 2.5. The total seepage inflow for the three A21 pump stations, as estimated by AMEC (2012), is presented in Table 12-1.

Table 12-1. Estimated seepage inflows for each DPS (AMEC, 2012).

Pump Station	Seepage Rate	
	(litres/min)	(litres/s)
DPS-7	246.7	4.1
DPS-8	327.5	5.5
DPS-9	1,374.1	22.9

It is important to note that the modeled seepage rates consider the dike in isolation, and do not account for the draw-down and depressurization effect of the groundwater sink represented by the open pit. The pit draw-down effect has been very evident in both piezometer and thermistor data for the A154 and A418 dikes, both upstream and downstream of the cut-off walls, as discussed in Section 8.3.5. Monitoring of those two dikes indicates that dike seepage, which was lower than design basis predictions before open pit sinking, decreased further with open pit sinking. For A21, the draw-down effect can be expected to occur more rapidly than for the previous pits, for the following reasons:

- Reduced set-back between the A21 pit and the A21 dike
- The A21 pit development will not be phased with push-backs. Instead, the pit will be developed to its ultimate footprint almost immediately
- The pit depressurization system designed by Golder (2012) is likely to further accelerate the draw-down effect.

Off-setting these considerations, the rate of A21 pit sinking will likely be slower than for the previous pits, and the depth of the pit is reduced relative to A154 and A416. Per Golder (2012), the anticipated rate of pit sinking, defined as the base of the pit relative to lake level (nominal El. 415 m) is as follows:

- End 2017 - 25 m
- End 2018 - 45 m
- End 2019 - 55 m
- End 2020 - 65 m
- End 2021 - 85 m
- End 2022 - 95 m
- End 2023 - 125 m
- End 2024 - 185 m.

12.2.2. Surface Runoff

The water management system is designed to handle runoff for the most critical event between the 1:100 year 24 hour rainfall event of 64 mm and the 1:25 year 24 hour snowmelt event of 51 mm. This is the criterion established for the A154/A418 dike water management systems and is being carried over for the A21 dike. The estimated inflows to the three pump stations for the 1:100 year rainfall and the1:25 year snowmelt events are presented in Table 12-2 and Table 12-3, respectively. A runoff coefficient of 0.9 was assumed for the 24-hour rainfall event, while a runoff coefficient of 1 was assumed for snowmelt.

Table 12-2. Runoff inflows to pump stations for 1:100 year 24 hour rainfall event.

Pump Station	Rainfall Duration (hours)	Rainfall Depth (mm)	Runoff Coefficient	Drainage Area (m^2)	Inflow Rate (l/s)	Inflow Volume (m^3)
DPS-7	24	63.8	0.9	34,215	22.7	1,965
DPS-8	24	63.8	0.9	29,070	19.3	1,669
DPS-9	24	63.8	0.9	67,025	44.5	3,849

Table 12-3. Runoff inflows to pump stations for 1:25 year 24 hour snowmelt event.

Pump Station	Snowmelt Duration (hours)	Snowmelt Runoff (mm)	Drainage Area (m^2)	Inflow Rate (l/s)	Inflow Volume (m^3)
DPS-7	24	50.7	34,215	20.1	1,735
DPS-8	24	50.7	29,070	17.1	1,474
DPS-9	24	50.7	67,025	39.3	3,398

Collection basins will be constructed at each of the pump stations to allow for the collection and storage of inflows from both seepage and surface runoff. Table 12-2 and Table 12-3 indicate that the 100-year, 24-hour rainfall is the critical event, therefore, each collection basin would require a live capacity to store the inflow volumes shown in Table 12-2.

12.3. Pumping Stations and Collection Basin Dimensions

The capacity of the three pumping stations allows for the inflows in the collection basin (surface runoff plus seepage through the dike and foundation) and the storage available in the wet well. Drawing 14300-41D2-1022 shows the elevation and location of the three pumping stations with respect to the runoff collection system. DPS base elevations are presented in AECOM (2013). Drawing 14300-41D2-1024 shows the required size of containment berm for each of the collection basins. The berms will be constructed of locally available till and will range in height from 1.3 to 1.9 m.

12.4. Re-grading and Ditching Requirements

The infield area will require minor grading to develop suitable slopes to direct storm water runoff and snowmelt toward the pump stations. Site preparation will generally involve:

- Removal of lakebed sediments, where necessary to prevent silting up of the DPS collection basins areas
- Excavation of infield surfaces in places to grade or channel water towards the low lying pump stations. Excavations for small ditches in the order of 0.5 m deep will be required
- Placement of compacted backfill in isolated depressions to maintain positive drainage. Construction of berms will also be required to prevent water from spilling into the pit from the infield area where it naturally grades towards the pit.

