Design of the Tailings Containment Area Doris North Project, Hope Bay, Nunavut, Canada

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Table of Contents

1 Introduction 1
1.1 Reference Documents 1

2 Project Area Description 3
2.1 Location and Access 3
2.2 Regional Geology 3
2.3 Seismicity 3
2.4 Climate 4
2.5 Permafrost 4
2.6 Hydrology 5
2.7 Hydrogeology 5

3 Design Criteria and Assumptions 6
3.1 Design Basis 6
3.1.1 Tailings Characteristics 6
3.1.2 Tailings Geochemistry and Supernatant Quality 7
3.2 Design Criteria 7
3.2.1 Dam Classification 7
3.2.2 Design Earthquake 8
3.2.3 Design Capacity 9
3.2.4 Design Freeboard 9
3.2.5 Design Flood 9
3.2.6 Stability 9

4 Site Selection 11
4.1 Site Selection Approach 11
4.2 Identified Sites 11
4.3 Results 12

5 Dam Design 14
5.1 Overview of Tailings Containment Area 14
5.2 Storage Characteristics 14
5.3 Foundation Conditions 15
5.3.1 Geotechnical Investigations 15
5.3.2 North Dam 16
5.3.3 South Dam 17
5.3.4 Spillway 17
5.4 Dam Design 17
5.4.1 North Dam 17
5.4.2 South Dam 18
5.5 Thermal Analysis 18
5.5.1 General 18
5.5.2 Method of Analysis 18
5.5.3 Results from Thermal Analysis 19
5.6 Creep Deformation Analysis 20
5.6.1 General 20
5.6.2 Method of Analysis 20
5.6.3 Results from Creep Deformation Analysis 20
5.7 Stability Analysis 21

5.7.1 General ... 21
5.7.2 Method of Analysis ... 21
5.7.3 Results from Stability Analysis ... 22
5.7.4 Liquefaction Potential ... 22
5.8 Dam Settlement Analysis ... 22
5.9 Seepage Control ... 23

6 Water Balance ... 24
6.1 General ... 24
6.2 Water Balance Calculation ... 24
6.3 Water Balance Results ... 25

7 Design of the Water Management Facilities ... 26
7.1 Tailings Deposition Infrastructure ... 26
7.2 Process Water Reclaim Infrastructure ... 26
7.3 Fresh Water Make-Up Infrastructure ... 27
7.4 Operational Discharge (Decant) Infrastructure ... 27
7.5 Spillway ... 28
7.6 Shoreline Erosion Protection ... 29

8 TCA Construction ... 30
8.1 Construction Materials ... 30
8.1.1 Dams, Spillway and Shoreline Erosion Protection ... 30
8.1.2 Pipelines and Emergency Dump Catch Basins ... 31
8.2 Construction Equipment ... 31
8.3 Construction Quality Control and Quality Assurance ... 32
8.4 Construction Quantities ... 32
8.5 Construction Procedure ... 32
8.5.1 Further Field Characterization ... 32
8.5.2 Dams ... 32
8.5.3 Spillway ... 33
8.5.4 Pipelines ... 34
8.5.5 Shoreline Erosion Protection ... 34

9 TCA Operation ... 35
9.1 Dams ... 35
9.2 Spillway ... 35
9.3 Tailings Deposition ... 35
9.4 Reclaim and Fresh Water Make-Up ... 36
9.5 Discharge (Water Management Strategy) ... 36
9.6 Operations Manuals ... 37

10 TCA Maintenance ... 39
10.1 Dams ... 39
10.2 Spillway ... 39
10.3 Pipelines ... 39
10.4 Emergency Dump Catch Basins ... 40
10.5 Shoreline Erosion Protection ... 40

11 TCA Monitoring ... 42
11.1 Dams ... 42
11.1.1 Routine Visual Inspections ... 42
11.1.2 Annual Dam Safety Inspection ... 42

11.1.3 Instrumentation 42
11.2 Spillway 43
11.3 Pipelines 43
11.4 Water Quality 44
11.5 Shoreline Erosion Protection 45

12 TCA Closure Methodology 46
12.1 General 46
12.2 Water Management 46
12.3 Water Cover 46
12.4 North Dam Breach 47

13 References 48

List of Tables

Table 1: Probabilistic seismic ground motion analysis at the Doris North Project site 3
Table 2: CDA dam classification in terms of consequences of failure 8
Table 3: TCA sites considered for the Doris North Project 11
Table 4: Master categories and criteria used to evaluate TCA sites 12
Table 5: Quantitative results of the base case MAA 13

List of Drawings

G-01 Location Plan
G-02 General Arrangement
G-03 Index
G-04 Thermistors and Geotechnical Drill Hole Plan and Details
G-04 Construction Fill Material Specifications
T-01 Tailings Containment Area Stage Curves and Deposition Plan
T-02 North Dam Layout and Key-Trench Details
T-03 North Dam Sections
T-04 North and South Dam Typical Details
T-05 South Dam Layout and Key-Trench Details
T-06 South Dam Sections
T-07 Typical Thermosyphon Details
T-08 Spillway Plan, Typical Sections and Details
T-09 North Dam Instrumentation Layout and Typical Details
T-10 South Dam Instrumentation Layout and Typical Details
T-11 Tailings Slurry, Reclaim, Fresh Water Make-Up and Decant Pipelines Layout
T-12 Typical Pipeline Details
T-13 Emergency Dump Catch Basin Layout and Typical Details
T-14 Shoreline Erosion Protection Typical Details

List of Appendices

Appendix A	SRK Technical Memorandum re: Doris North Project Tailings Properties
Appendix B	Thermal Design of Tailings Dams (EBA 2006)
Appendix C	SRK Technical Memorandum re: Water Cover Design for Tail Lake
Appendix D	Winter 2006 Geotechnical Field Investigation (SRK 2006b)
Appendix E	SRK Technical Memorandum re: Wave Run-up Calculations
Appendix F	SRK Technical Memorandum re: Water Balance Calculation
Appendix G	SRK Technical Memorandum re: Shoreline Erosion Protection Adaptive Management Plan
Appendix H	SRK Technical Memorandum re: Addressing NWB Technical Review Comments

1 Introduction

This report presents the design of the tailings containment area (TCA) for the Doris North Project owned by Miramar Hope Bay Ltd. (MHBL). The document provides the TCA design, operating plan and related water management, as well as a conceptual closure plan for the facility.

This report has been prepared as part of the Water Licence Application to the Nunavut Water Board (NWB). The following documents form part of the designs presented in this report, and should therefore be read in conjunction with this document:

- SRK Consulting (2007a). *Design of the Surface Infrastructure Components*, Doris North Project, Hope Bay, Nunavut, Canada. Report submitted to MHBL, March.
- SRK Consulting (2007b). *Technical Specifications for Tailings Containment Area and Surface Infrastructure Components*, Doris North Project, Hope Bay, Nunavut, Canada. Report submitted to MHBL, March.
- SRK Consulting (2007c). *Water Quality Model*, Doris North Project, Hope Bay, Nunavut, Canada. Report submitted to MHBL, March.

The drawings referenced in this report form part of a set of drawings completed for the TCA and surface infrastructure designs for the Doris North Project, and are bound as a separate volume:

- SRK Consulting (2007d). *Engineering Drawings for Tailings Containment Area and Surface Infrastructure Components*, Doris North Project, Hope Bay, Nunavut, Canada. Drawings submitted to MHBL, March.

This report was first issued in October 2006, but has been revised taking into account technical review comments by the NWB and other interveners received in December 2006 and January 2007. In addition to the revising this report, additional technical responses to NWB review comments are documented in Appendix H.

1.1 Reference Documents

Although this document is considered a stand-alone report, it does build on a series of historic SRK reports that are in the Public Domain and that contain relevant information. The following is a list of these reports:

- SRK Consulting (2002). Hope Bay Doris North Project, *Tail Lake Dam Site Geotechnical Investigation and Conceptual Design Report*, Nunavut, Canada. Report Submitted to MHBL, December.

  Sections 3 and 4 of this report documents the dam foundation field characterization programs carried out in April and September.

- SRK Consulting (2003). Hope Bay Doris North Project, *Tailings Impoundment Preliminary Design*, Nunavut, Canada. Report submitted to MHBL, October.

  This report contains complete details of all the geotechnical investigations at the dam sites up to this date.

- SRK Consulting (2005a). *Preliminary Tailings Dam Design*. Doris North Project, Hope Bay, Nunavut, Canada. Report submitted to MHBL, October.

  This report is the primary precursor document to this current design report, and is considered to be a complete reference document to be read in conjunction with this report. The current reports present significantly greater detail with respect to dam geometry and thermal analysis; however, basic design criteria and background geotechnical data can be sourced from the October 2005 report.

- SRK Consulting (2005b). *Groundwater Assessment*. Doris North Project, Hope Bay, Nunavut, Canada. Report submitted to MHBL, October.

  This report resents data and analysis confirming the fact that the TCA is hydrogeologically isolated to the TCA catchment. This includes comments regarding the extent of talik under Tail Lake.

- SRK Consulting (2006a). *Evaluation of Tailings Management Alternatives*. Doris North Project, Hope Bay, Nunavut, Canada. Report submitted to MHBL, August.

  This is the final comprehensive document dealing with tailings alterative evaluation. This report was released to DFO and EC in support of the MMER Schedule II Permit Application for Tail Lake.

2 Project Area Description

2.1 Location and Access

The Doris North Project is located approximately 400 km east of Kugluktuk and 160 km southwest of Cambridge Bay in the West Kitikmeot Region of the Territory of Nunavut. The project location is shown on Dwg. G-01.

Access to the site is by air (float planes in the summer, and an ice airstrip in the winter); with an annual barge sealift re-supply in Roberts Bay during the open water season.

2.2 Regional Geology

The Doris North Project is in the faulted Bathurst Block, forming the northeast portion of the Slave Structural Province, a geological sub-province of the Canadian Shield. The region is underlain by the late Archean Hope Bay Greenstone belt, which is seven to 20 km wide and over 80 km long in a north-south direction. The belt is mainly comprised of mafic metavolcanic (mainly meta-basalts) and meta-sedimentary rocks that are bound by Archean granite intrusives and gneisses. The greenstone package has been deformed during multiple events, and is transected by major north-south trending shear zones that appear to exert a significant control on the occurrence of mineralization, particularly where major flexures are apparent and coincident with antiforms.

2.3 Seismicity

A site specific seismic hazard assessment was done by the Geological Survey of Canada, according to the procedures documented in Adams and Halchuck (2003). Peak ground accelerations and velocities for various annual probabilities of exceedance were determined and are listed in Table 1.

Table 1: Probabilistic seismic ground motion analysis at the Doris North Project site

Annual Probability of Exceedence	Return Period (Years)	Peak Ground Acceleration (g)	Peak Ground Velocity (cm/sec)
0.01	100	0.014	0.033
0.005	200	0.018	0.039
0.0021	475	0.023	0.049
0.0010	1,000	0.028	0.060
0.0004*	2,475	0.059	-

*The 1:2,475 return period data is not site specific to the Doris North Project area, but are for Kugluktuk.

The Doris North Project falls within the “stable” zone of Canada. This region has too few earthquakes to define reliable seismic source zones. However, international experience suggests that large earthquakes can occur anywhere in Canada, although the probability is very low.

Within this “stable” zone, the project area falls in acceleration zone 1 (Za = 1) and experiences zonal accelerations of 0.05 g. The velocity zone in which the area falls is zone 0 (Zv = 0) which corresponds to zonal velocities of 0.05 m/s. These zonal classifications are the lowest zones classified on the seismic hazard maps of Canada (Adams and Halchuck 2003).

2.4 Climate

Baseline climate data for the project was collected at the Boston and Windy camps during exploration (August 1993 thru 2003, with some interruptions). Climatic data was also collected from the Doris North site since May 2003. The site specific data combined with data from four longer-term regional weather stations operated by Environment Canada (Lupin, Cambridge Bay, Lady Franklin Point and Kugluktuk) were used to develop annual climate profiles for the Doris North site.

The Doris North site has a low arctic ecoclimate with a mean annual temperature of -12.1^{o}C with winter (October to May) and summer (June to September) mean daily temperature ranges of -50^{o}C to +11^{o}C and -14^{o}C to +30^{o}C, respectively; and mean precipitation ranges from 94 mm to 207 mm, with only about 40% falling as rain. Annual lake evaporation (typically occurring between June and September) is about 220 mm.

A detailed discussion on how this climatic data was used in thermal modelling of the North and South dams are provided in Section 2.2 of EBA (2006). This report has been included as Appendix B to this report.

2.5 Permafrost

The Doris North site is underlain by continuous permafrost that has been estimated to extend to depths in the order of 550 m (SRK 2005a). This permafrost depth is based on a 200 m deep drill hole (SRK-50, see Dwg. G-04) where the mean surface temperature is about -6.3 °C and the geothermal gradient is 11.4 °C.km^{-1}. The geothermal gradient in the upper 100 m appears to be isothermal or slightly negative. For comparison, the deep ground temperature profile measured at the Boston Camp, 60 km south of the site, also suggested a similar permafrost depth, about 560 m (EBA 1996; Golder 2001). The mean annual surface temperature is however colder at -10 °C and the geothermal gradient is higher at 18 °C.km^{-1}. The difference in the ground temperature profiles at these two sites can be attributed to different surface conditions and the thermal conductivity of the ground at depth. The geothermal gradient measured at the Doris North site is probably representative of the conditions at the TCA.

Temperature data collected around Tail Lake indicates that the active layer in the marine clay/silt soils appears to be about 0.5 m, while the sand deposit has an active zone no greater than 2 m. The depth of zero annual amplitude varies between 11 and 17 m. The ground temperatures at the depth of zero annual amplitude are generally in the range of -9 to -7 °C.

2.6 Hydrology

The Doris North Project is located primarily in the Doris Lake outflow drainage basin. Tail Lake basin, part of the Doris basin, is the Projects TCA. Peak flows typically occur in June during snowmelt. A second smaller peak may occur from rainfall in late August or early September. The streams in the study area are usually frozen with negligible flow from November until May. The mean flow from June to October for Tail, Doris and Little Roberts Lake outflows are about 0.03, 0.85, and 1.73 m^3/s, respectively (AMEC 2003a).

2.7 Hydrogeology

The permafrost underlying the Project site is generally impervious to groundwater movements. Groundwater movement will only occur in the shallow active layer (0.5 m to 2 m) during its seasonal thaw period. There is no hydraulic connection between the taliks beneath Tail and Doris Lakes, as has been demonstrated though a series of deep drill holes (SRK 2005b).

3 Design Criteria and Assumptions

3.1 Design Basis

3.1.1 Tailings Characteristics

MBHL is proposing to extract about 458,000 tonnes of ore from the underground Doris North mine. The ore will be processed at an average rate of 720 tonnes per day to yield 306,830 ounces of gold over a 24-month operating period.

A site plan of the surface facilities is shown on Dwg. G-02. Ore will be trucked to a surface stockpile, crushed and conveyed to an ore stockpile and then fed to a single ball mill. The slurry output from the ball mill will pass through a gravity concentrator to recover the “free milling” gold. Approximately 40% of the gold in the ore will be recovered by gravity. The balance of the gold will be extracted by physical separation of the minerals that contain the gold and then extracting the gold by chemical leaching.

To achieve physical separation, the slurry output from the gravity circuit will be subjected to a froth flotation process to recover gold bearing sulphide minerals in a flotation concentrate. The flotation circuit will reduce the material for further processing to approximately 72 tonnes per day (~10% of original ore processed). The remaining 90% (~648 TPD) will be discharged to the tailings impoundment with no further treatment.

The flotation concentrate will be dewatered partially and the excess water will be discharged with the tailings to Tail Lake. The concentrate will then be leached with cyanide in a conventional agitated leach circuit. The leached slurry will be filtered and washed. The gold bearing cyanide solution (filtrate) will then go to either electrowinning or Merrill crow for gold recovery. The filtered and washed flotation concentrate will be placed as backfill in the underground workings and will not be discharged with the tailings to Tail Lake.

It will be necessary to discard a water bleed stream to maintain the overall leach circuit water balance. The clear bleed solution (about 40 % of the leach circuit filtrate) will be generated as a filtrate after cyanide treatment using the Caro’s acid process. The clear treated bleed solution will be mixed with the flotation tailings stream for disposal in Tail Lake.