Given that lakebed till is abundant in the grading area, most of the excavated in-situ materials can be used for backfill. The grading design has been completed such that there is no excavation of rock. For erosion protection on the faces of the containment berms, 200 mm crushed stone will be required. Drawings 14300-41D2-1021 and 1023 shows the general grading plan and sections for the infield earthworks design. Also shown on Drawing 41300-41D2-1021 are the proposed pit depressurization wells as designed by Golder (2012). Due to the location and size of the collection basins for DPS-08 and 09, many of these wells are within the collection basin and therefore could be inundated by water. Further consideration for the construction of and access to the depressurization wells will be required, and relocations of some wells may be desirable.

12.5. Water Handling Design Scope Covered by Others

Several components of the water handling system are being designed by others:

- Mechanical (pumping and piping) for:
    - DPS system
    - Pit wall depressurization system
    - In-pit dewatering system.
- Electrical for water management systems, instrumentation systems and power to these systems.

13.0 CONSTRUCTION QUALITY ASSURANCE AND CONTROL (QA/QC)

13.1. General

A rigorous Quality Assurance and Quality Control (QA/QC) program is essential for the construction of the Diavik dikes. Such plans were prepared by NKSL for the construction of the A154 and A418 dikes, and AMEC (2012) provided a plan for the A21 dike. Contractors will be responsible for QC, and the Engineer-of-Record (BGC) for QA. The QA/QC plan for the A21 dike is provided in Appendix N. The scope of the QA/QC plan addresses the following stages of the works:

- Pre-construction:
  - Review contractor submittals and methodology
  - Refine QA/QC program as appropriate based on contractor submittals.
- During construction:
  - Material testing program
  - Survey control and underwater inspection
  - Monitoring of work in progress
  - Continuous monitoring and reporting.

The highlights addressed within the QA/QC plan (Appendix N) are outlined below.

13.2. Crushing Plant

- Gradation control for the stockpiles and crushed rockfill zones
- Checking for segregation in the stockpiles and adjustments to stockpile management procedures as required.

13.3. Foundation Preparation

- Bathymetric survey of the lakebed within the foundation footprints before and after sediment removal for the A21 dike
- Underwater inspections upon completion of the sediment and boulder removal operations
- Visual inspection of prepared foundations for on-land abutments
- Sampling for water quality and turbidity measurements.

13.4. Fill Placement

- Survey controls for on-land foundation preparation, dredging, filter blanket placement, and dike construction
- Material placement procedures for filter blanket, underwater buttresses in adverse bathymetry areas, fill on on-land abutments, main dike embankment advanced into lake via slip displacement methods, and fill placed in the dry to raise the main dike embankment above lake level

- Compaction of the fill materials placement in the dry
- Conformity of fill gradation and compaction in the dry to specifications and degree of segregation
- Underwater slopes formed by Zones 1, 2 and 3 during dike embankment advance into the lake via slip displacement
- Installation of protective layers.

13.5. Bedrock Grout Curtain

- Calibration of gauges, meters and recording units
- Batch plant and mix control
- Sequence of grouting and grouting intervals
- Drill logs and verticality control
- Conformity of grout mix to specifications
- Recording of grout pressure, density and flow rates, material quantities on continuous basis
- Grout sampling and laboratory testing - cube test
- Water pressure (packer) testing.

13.6. Pre-Drilling and CSM

- Survey of borehole location prior to the drilling operation
- Borehole depth check
- Borehole verticality check
- Removal of obstructions and noting obstruction depths
- Monitoring type and volume of evacuated and backfill material
- Maintaining field records of pre-drilling operation
- Verification of CSM mix design
- Monitoring of CSM panel installation
- Verification of panel verticality
- Verification of panel overlap
- Monitoring of CSM production parameters (such as depth and penetration/extraction rates, flow rate and total volume of slurry, slurry pressure, plumped slurry volume with time and with depth, inclination, rotation speed and torque of mixing wheels)
- Cut-off wall material sampling and laboratory testing during pre-production phase, production phase (wet sampling) and post-production phase (coring)
- Maintaining field records of the CSM operation
- Wall integrity check with cored boreholes.

13.7. Jet Grout

- Calibration of gauges, meters and recording units
- Batch plant and mix control
- Sequence of grouting

- Definition of overburden-bedrock contact and elevation
- Drill logs and verticality control
- Conformity of materials and grout slurry to specifications
- Recording of grout pressure, density and life rates, material quantities on continuous basis
- Integrity check with verification holes
- Water inflow testing
- Grout sampling and laboratory testing.