Since tailings will not be used to structurally or hydraulically enhance the dam design, the tailings properties do not play a role in the dam design; however, for completeness Appendix A contains a detailed description of the physical tailings properties for the Doris North Project. A complete discussion of the tailings settlement characteristics and tailings geochemistry is presented in SRK (2007c).

3.1.2 Tailings Geochemistry and Supernatant Quality

The Doris North tailings will have low acid generating potential. Static acid base accounting (ABA) testing gave a total neutralization potential ratio of 8.8 and a carbonate neutralization potential of 10.6. The total Sulphur concentration was 0.34 wt% S with non-detectable Sulphate sulphur. The net neutralization potential was +82.7 kg $CaCO_3$/tonne equivalent and a carbonate net neutralization potential of +101.9 kg $CaCO_3$/tonne equivalent.

The tailings solution will meet MMER discharge criteria, is not acutely toxic and will have a supernatant total suspended solids concentration of < 5 mg/L after 24 hours of settling time.

Once extracted from the underground workings, the ore is exposed to atmospheric conditions and the sulphide minerals are exposed to oxidizing conditions. The estimated solute release from the tailings, represent oxidation reactions that occur during the handling and processing of the ore. In addition, any readily available water soluble solutes associated with the ore rock will also be dissolved during processing and report to the tailings water.

Subsequent to deposition, the tailings will at all times be covered by a water depth in excess of 3 m. The water cover will prevent any oxygen entry to the tailings and therefore, the sulphide minerals contained in the tailings will be prevented from oxidizing. Consequently no additional solute release from the tailings will occur after the tailings have been deposited in the TCA. Furthermore the tailings will be fully submerged and water will be decanted from the surface of the TCA, such that no hydraulic gradients will develop that could cause the pore water to be displaced from the tailings. It is therefore expected that the tailings porewater will be “locked” interstitially in the tailings indefinitely. Solute release from the tailings once they have been deposited in the TCA is therefore considered to be negligible.

3.2 Design Criteria

The design criteria at the TCA and the associated ancillary facilities follow the guidelines provided in the Canadian Dam Safety Guidelines (Canadian Dam Association (CDA) 1999).

3.2.1 Dam Classification

The dam classification system recommended in the CDA (1999) guidelines is shown in Table 2.

Table 2: CDA dam classification in terms of consequences of failure

Consequence Category	**Potential Incremental Consequences of Failure[a]**	
	Life Safety[b]	**Socioeconomic, Financial & Environmental[c]**
Very High	**Large number of fatalities**	**Extreme damages**
High	**Some fatalities**	**Large damages**
<u>Low</u>	**No fatalities anticipated**	**<u>Moderate damages</u>**
Very Low	**<u>No fatalities</u>**	**Minor damages beyond owner's property**

a) Incremental to the impacts which would occur under the same natural conditions (flood, earthquake or other event) but without the failure of the dam. The consequence (i.e. loss of life or economic loses) with the higher rating determines which category is assigned to the structure. In the case of tailings dams, consequence categories should be assigned for each stage in the life cycle of the dam.

b) The criteria which define the Consequence Categories should be established between the Owner and the regulatory authorities, consistent with societal expectations. Where regulatory authorities do not exist, or do not provide guidance, the criteria should be set by the Owner to be consistent with societal expectations. The criteria may be based on levels of risk which are acceptable or tolerable to society.

c) The Owner may wish to establish separate corporate financial criteria which reflect their ability to absorb or otherwise manage the direct financial loss to their business and their ability to pay for damages to others.

There will be two dams associated with the TCA. The potential incremental consequences of failure of any of these two dams with regard to life safety factors are classified as "no fatalities", corresponding to a consequence category of "very low". This selection is based upon the remote nature of the site, the low seismic hazard, the climate, the type of dam, and the foundation conditions.

The potential incremental consequence of failure with regard to socioeconomic, financial and environmental factors is classified as "moderate damages", corresponding to a "low" consequence category. Selection of this classification is primarily based upon the financial impacts associated with a dam failure. The socioeconomic and environmental impacts associated with a dam failure at the TCA are very low due to the lack of downstream human habitation and the relatively low level of potential contamination. Based on these factors, the TCA dams are classified as "low" in terms of consequences of failure. However, for all practical purposes, there are no significant differences between design criteria based on a "low" versus a "very low" consequence rating.

3.2.2 Design Earthquake

The CDA (1999) guidelines indicate that the minimum criterion for the design earthquake for a dam in the "low" consequence category would be an earthquake with an annual exceedance probability of 0.01 to 0.001. These probabilities represent return periods of 100 and 1,000 years, respectively. The Geological Survey of Canada indicated that the 1,000 year event has a peak ground acceleration (PGA) of 0.028 g. However, the National Building Code of Canada, suggest that it would be prudent for designers to evaluate the performance of structures during an earthquake with a 2,475-year return period. Although no estimates are available for the Doris North site for that return period, a peak ground acceleration of 0.06 g was estimated for a 2,475-year return period earthquake based on Kugluktuk data.

3.2.3 Design Capacity

A detailed water balance for Tail Lake is documented in Appendix F. This water balance was used in conjunction with the water quality model (SRK 2007c) to determine an appropriate design capacity for Tail Lake. An iterative procedure was followed, where storage capacity was balanced with decant requirements, such that a robust water management design could be implemented that would account for all foreseeable uncertainties and upset conditions, whilst keeping the dams as small as practical for reasons of minimising potential shoreline erosion concerns as well as keeping the construction costs down. Based on this evaluation a Full Supply Level (FSL) of 33.5 m was selected as the design capacity. At this capacity, all proposed water management strategies would be able to function effectively, whilst still allowing a significant but reasonable margin of safety as deemed appropriate by MHBL.

3.2.4 Design Freeboard

The design freeboard requirement for wind induced waves was calculated as 0.3 m. Complete details of this calculation are presented in Appendix E. Considering the peak flood depth in the spillway will be about 0.2 m (see next section), this means a total hydraulic freeboard requirement of 0.5 m, which half of the 1.0 m that has been allowed for.

3.2.5 Design Flood

The CDA guidelines indicates that the minimum criterion for the inflow design flood (IDF) for a dam which coincides with the “low” consequence category would be a flood with an annual exceedence probability of 0.01 (100-year return period) to 0.001 (1,000-year return period). Due to the relatively large catchment of the TCA (440 ha), as well as the real possibility that the spillway may never be put in service, the spillway will be designed to pass a 24-hour, 500-year return period flood (annual exceedence probability of 0.05).

As an added margin of safety, the spillway flood capacity calculation assumes no attenuation of the flood peak within the TCA catchment, and 100% of the design precipitation event will pass over the spillway, without accounting for infiltration or evaporation.

3.2.6 Stability

Slope Stability

The current stability requirements for earth and rock fill dams, advocated by the International Committee on Large Dams (ICOLD) and the CDA (1999), were adopted for the design of the TCA dams. The minimum acceptable factors of safety are 1.5 under static loading conditions and 1.1 under earthquake loading conditions. Complete details regarding the physical stability design criteria are presented in EBA (2006), Section 6.2, which is included as Appendix B.

Thermal Stability

The TCA dams have been designed as frozen core dams. The dams have been designed to maintain “critical sections” of the core and the underlying saline permafrost foundation sufficiently cold and over a wide enough section to be an impermeable barrier to seepage. The critical section of the core is defined as the part of the core that is colder than -2^{o}C during impoundment under normal operating conditions, or colder than -1^{o}C during impoundment under upset conditions. The critical section of the saline permafrost foundation is defined as the portion of the saline permafrost layer that is colder than -8^{o}C under normal or upset conditions. Complete thermal design criteria, including allowance for climate warming and upset conditions are presented in EBA (2006), Section 4.3, which has been included as Appendix B.

Creep Deformation

Strains will develop within the dam embankments and underlying permafrost foundation in response to long-term creep deformations. The Doris North dams have been designed to maintain the long-term integrity of the frozen core and permafrost foundation by limiting the long-term shear strains in these two areas to less than 2 and 10 percent, respectively. Localized zones of collapse or cracking away from the frozen core are tolerated, as this should not jeopardize the overall stability of the embankment. The design criteria used to evaluate creep deformation is presented in detail in Appendix B, Section 5.2.

4 Site Selection

4.1 Site Selection Approach

Site selection to identify the most appropriate location for the TCA has been a continuously evolving process since 2002. Complete details of the site selection process have been documented in a Comprehensive Tailings Alternatives Assessment Report (SRK 2006a).

Final site selection was done using the Multiple Accounts Analysis (MAA) methodology. This methodology allows for an unbiased quantitative assessment of all alternatives taking into account technical, economic, environmental and socioeconomic considerations of the project in question.

4.2 Identified Sites

A total of 22 tailings management alternatives, at 15 sites were selected for the Doris North Project. These sites are listed in Table 3. After completing a reconnaissance-level inspection at each of these sites, the relevant information pertaining to specific site selection criteria was documented. The evaluated criteria, which were grouped according to four master categories, are listed in Table 4. Comprehensive tables containing this information are provided in SRK (2006a).

Table 3: TCA sites considered for the Doris North Project

Disposal Method	Location	Site Number
Sub-aqueous slurry	Sub-marine disposal in Roberts Bay	#1
Sub-aerial slurry	Unnamed Lake southwest of Little Roberts Lake	#2
Sub-aerial slurry	Unnamed Lake west of Doris Creek	#3
Sub-aerial slurry	Unnamed Lake east of Windy Creek	#4
Sub-aerial slurry	Saddle west of Doris Mesa	#5
Sub-aerial slurry	Unnamed Lake west of Doris Lake	#6
Sub-aerial filtered tailings	Quarry #2	#7
Sub-aerial filtered tailings	Open Pit	#8
Sub-aerial filtered tailings	Underground Mine Backfill	#9
Sub-aqueous slurry	Doris Lake	#10
Sub-aqueous or sub-aerial slurry	Tail Lake	#11A & B
Sub-aqueous or sub-aerial slurry	Partial Tail Lake	#11C & D
Sub-aqueous or sub-aerial slurry	Twin Lakes	#12A & B
Sub-aqueous or sub-aerial slurry	South Windy Lake	#13A & B
Sub-aqueous slurry	Spyder Lake (Boston Site)	#14
Sub-aqueous or sub-aerial slurry	Stickleback Lake (Boston Site)	#15A & B

Table 4: Master categories and criteria used to evaluate TCA sites

Technical/Operational	Economic	Environmental	Socioeconomic
• Distance from mill site • Dam details • Dams and ancillary facilities footprint size • Size and volume of impacted lakes • Total permanent habitat loss (lakes and land) • Construction quarry volume requirements • Potential for increased tailings capacity • Ability to recycle tailings supernatant water • Technical, operational, and environmental uncertainty and flexibility • Precedent • Catchment boundaries • Technical feasibility and risks • Operational, closure and regulatory risks/uncertainties • Post closure land-use	• Capital costs • Operational costs • Closure costs • Post closure costs • Fish compensation and monitoring costs • Total costs • Economic risks • Construction risks	• ARD, ML potential • Topographical issues • Geotechnical and seismic issues • Hydrology issues • Geohydrology issues • Atmospheric issues • Water quality issues • Global warming • Arctic Char • Lake Trout • Lake Whitefish • Ninespine Stickleback • Fish habitat compensation and monitoring effort • Caribou • Wolverine • Grizzly bear • Upland birds • Waterfowl • Raptors	• Archaeological sites • Employment opportunities • Training opportunities • Regional economics • Community services • Maintenance of traditional lifestyle • Spiritual well being • Perceived community response • Landowner opinion • Overall perceived socioeconomic consequences and relative preferences

4.3 Results

A Pre-Screening Assessment (i.e. Fatal Flaw Analysis) eliminated 14 of the tailings management alternatives, leaving only nine sites upon which the MAA was carried out. The results of the base case evaluation are presented in Table 5. In addition, a large suite of sensitivity analysis runs were completed to test the bias in the assessment procedure. The details of these are presented in SRK (2006a). The final outcome of the site selection evaluation was that sub-aqueous disposal of tailings in Tail Lake would be the most appropriate tailings disposal alternative for the Doris North Project.

Table 5: Quantitative results of the base case MAA

Alternative	Score					Rank
	Envi-ronment	Socio-eco-nomic	Ope-rational	Eco-nomic	Total	
Site #2 – Unnamed lake southwest of Little Roberts Lake	0.55	0.30	0.33	0.60	1.78	4
Site #3 – Unnamed lake west of Doris Creek	0.52	0.30	0.33	0.50	1.65	6
Site #4 – Unnamed lake east of Doris Creek	0.45	0.30	0.30	0.30	1.35	8
Site #5 – Saddle west of Doris mesa	0.50	0.50	0.27	0.30	1.57	7
Site #6 – Unnamed lake west of Doris Lake	0.43	0.30	0.30	0.30	1.33	9
Site #10 – Doris Lake	0.53	0.15	0.70	0.70	2.08	2
Site #11A – Tail Lake	0.62	0.80	0.73	0.80	2.95	1
Site #12A – Twin Lakes	0.62	0.30	0.37	0.70	1.98	3
Site #13A – South Windy Lake	0.57	0.30	0.33	0.50	1.70	5

5 Dam Design

5.1 Overview of Tailings Containment Area

Operation of the TCA is based on sub-aqueous disposal of tailings, requiring a minimum water cover of 3 m at any given time. Furthermore, tailings is to be deposited within the deepest sections of the TCA, and only water will be in contact with the two containment dams at any given time. Based on this mode of operation, the North and South Dams have been designed as lined, frozen core water retaining structures.

The TCA is located south-east of the mill and mine location, as indicated on Dwg. G-02, and will consist of the following components:

- Two earthen containment structures (North and South Dams);
- Spillway (at the North Dam);
- Tailings deposition infrastructure;
- Process water reclaim infrastructure;
- Fresh water make-up infrastructure;
- Operational discharge (decant) infrastructure; and
- Shoreline erosion protection infrastructure.

5.2 Storage Characteristics

Tail Lake, with its normal water level of about elevation 28.3 meters above mean sea level (mamsl), occupies about 83 ha of an isolated basin south-east of the mill site. The lake is generally shallow, but does have a few pockets as deep as 6 m, as confirmed by bathymetric survey (Golder 2006).

Topographic and bathymetric data were used to develop elevation-capacity curves (stage curves) for the TCA. These are illustrated on Dwg. T-01. The drawing shows the storage capacity of the entire TCA assuming that the stored volume of tailings and/or water is struck level, i.e. a horizontal slope over the entire TCA.

The maximum operating elevation for the containment dams is 33.5 m, which corresponds to the invert elevation of the TCA spillway. The storage capacity of the entire TCA at elevation 33.5 m is about 7.4 million m^3, compared to a tailings volume requirement of about 400,000 m^3. There is an additional 1.8 m of normal freeboard, to an elevation 35.3 m at the North Dam, and an additional 2.3 m of normal freeboard, to an elevation of 35.8 m, at the South Dam. The maximum effective storage or containment elevation for the facility, i.e. emergency situation, is thus at elevation 35.3 m. However, since the dams rely on a frozen core, each dam has an additional 2.2 m of freeboard, which acts as thermal insulation above the frozen core (and GCL termination). Therefore the North Dam crest elevation is at 37.5 m, and the South Dam crest elevation is at 38.0 m. The normal freeboard for each dam as described here exceeds that which is required due to wave run-up, as is documented in Appendix E, and Section 3.2.4.

Tailings will be deposited sub-aqueously (summer and winter), starting at the deepest pockets within the TCA. Tailings will not be deposited on the lake ice during freeze-up, and tailings will under no circumstances be deposited such that it would come into contact with any of the two dams. Although the intent is to move the tailings deposition point frequently so as to deposit tailings as level (i.e. horizontal) as practical, it is understood that there will be some minor undulations which may have to be levelled by some other means. These will however not compromise the minimum water cover requirements for the TCA as is demonstrated in Appendix C.

The TCA has been designed to operate as a zero discharge facility during the two years of operation, if necessary. In addition, under the most conservative water balance assumptions, the TCA would take over five years to reach the FSL of 33.5 m.

In actual fact, the TCA water management plan stipulates that the maximum water level in the TCA would be about 29.2 m, and that within five years after start of tailings deposition, the natural inflow to the TCA would be equal to the amount of annual allowable discharge. Under this scenario, the dams will never reach FSL.