13.8. Instrumentation

- Condition and calibration
- Location and installation details
- Readings, reporting and interpretation.

13.9. Dewatering

- Pump and pipeline locations
- Monitoring of water levels and flow rates
- Monitoring of dike instrumentation and dike performance during dewatering.

14.0 ADDITIONAL INVESTIGATIONS

14.1. General

The site investigations completed to date in support of the A21 dike design are described by Golder (2006a, 2007), and are provided in Appendices A and B. The scope of those site investigations is illustrated on Drawings 14300-41D2-1010.1 through 1010.6.

The lakebed bathymetry shown on the drawings is based on a Cori Survey Operations bathymetric survey carried out in July and August of 1999, with Challenger bathymetry data from 1995/1996 surveys used to fill in data gaps.

Prior to and during dike construction, additional geotechnical and bathymetric investigations of the A21 dike alignment are recommended, as outlined in the following sections. None of these additional investigations are deemed essential from a design perspective - the investigations completed to date are judged sufficient to support the design as presented herein. The overall purpose of the investigations is to:

- Increase confidence in the interpreted stratigraphy, and refine quantity estimates in advance of construction. Drawings 14300-41D2-1011.1 through 1011.4 indicate some discrepancies between interpreted stratigraphy and subsurface information obtained from boreholes completed in 2007 primarily related to the depth of bedrock beneath the dike cut-off wall. A contributor to the discrepancy is the offset of the boreholes from the reference line of the dike. Additional stratigraphic profiling is proposed to be collected along the reference line (cut-off wall) alignment of the A21 dike which would reduce the likelihood of unanticipated conditions being encountered during construction
- Provide additional detailed information to refine quantity estimates for the dike construction
- Assist in refinement of construction planning and scheduling
- Finalize the locations of the thermosyphon groups on the dike abutments.

All proposed site investigation activities are shown on Drawing 14300-D2-1010.7.

14.2. Updated Bathymetric Survey

The current bathymetric data available for the A21 dike alignment is between about 15 and 19 years old. Given the sensitivity of dike quantities to the bathymetry, and the importance of establishing a solid pre-dredging baseline survey along the dike alignment, an updated survey is recommended. Echo-sounding technology has advanced significant over the last two decades and should be taken advantage of for A21. Advanced bathymetric survey methods may also enable the following:

- Identification of the contact between the very soft upper horizon of lakebed sediments to be dredged, and the underlying firmer horizon of lakebed sediments not requiring removal

- Identification of the contact between the lakebed sediments and the underlying lakebed till
- Identification of boulders atop and within the lakebed sediments and, potentially, large problematic boulders within the lakebed till along the alignment of the cut-off wall.

The survey should be undertaken in the summer of 2015, a year in advance of the marine work that is to commence in 2016.

14.3. Boulders Survey

Concentrations of boulders in the lake shallows are commonplace at the Diavik site. Shallow areas with visible boulder lags are evident along the A21 dike alignment as shown on Drawing 14300-41D2-1033.1, and on Figure 5-1. As shown on Drawing 14300-41D2-1004.1, there are extensive areas along the dike alignment in shallow water where foundation preparation via excavator (working from the dike) or clamshell are expected to be required.

Additional work to better quantify areas of boulder concentrations along the dike alignment is recommended for the summer of 2015, to allow for better quantification for boulder removal, and to allow appropriate planning given the schedule-critical nature of lakebed foundation preparation. It may be that current, state-of-the-art bathymetric survey equipment could be deployed, as part of the survey recommended above in Section 14.2, to facilitate this. Boat surveys during periods of relatively calm water on the lake to assess boulder conditions in the shallows are also recommended, with GPS tracking to delineate visible boulder lags. The overall objective is to take all reasonable steps to quantify the boulder removal work as well as possible to avoid any schedule slippage due to unanticipated conditions.

14.4. Abutment Thermistor Installations and Geophysical Surveys

Drawing 14300-41D2-1010.6 shows the locations of borehole thermistor strings installed to date along the A21 dike alignment. Based on the thermistor data, and the interpreted location of the permafrost-talik contact at the north and south dike abutments, preliminary locations for the thermosyphon groups (see Section 5.8 and Drawing 14300-41D2-1018), which straddle the contact 15 m to either side, have been established (see Drawing 14300-41D2-1017 for south abutment group, and Drawing 14300-41D2-1015 for the north abutment group). Additional boreholes with thermistor strings are recommended for each abutment to better define the locations for the thermosyphon groups. These installations can be undertaken in the fall of 2016, upon completion of the 4th stage of embankment construction per Drawing 14300-41D2-1025.1. The thermosyphon banks are to be installed the following spring, and it is important to get at least 4 months of data to confirm the permafrost-talik contacts.