Notwithstanding any of these conditions, the dams have been designed with a 25-year design life in mind, which provides a substantial safety factor. The 25-year design takes into account global warming and upset conditions.

5.3 Foundation Conditions

5.3.1 Geotechnical Investigations

Eight geotechnical drilling programs and a geophysical survey have been undertaken at the Doris North site since 2002, many of which specifically targeted geotechnical and thermal information at the TCA. Details of these programs are presented on Dwg. G-04, and can be summarized as follows:

- The winter 2002 investigation comprised nine drill holes along three section lines across Tail Lake (SRK 2003).
- The fall 2002 program consisted of five drill holes at the North Dam location (SRK 2003).
- The winter 2003 program included a single drill hole along the eastern Tail Lake shoreline, as well as 4 drill holes at the South Dam and 3 drill holes at the North Dam (SRK 2003).
- The summer 2003 program involved further drilling at the North (1 hole) and South Dam (2 holes), as well as three deep holes around the Tail Lake perimeter to investigate potential talik development. Three shallow auger holes and six shallow test pits was also completed at the North Dam (SRK 2003).

- The summer 2004 program consisted of drilling one hole at the North Dam spillway, and three along the perimeter of Tail Lake (SRK 2005a).
- Two geotechnical bulk samples were collected along the North Dam alignment during the 2004 winter season. This program also included the installation of a 200 m long thermistor string located in the vicinity of the mill, between the mine and the Doris Lake shoreline (SRK 2005c).
- The winter 2005 investigation included 5 drill holes at or adjacent to the North Dam alignment and spillway, and three additional drill holes along the shoreline of Tail Lake (SRK 2005d).
- The winter 2006 program comprised two drill holes at the South Dam as well as a geophysical survey along both dam alignments. The geophysical survey also covered the entire TCA perimeter at the FSL elevation of 33.5 m (SRK 2006b, also included as Appendix D; the geophysical survey did not yield very useful results, and therefore the report does not contain detailed assessment of the data).

Many of these subsurface investigations included the installation of thermistors as illustrated in Dwg. G-04. Ten strings were installed along the North Dam alignment, four at the South Dam and ten around the perimeter of Tail Lake.

5.3.2 North Dam

The North Dam will be situated about 200 m downstream of the northern extremity of Tail Lake. The dam alignment, shown in more detail in Dwg. T-02, is within a relatively narrow valley and is essentially perpendicular to the valley. The valley bottom is about elevation 26 m and consists of a narrow marshy area that drains flow from Tail Lake. This surface flow discharges into Doris Lake.

Ground temperature measurements along the dam alignment confirm the presence of cold permafrost with mean annual ground temperatures ranging between -9°C and -7°C. No talik is present under the Tail Lake outflow channel.

The interpreted stratigraphy at the North Dam is shown in Dwg. T-03, and is characterised by two distinct zones. About two thirds of the dam longitudinal section, is dominated by an ice saturated sand deposit that is approximately 10 to 15 m thick. The sand deposit is overlain by a silt and clay layer that is less than 3 m thick. The remaining one-third portion is dominated by marine clayey silt that is up to 15 m thick. The fine-grained materials are ice-saturated and contain excess ground ice. The overburden soils are up to 20 m thick in the base of the valley and thin out at the dam abutments. Bedrock is generally competent basalt.

Pore water salinity measured from selected fine-grained soil samples typically ranged from 30 parts per thousand (ppt) to 50 ppt. The pore water salinity of the sand deposit is typically 4 ppt or less and is considered non-saline.

5.3.3 South Dam

The South Dam will be situated about 400 m south of Tail Lake, on the watershed with Ogama Lake. The proposed dam alignment is along a flat valley section that remains above elevation 33 m (Dwg. T-05).

Ground temperature and pore water salinity measurements at the South Dam alignment are consistent with those observed at the North Dam.

The inferred stratigraphy along the South Dam alignment is illustrated in Dwg. T-06. Soil conditions typically consist of marine silt and clay overlying gravel. The marine deposit, which is up to 20 m thick at the base of the valley is ice-saturated and contains excess ground ice. The gravel layer is up to 15 m thick. The overburden soils thin out at the dam abutments. Bedrock consists of basalt and agrillite and is typically competent.

5.3.4 Spillway

The spillway will be situated immediately north of the North Dam alignment. Permafrost overburden along the spillway alignment is generally less than 3 m thick, and consists of predominantly sand, although there are some ice-rich silt and clay pockets. Bedrock consists of basalt and is typically competent.

5.4 Dam Design

5.4.1 North Dam

Based on the proposed operating methodology, tailings will be deposited in the deepest sections of the TCA, stating at an elevation of about 22 m and ending at about 24.3 m, as compared to the normal water level in Tail Lake at elevation 28.3 m. Only water will be in contact with the North Dam. The North Dam has therefore been designed as a water retaining structure utilizing a central frozen core with a geosynthetic clay liner (GCL) installed against the upstream side of the core, as shown on Dwg. T-03. The frozen core and GCL are required to an elevation of 35.3 m to provide engineering and environmental containment under extreme operating conditions. The dam will be constructed in a single construction season because of the requirements to keep the core perennially frozen.

The core material would be composed of on-site processed, i.e. crushed quarry rock. The frozen core would be keyed and frozen into the central key-trench (Dwg. T-02 and T-07). A transition layer consisting of larger crushed quarry rock will be placed against the frozen core to protect it from the outer run of quarry shell. A system of horizontal thermosyphons installed within the key trench has been incorporated in the dam design to minimize long-term deformations due to creep-induced settlements over the life of the dam.

Although the frozen core is intended to prevent seepage, the GCL will provide secondary water retaining capability in case the core develops cracks due to thermal expansion and contraction, as well as creep deformation and settlement. The core (and GCL) would extend 1.8 m above the FSL and the outer shell will be at least 2.2 m thick to provide insulation to the frozen core.

The upstream and downstream slopes of the dam will be 6 to 1 (horizontal to vertical) and 4 to 1 respectively. These slopes have been designed to ensure physical and thermal stability against creep deformation induced by long-term loading of the dam structure over the deep ice-rich saline foundation soils.

5.4.2 South Dam

The design of the South Dam is essentially identical to the North Dam, with the exception of additional allowance for creep deformation, due to the thicker extent of ice-rich, saline permafrost soils beneath the dam. Dwg. T-05 illustrates the South Dam cross-section. The frozen core (and GCL) is required to an elevation of 35.8 m, and the final crest height will be at 38 m. Both the upstream and downstream dam slopes will be 6 to 1 (horizontal to vertical).

The intended operating water level in the TCA will be at elevation 29.2 m, which is below the lowest part of the South Dam at 33.0 m. Furthermore, even the most conservative water balance predictions suggest that it would take at least 4 years before the TCA water level reaches 33 m (Dwg. T-01). The construction of the South Dam should therefore be delayed until such time that the latest water balance predictions, under the most conservative assumptions suggest the water level in the TCA would reach elevation 33 m within two years. This would allow more than enough time to mobilize equipment and supplies to construct the dam.

5.5 Thermal Analysis

5.5.1 General

Thermal analyses were performed for the North and South dams because they are primary water retaining structures which rely on maintaining the dam core and permafrost foundation perennially frozen to provide containment.

A summary of the thermal analysis is provided here. Appendix B provides complete details of the thermal modeling, which has been carried out by EBA Engineering Consultants Ltd. (EBA). To prevent misinterpretation of information, many of the summaries below have been paraphrased from the EBA report.

5.5.2 Method of Analysis

Thermal analysis was carried out using a proprietary finite element computer program, GEOTHERM, developed by EBA. The model was calibrated for site specific conditions, using site climatic data for 2004-2005, and select thermistor and borehole data collected at both dam alignments. The results of the thermal model calibration, detailed in Appendix B, showed good agreement between the predicted and measured ground temperature profiles at both dam alignments. Therefore, the selected input parameters were considered reasonable and the thermal model was judged to be appropriate for thermal design of the dams.

Thermal analysis was carried out for dam cross sections near the deepest sections of each of the North and South Dam alignments. These selected soil profiles are considered the worst foundation conditions along each proposed dam alignment from a creep-deformation perspective.

Thermal analysis was carried out to model every step from dam construction through subsequent TCA impoundment. It was assumed that both the North and South Dams would be constructed the same season. The thermal analysis conservatively assumed that the water level in the TCA would be at elevations 31 m and 33.5 m by the first and second freshets (assumed to be June 1), respectively, following construction of the dams. Furthermore, it was assumed that from that point forward the dam would remain at FSL for the remainder of the 25-year design life.

The Doris North dams have been designed assuming global warming over their 25-year design life as the normal environmental condition. The effects of two extreme warm years immediately following dam construction have also been evaluated as an upset condition.

The design of the dam includes a series of horizontal thermosyphon loops which will ensure that the core and foundation soils beneath the core are sufficiently cold to limit creep-induced deformations and to be impervious to seepage.

5.5.3 Results from Thermal Analysis

Complete figures demonstrating the ground temperature distributions during the month of December in Years 1, 2, 5, 10 and 25, respectively following construction are presented in Appendix B, for both the North and South Dams. December is the time of the year when temperatures below the frozen core are the warmest.

North Dam

The results show that upstream of the frozen core, the permafrost foundation is progressively getting warmer over the years in response to the continued warming influence of the TCA water level. Assuming the marine clay and silt layer fully thaw at -3°C, much of the marine clay and silt upstream of the core is predicted to thaw after 25 years of full impoundment. Downstream of the frozen core, the permafrost foundation is predicted to progressively warm due to the long-term effects of snow drifting and the modelled climate warming trend. The degree of permafrost warming below the downstream shell is not expected to be as great as below the upstream shell.

Beneath the frozen core, permafrost temperatures are predicted to progressively cool with time over the first ten years or so due to the thermal influence of the thermosyphons, but then become warmer because of long-term climate warming. The results show that the frozen core is well frozen (temperatures mainly below -5°C), as is the permafrost foundation beneath the core (temperatures colder than -8°C).

An upset condition was evaluated by assuming extreme warm years for the first two years following initial impoundment. These events have a greater influence on the dam embankment temperatures (e.g. the active layer penetration) than on the permafrost foundation temperatures, and as a result the thermal stability design criteria for the dams can still be maintained.

South Dam

The long-term geothermal response of the South Dam embankment and its permafrost foundation is similar to that described for the North Dam above.

5.6 Creep Deformation Analysis

5.6.1 General

Strains will develop within the dam embankments and underlying permafrost foundations in response to long-term creep deformations. A summary of the deformation analysis is provided here. Appendix B provides complete details of the deformation assessment.

5.6.2 Method of Analysis

The commercial, two-dimensional, explicit finite difference stress analysis program FLAC, developed by HCItasca, was used to carry out a two-dimensional plane-strain analysis. The analysis was carried out using the same dam section and foundation conditions used in the thermal evaluation.

5.6.3 Results from Creep Deformation Analysis

Complete figures demonstrating the results of the deformation analysis are presented in Appendix B, for both the North and South Dams.

Substantial settlement of both dam cores is predicted; however, it should be noted that the predicted creep deformations are considered to be very conservative. Based on these results, the design approach has been to initially over-build the crest of the frozen core of the dams to accommodate some, but not all, of the predicted settlement over the 25-year design life.

North Dam

Appendix B contains figures that show the predicted horizontal and vertical displacements, ten years after construction of the North Dam. Predicted deformations are largely away from the frozen core, whilst the core itself is expected to behave rigidly.

The predicted maximum shear strain and shear strain rate, ten years after construction, show that the frozen core and permafrost foundation are expected to remain ductile, and brittle rupture is not expected. The most likely mechanisms of slope failure are deep-seated, rotational slips located upstream and downstream of the frozen core.

Ten years after construction, the frozen core and underlying permafrost foundation soils will have strengths that far exceed the predicted shear stresses on the dam.

South Dam

Deformation trends at the South Dam are similar to those at the North Dam. Although the dam is thinner and is designed to retain a lower head of water than the North Dam, even greater creep deformations are predicted at the South Dam because the creep-susceptible marine clay and silt foundation is much thicker at this dam alignment than it is at the North Dam alignment.

Conclusion

The creep deformation analyses indicate relatively high movement and strains in the foundation upstream and downstream of the frozen core, and relatively small movements and strains in the dam core and underlying foundation soils. The strains are predicted to occur very slowly and in a ductile manner. With monitoring of the actual dam displacements, potential stability concerns can be identified and mitigation measures can be put in place. Therefore acceptable dam performance is anticipated.

5.7 Stability Analysis

5.7.1 General

Limit equilibrium analysis were carried out to determine the factor of safety against slope failure during construction and operation of the North and South Dams. Again, complete details are presented in Appendix B.

5.7.2 Method of Analysis

The commercially available two-dimensional, limit equilibrium software, SLOPE/W, developed by Geo-Slope International Ltd. was used to conduct the stability analysis. The stability analysis were carried out for the dam sections near their maximum thickness, i.e. the same geometry and soil profiles used in the thermal and creep deformation analysis. Two cases were evaluated: one assuming full impoundment (i.e. water level at 33.5 m), and another assuming no water against the dam. A deep-seated slip surface and failure along the GCL were evaluated.

5.7.3 Results from Stability Analysis

Minimum factors of safety were determined from cases where there was no water impounded against the dam. This is because in total stress analyses, the weight of the water acts as a resisting load against slope failures beneath the upstream slope.

The minimum factors of safety calculated are for deep-seated, circular slip failures from the opposite slope (e.g., from the downstream dam slope for the upstream slip surface), through the marine silt/clay layer and daylighting beyond the slope toe. The calculated factors of safety satisfy dam safety requirements.

5.7.4 Liquefaction Potential

The peak horizontal ground acceleration for the area is very low at 0.06 g and, as a consequence, liquefaction of the thawed marine silt and clay due to earthquake loading is not expected to be a concern.

5.8 Dam Settlement Analysis

Thaw settlements were estimated based on the predicted thaw penetration into the typical cross section described in the thermal evaluation. In the extreme condition where the dams continuously retain water at its FSL over a 25-year period, up to 6 m of the frozen saline fine-grained soils is predicted to thaw below the upstream shell; however, below the core, the permafrost is predicted to remain well-frozen. Thaw settlements are therefore not expected to affect the integrity of the frozen core.

Deformation is predicted to occur due to permafrost creep and thaw and, to a lesser extent, consolidation of the marine clay and silt foundation soils. Given the variability in soil conditions along each dam alignment, there is a potential for differential movements across the dam embankment. This is particularly true at the North Dam, where most of the dam is sited on non-saline, ice-poor frozen sand and displacements are expected to be relatively small, compared to the remaining portion of the dam, which is sited on saline clays and silts, over which larger displacements are predicted.

Predicted deformations are considered conservative and are expected to develop slowly. Dam thermal and displacement data should be regularly reviewed to verify that the dams are behaving as predicted. Remediation measures, such as flattening slopes or installing sloped thermosyphons, should be implemented should differential dam movements be seen to pose a risk of rupture of the frozen core and/or permafrost foundation.

5.9 Seepage Control

The control of seepage at the TCA is dependent on the integrity of its two containment dams, and the extent of the talik beneath the TCA.

Extensive thermistor data along the North and South Dam alignments confirm that there are no talik zones present, even under the Tail Lake outflow. A series of shallow thermistors (up to 12 m depth) around the TCA perimeter between elevations 28 m and 33.5 m, show no indication of an extended Tail Lake talik. Furthermore, deep thermistor data (up to 50 m deep) along the Tail Lake watershed, including two locations between Tail and Doris Lakes, confirm that there is no talik between these lakes for at least 50 m depth, with the permafrost temperature at that depth still cold at -8°C. Finally, a 200 m deep drill hole (SRK-50), only 100 m from the Doris Lake shoreline, confirm -4°C permafrost even at that depth. Therefore, to the extent that the frozen cores of the various dams are thermally contiguous, with the natural permafrost, the seepage losses should be non-existent.
SRK (2005b) presents a detailed discussion of the hydrogeological conditions at the site.

Conventional seepage analysis is not well suited to estimate seepage through frozen core dams. However, for water balance purposes only, it was assumed that seepage from the TCA will be about 0.9 L/s (Appendix F). This amount of seepage may be attributable to slight imperfections in the seepage control system as a result of, for example, minor construction flaws or imperfect freezing at select locations.