The details for the additional drilling (vertical boreholes) and thermistor string installations are as given in Table 14-1. The locations are as shown on Drawing 14300-41D2-1010.7. No thermistors are judged necessary at the north abutment.

Table 14-1. Additional drilling and thermistor strings installation.

Borehole Number	Northing (m)	Easting (m)	Depth (m)	Thermistor beads
ST-1	7,148,834	534,027	32	10 in total, at following depths below Stage 4 dike crest at El. 418 m: 4,6,8,10,12,15,18,22,27,32
ST-2	7,148,904	534,123	34	10 in total, at following depths below Stage 4 dike crest at El. 418 m: 6,8,10,12,14,17,20,24,29,34

In addition to the two thermistor installations, delineation of the permafrost-talik contacts via Ohmmapper resistivity survey is recommended. Ohmmapper is a capacitively-coupled resistivity meter that measures the electrical properties of rock and soil without galvanic electrodes used in traditional resistivity surveys. A simple coaxial-cable array with transmitter and receiver sections is pulled along the ground either by a single person or attached to a small all-terrain vehicle. The survey should be performed from the lake ice in the late winter of 2015-2016, prior to embankment construction. The areas to be mapped extend from Sta. 0+000 m to 0+100 m (north abutment), and Sta. 1+500 m to 1+800 m (south abutment). For each reach, five survey lines (centered around the dike reference line), along the axis of the dike and spaced at 10 m, should be undertaken, for a total of 2,000 m of survey line.

14.5. Overwater Acoustic Profiling

Ground penetrating radar (GPR) surveys along the A21 dike alignment were undertaken in the winter of 2006-07 by Aurora Geosciences (Golder, 2007), from the frozen lake surface to delineate lakebed stratigraphy, and to identify the bedrock surface. Seismic refraction survey lines were undertaken that same winter, by Golder (Golder, 2007). The locations of the GPR and seismic refraction survey lines are as shown on Drawing 14300-41D2-1010.5. The survey lines were typically oriented parallel or transverse to the 2007 alignment of the dike.

Given the modification of the dike alignment since the 2006-07 surveys, and to provide additional data to better define subsurface stratigraphy along the dike alignment, an overwater acoustic profiling program is recommended. The method consists of towing an energy source appropriate for the depth of investigation and bottom materials behind a survey vessel along a traverse line. The acoustic signal reflected from the sub-bottom is received by a towed hydrophone array and amplified and digitally recorded as a continuous profile. In the field, the data can be visually reviewed in real time on the high resolution display of a notebook computer. Bathymetry information is recorded synchronous with the positioning and seismic information using a narrow beam fathometer. Pre-dredge surveys may be carried out, together with bathymetric surveys, to identify hazards such as shallow bedrock, and to classify the sediment types. This information determines how much volume is to be removed by dredge operations, and allows appropriate deployment of suction versus clamshell dredging. Post-dredge bathymetry would verify dredge contract fulfillment, and that the depth meets all navigation requirements.

Potentially, the same contractor engaged to undertake the bathymetric survey could carry out the acoustic profiling work. Overwater seismic refraction profiling could be included to define materials via seismic velocities, to tie the acoustic profiling work to the 2006-07 seismic refraction survey results which established typical seismic velocities for the lakebed sediments, lakebed till, and bedrock.

These surveys should be undertaken in the summer of 2015, coincident with the updated bathymetric surveys. The acoustic profiles recommended are shown on Drawing 14300-41D2-1010.7, and listed Table 14-2 (for profiles along the cut-off wall alignment) and Table 14-3 (for profiles transverse to the cut-off wall alignment).

Table 14-2. Overwater acoustic profiling lines - along the cut-off wall.

Profile Number	From:		To:		Length (m)
	N	E	N	E	
COW-1	7,148,815	533,832	7,148,868	534,151	323
COW-2	7,148,802	534,035	7,149,231	534,458	603
COW-3	7,149,139	534,427	7,149,519	534,420	380
COW-4	7,149,444	534,446	7,149,644	534,227	298
COW-5	7,149,628	534,288	7,149,612	533,899	390
COW-6	7,149,642	533,976	7,149,514	533,758	253

Table 14-3. Overwater acoustic profiling lines - transverse to the cut-off wall.