This allowance for seepage is deemed very conservative considering the expected maximum TCA operating level water is expected to be at elevation 29.2 m, which implies a maximum hydraulic head at the North Dam of 3.2 m, and zero hydraulic head at the South Dam. Even at the FSL, the maximum hydraulic heads at the North and South Dams respectively are 7.3 m and 0.3 m.

The toe areas at both dams will be monitored. To the extent that any seepage is observed, and depending on the amount and water quality of such seepage, consideration will be given to collecting this seepage in sumps and pumping it back to the TCA.

6 Water Balance

6.1 General

The water balance formed the basis for determining the dam design height, and water quality predictions for the operational, closure and post-closure periods. The detailed description of the overall water balance is provided in Appendix F. This section provides a brief overview of the TCA water balance. Primary assumptions that were adopted for the water balance include the following:

- The TCA will be completely isolated with respect to surface and groundwater from the adjoining Doris and Ogama Lake catchments by two water retaining structures; the North and South Dam respectively.
- Tailings deposition will be sub-aqueous and will be managed such that the final tailings surface will be relatively horizontal.
- Tail Lake will not be pumped out prior to constructing the dams or starting deposition. The volume of Tail Lake at its normal elevation of 28.3 m is about 2.2 million m^3.
- Annual discharge (decant) release from the TCA are planned; however, the TCA is designed to accommodate full containment (tailings and all natural runoff) for the two-year mine life, plus an additional period after mining ceases.
- The water balance is calculated in monthly time steps. The calculations use a year that starts in March and ends in February.
- The impact of varying climate and hydrology on the water balance has been evaluated.

6.2 Water Balance Calculation

The key input assumptions on which the water balance is based are repeated below:

- *Total precipitation:* The water balance is conducted using average climatic year data; however, it is recognized that extreme events can affect the outcome. The water balance sensitivity analysis therefore includes an evaluation of extreme wet and dry years. The average annual precipitation (rainfall and snow water equivalent) is about 207 mm.
- *Potential lake evaporation:* The average lake evaporation has been determined to be about 220 mm per year. The sensitivity analysis evaluated evaporation to ±20% of this value.
- *Water yield:* For the purpose of the water balance, the base case water yield was conservatively assumed to be 180 mm, and the effect of lower water yields (111 mm) were evaluated through sensitivity analysis.
- *Seepage:* Seepage from the TCA can be via three primary routes; North Dam, South Dam and deep recharge through the lake basin. In reality, the North and South Dams will be frozen core dams, which should not have any seepage. Average condition theoretical seepage calculations

are described in the water balance calculation and have been used in the TCA water balance. It was however assumed that all seepage from the North and South Dams would be intercepted and pumped back to the TCA. The average deep seepage rate is so low that it has been omitted from any water balance calculations.

- *Tailings slurry feed:* The average steady state tailings production rate will be about 648 tonnes per day. The specific gravity of the tailings solids will be 2.7. The tailings slurry will be discharged at about 39.6 % solids, and will have a submerged in-place tailings void ratio of 1.2. This will result in a daily slurry feed of 1,634 m^3 (648 m^3/day solids and 986 m^3/day water).
- *Reclaim water:* The TCA is relatively shallow; a reduced water volume created by the freezing conditions (ice) may prevent water recovery for reclaim. Consequently, 100% recirculation water (986 m^3/day) is assumed for four months of the year only (June through September). During the remainder of the year fresh water make-up will be from Doris Lake.
- *Sewage sludge:* The sewage treatment plant outflow will be pumped to the TCA as part of the tailings feed stream. We assumed a 175-man camp, for a total sewage treatment plant load of about 68.6 m^3/day.
- *Underground mine discharge:* Although the Doris North Project would in all likelihood not experience any mine water inflow (SRK 2005b), a conservative assumption has been made that a mine inflow of 235m^3/day would occur for the life of the Project. This water would be captured in the mine and pumped to the TCA.
- *Discharge (decant) rate:* It was assumed that while the TCA is actively managed, annual discharge from the TCA will occur directly into Doris Creek. The allowable rate of discharge would be limited to maintain receiving water quality objectives in Doris Creek (SRK 2007c). The rate of discharge will be determined based on actual water quality in the TCA and Doris Creek, and the flows in Doris Creek.

6.3 Water Balance Results

The primary purpose of the water balance was to determine an appropriate height for the containment dams, such that there would be sufficient storage capacity in the TCA. Based on the water balance it was determined that an optimal design FSL for the TCA would be 33.5 m. Under the most conservative water balance assumptions, the TCA can operate as a zero discharge facility for just under 5½ years before reaching the FSL. Using more realistic water balance assumptions the TCA can operate as a zero discharge facility for at least 7½ years.

The water balance also illustrates that, by allowing an annual discharge the time to reach FSL in the TCA is dramatically increased. Allowing as little as 100,000 m^3/year of discharge increases the time to FSL under the base case to just under 9½ years, which is a 27% increase in storage time. If the annual discharge is 500,000 m^3/year, the FSL in the TCA will likely not be reached, since the decant rate will exceed the annual inflow.

7 Design of the Water Management Facilities

This section deals with the design of the facilities required to perform the tailings disposal and manage the supernatant within the TCA.

7.1 Tailings Deposition Infrastructure

Tailings is delivered to the TCA via a 4,630 m long single 127 mm diameter pre-insulated pipeline (Dwg. T-11 and T-12). Tailings pumps are installed at the mill and have duty point of 20 L/s at 263 m of total dynamic head. The pipeline is designed to convey 20 L/s of tailings with a solids concentration of 39.6 % by weight. Design tailings relative density is 1.3 tonne/m^3. The flow velocity in the pipe of 1.65 m/s must be maintained to prevent solids settling.

The pipeline follows the secondary road ensuring accessibility to the pipeline for monitoring and maintenance. The pipeline will be placed directly onto the road surface, towards one side with no physical separation between the pipeline and the traffic. The only section of the pipeline that will have spill containment is a 150 m long section where the pipeline crosses Doris Creek. Along this section, a double wall pipeline will be used.

The pipeline has been designed to have a minimum longitudinal grade of at least 1% at any location in order to ensure gravity draining of the pipeline towards five low points where emergency dump catch basins are located. Stoppage of the pipeline whilst full of slurry will lead to solids settling and/or line freezing. Therefore, during stoppage the pipeline will be drained into the emergency dump catch basins. Valves located immediately above each emergency dump catch basin will be manually opened to allow gravity drainage of the tailings line. Evacuated tailings will be collected and temporarily stored in the emergency dump catch basins. The emergency dump catch basins are HDPE lined to ensure containment, and have been sized to allow two sequential fillings plus an additional 0.5 m of freeboard (Dwg. T-13). Emergency dump catch basins outside of the TCA catchment is sized to allow containment of the reclaim water as well.

Tailings will be deposited sub-aqueously into the TCA using a floating line in the summer and a line through the ice-pack in the winter. Tailings will initially be deposited in the deepest sections of the TCA and the deposition points will be moved regularly to ensure that tailings are equally spread with minimal highs and lows. A total of five tailings deposition pipes are provided to allow for flexibility in tailings deposition and facilitate even deposition of tailings solids. Only one deposition pipe will be operational at a time.

7.2 Process Water Reclaim Infrastructure

Maximum recycle of water is the objective; however, since the TCA is shallow, and up to 2 m of ice every winter will further reduce capacity, there is a possibility that reclaim water may only be recovered from the TCA during the open water summer months. The reclaim pipeline has subsequently been designed to operate year round, for only four months of the year, or any other time period as applicable.

A reclaim barge on the TCA will house a vertical turbine reclaim pump with a design duty point of 14 L/s at 63 m of total dynamic head. The reclaim pipeline will be a pre-insulated and heat traced 102 mm diameter HDPE line. The pipeline will be about 1,800 m long. The pipeline will follow the secondary access road to the mill, and will be placed on the road shoulder immediately adjacent to the tailings delivery line.

The reclaim line will also have drain valves at the emergency dump catch basins. Where the emergency dump catch basins are outside of the TCA catchment, the reclaim line will drain into the emergency dump catch basins. However, where the emergency dump catch basins are within the TCA catchment, the reclaim line will be drained out directly onto the tundra downstream of the secondary road.