Profile Number	From:		To:		Length (m)
	N	E	N	E	
TR-1	7,149,024	534,129	7,148,912	534,254	167
TR-2	7,149,127	534,213	7,148,985	534,359	203
TR-3	7,149,198	534,277	7,149,057	534,431	208
TR-4	7,149,257	534,285	7,149,192	534,503	227
TR-5	7,149,343	534,244	7,149,327	534,491	248
TR-6	7,149,419	534,340	7,149,455	534,492	157
TR-7	7,149,437	534,329	7,149,563	534,449	175
TR-8	7,149,484	534,294	7,149,625	534,421	190
TR-9	7,149,511	534,252	7,149,643	534,364	173
TR-10	7,149,522	534,200	7,149,681	534,222	161
TR-11	7,149,530	533,985	7,149,686	533,969	157
TR-12	7,149,518	533,940	7,149,661	533,877	157
TR-13	7,149,481	533,854	7,149,599	533,780	139

14.6. Geotechnical Drilling

Seven geotechnical boreholes are proposed along the cut-off wall alignment, to provide infill geotechnical data, and complement the 2006/2007 investigation campaigns in areas lacking data as a result of dike alignment shift. The objective of these boreholes is to reduce uncertainty in advance of cut-off wall construction. As such, these boreholes should be drilled following completion of the first year of embankment construction, and be carried out in advance of the bedrock curtain grouting. Similar programs were carried out following the first construction season for each of the A154 and A418 dikes. The objectives of these boreholes are:

- Obtain geotechnical data along the cut-off wall alignment where the dike alignment was realigned in 2012
- Confirm thickness of subsurface strata (no sampling of the lakebed sediments or tills required, so sonic drilling equipment is not required)
- Confirm depth to bedrock
- Obtain bedrock core for detailed geotechnical logging
- Confirm hydraulic properties of bedrock, drilling and conducting packer tests deeper than the planned depth of the grout curtain in:
    - The deeper water portion of the dike
    - Along the northeast where Golder (2012) infers the presence of a zone of enhanced hydraulic conductivity.

This program could be carried out using the same drill rig used for the abutment thermistor installations (Section 14.4).

The proposed borehole locations are as shown on Drawing 14300-41D2-1010.7, and the borehole particulars as given in Table 14-4. All boreholes would be drilled at an inclination of 15 degrees from vertical, with azimuths along the cut-off wall axis, and extend to a depth of 30 m (vertical) into bedrock.

Table 14-4. Overwater acoustic profiling lines - along the cut-off wall.

Borehole Number	Location:		Estimated depth along borehole axis (m)
	N	E	
BH-1	7,149,055	534,286	50
BH-2	7,149,127	534,356	49
BH-3	7,149,254	534,424	53
BH-4	7,149,437	534,416	40
BH-5	7,149,545	534,348	40
BH-6	7,149,606	534,263	44
BH-7	7,149,613	533,977	41

15.0 DIKE CLOSURE AND RECLAMATION CONCEPT

The proposed plan for the A21 open pit, upon completion of mining, is to return the area to Lac de Gras. Near the end of mining operations, the buffer area between the perimeter of the mine pit and the toe of the dike will be contoured with rock and till as required, to create a 2-3 m thick blanket with undulating topography suitable for fish rearing habitat (a habitat type that is limiting fish populations in Lac de Gras). The dike will be carefully breached in one or two small areas allowing the pit and the areas behind the A21 dike to fill with Lac de Gras water. After allowing time for any suspended solids that may have been generated during filling to settle, additional small sections of the dike will be excavated to allow water circulation, navigation and fish passage. Other infrastructure, including the causeway connecting South Island to East Island, will be removed.

16.0 CLOSURE

We trust the above satisfies your requirements at this time. Should you have any questions or comments, please do not hesitate to contact us.

Yours sincerely,

BGC ENGINEERING INC.
per:

REGISTERED PROFESSIONAL ENGINEER
T.E. MARTIN
LICENSEE
Nov/28/14
NWT/NU

Todd E. Martin, P.Eng., NT.
Principal Geotechnical Engineer

Ken Halisheff, P.Eng., B.C.
Senior Geotechnical Engineer

F/R

Lukas Arenson, Dr.Sc.Techn. ETH, P.Eng. (BC, YT, NT/NU)
Senior Geotechnical Engineer

Reviewed by:

Roy Mayfield, Ph.D., P.E.
Principal Geotechnical Engineer

TM/KH/LA/rm/ht

PERMIT TO PRACTICE
BGC ENGINEERING INC.

Signature ____________

Date Nov/28/14

PERMIT NUMBER: P 285

The Association of Professional Engineers, Geologists and Geophysicists of the Northwest Territories

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