7.3 Fresh Water Make-Up Infrastructure

Since there is a real possibility that reclaim water may not be obtained from the TCA year round, a contingency fresh water make-up system is provided to draw water from Doris Lake. A pump will be installed on the float plane dock laydown area. The pump will have a duty point of 14 L/s at 45 m of total dynamic head. The intake pipe will be placed on the lake bottom and will terminate at a depth of 5 m. The intake pipe will be a 25 m long, 102 mm diameter pre-insulated and heat traced HDPE line. The intake pipe will be held in place with a cover of quarry fill. The pipe inlet will have an approved fish screen.

The fresh water make-up pipeline will be placed directly onto the float plane dock access road shoulder, with no physical separation from road traffic.

7.4 Operational Discharge (Decant) Infrastructure

The discharge system will comprise the installation of a control system that will accurately control and measure the discharge flow rate over a flow range spanning 50 L/s to 275 L/s. A programmable logic controller (PLC) will be used to both control the discharge rate as well as log instantaneous flow rates and cumulative discharge volumes. The flow would be controlled with an actuated flow control valve, with excess flow recycled back to the TCA. The PLC will actuate the flow control valve to discharge TCA water at a fixed ratio, equal to the target discharge rate (TDR), relative to the flow in Doris Creek.

The pump intake in the TCA (for the operational period) will be mounted on a floating barge (same system as the reclaim pump) well away from the tailings discharge point to minimise suspended solids in the intake. Silt curtains will be installed around the pump intake to minimise intake of suspended solids.

A vertical turbine pump will be located on the barge with a duty point of 275 L/s at 29 m total dynamic head. The pipeline up to the pump house pad will be about 500 m long and will be a 405 mm (16”) HDPE pipe (not insulated or heat traced, since discharge will only be done during the summer months). In the pump house a throttling valve will be used to regulate the actual discharge flow down to 50 L/sec if required. The excess will be drained back to the TCA.

From the pump house, a 900 m long gravity pipeline will convey the discharge to the decant point. This pipeline will be a 355 mm (14”) HDPE line (not insulated or heat traced). The pipeline will be placed directly onto the shoulder of the secondary and decant access roads, immediately adjacent the tailings discharge and reclaim pipelines.

The discharge to Doris Creek will be located sufficiently downstream from the flow monitoring location to ensure that the discharge will not interfere with flow measurements in Doris Creek, but sufficiently upstream of the waterfall to ensure complete mixing with Doris Creek water. The outlet would be placed such that the discharge flow would not lead to erosion or degradation of the creek bed.

7.5 Spillway

An operational spillway has been designed for the TCA at the North Dam, at elevation 33.5 m. This side-spillway will be 18 m wide, about 190 m long along its centerline, with an average gradient of about 0.8% (Dwg. T-08). The design flood of 3.3 m^3/s will pass through the spillway with a maximum flow depth of about 0.2 m at a sub-critical flow velocity of about 1.1 m/s.

The most conservative water balance estimates for the TCA, suggest that the FSL of 33.5 m will only be reached about five years after construction of the North and South Dams. Furthermore, since the anticipated maximum operating water level in the TCA is at an elevation of 29.2 m, there is a very real possibility that the spillway may never actually be required.

Details regarding the predicted timing of spillway construction are presented on Dwg. T-01, which suggest that under the most conservative water balance predictions the spillway should be constructed four years after completion of the North Dam. This provides 1½ years to mobilize a contractor and implement the design, before the spillway would be required. It is SRK’s Professional opinion that this timeframe is more than adequate.

Irrespective of when the spillway is constructed, it should be pointed out that under no circumstance will water that does flow over the spillway exceed MMER water quality. Complete details of the water quality in Tail Lake, and how that will relate to any discharge from the spillway is presented in SRK (2007c).

7.6 Shoreline Erosion Protection

The proposed water management strategy for the TCA will result in a maximum water level of 29.2 m, with a total flooded footprint of about 13 ha. Prior to tailings deposition, about 20% of this surface area will be pro-actively mitigated. Areas to be pro-actively mitigated are indicated on Dwg. T-14 and are areas that are subject to the greatest fetch distances.

Pro-active erosion protection will consist of laying a geotextile directly onto the tundra and covering it with a 0.5 m thick layer of run of quarry material as illustrated in Dwg. T-14. If during operation, and prior to final closure, there is any physical evidence of shoreline erosion, repairs should be carried out as per the details presented in Dwg. T-14.

At final closure, i.e. as soon as the water level is lowered to 28.3 m, the entire flooded perimeter of the TCA shoreline should be inspected and in areas where visible erosion is seen to occur, shoreline erosion protection must be put in place.

A comprehensive evaluation of the shoreline erosion processes, risks and mitigation measures are documented in SRK (2005e), which is also included as an appendix to SRK (2007c).

A complete Shoreline Erosion Protection Adaptive Management Plan is included as Appendix G.

8 TCA Construction

8.1 Construction Materials

8.1.1 Dams, Spillway and Shoreline Erosion Protection

Construction fill material for the dams, spillway and shoreline erosion protection consists of core, transition and run of quarry material. This granular fill will be produced on site from one of four local quarries. Complete geological, mineralogical and geochemical details on these quarry sites are documented in MHBL (2003), AMEC (2003b), and SRK (2006c). The grain size distribution envelopes for all the construction fill is presented in Dwg. G-05.

Other materials that will be used to construct the dams, spillway and shoreline erosion protection measures include; GCL, thermosyphons and geotextile. Complete details of all these materials are provided in the Technical Specifications (SRK 2007b); however, for completeness a brief summary is presented below:

- *Core material:* the dam frozen core will be constructed with this well graded processed crushed rock with a maximum particle size of 20 mm. Since each lift of this material will have to be frozen prior to the placement of the next lift, it is important that this material contains sufficient fines to provide water retaining capabilities for the duration of the freezing process.
- *Transition material:* The transition material will be used immediately adjacent to the core material on the dam, and will act as a filter layer between the core and the outer shell. This material will be well graded, screened crushed rock with a maximum particle size of 150 mm.
- *Run of quarry material:* This material will be used to construct the outer shell of the dams, as well as armouring for the spillway and for the shoreline erosion protection. It will consist of run of quarry rock and have a maximum size of 500 mm. Fabrication of this material will be dependent on the condition of the rock and the blasting procedure at the quarry.
- *Impervious membrane (GCL):* Secondary containment for the dams will be provided by the GCL. The GCL will consist of a pre-manufactured three layer assembly of sodium bentonite enclosed between two geotextiles (one woven and one non-woven).
- *Thermosyphons:* Additional cooling of the dam core and foundation over the 25-year design life of the dams will be provided by horizontal looped thermosyphons. A thermosyphon is a hollow pipe filled with pressurized carbon dioxide (CO_2) that evaporates and condensates depending on the ambient air and ground temperatures. It essentially consists of two main components; the evaporator and the condenser/radiator. The evaporator is the portion of the thermosyphon that is buried in the ground where the heat is extracted from the ground, i.e. where cooling occurs. The radiator is the component that is installed above grade and is generally installed in a vertical position. The section joining the radiator and the evaporator is called the riser. The radiator is covered with protruded fins that increase the surface area over which heat is exchanged between

the thermosyphon and the ambient air. Thermosyphons are active only when the ambient air is colder than the ground surrounding the evaporators (buried portion). The heat extraction simply ceases over the period (summer) when the ambient air temperature is warmer than the ground temperature.

- *Geotextile:* Non-woven geotextile is used as a filter layer beneath the run of quarry fill for shoreline erosion protection. The geotextile acts as a filter layer should erosion occur.

8.1.2 Pipelines and Emergency Dump Catch Basins

Complete details of the construction materials required for the pipeline installations and the emergency dump catch basins are provided in the Technical Specifications (SRK 2007b); however, for completeness, the main items are summarized here:

- *Tailings deposition pipe:* This pipeline will consist of six different pipe classes, all having a nominal diameter of 127 mm. The first segment of the pipeline will be an HDPE lined steel pipe, whilst the remaining five segments will be HDPE pipe with different pressure ratings. All pipe segments will have insulation. The steel pipe will be site welded, whilst the HDPE pipes will be fusion welded on site. All joints to valves and/or bends will be flanged.
- *Reclaim and fresh water make-up pipes:* These pipes will have different pressure ratings, but will both be 102 mm diameter pre-insulated and heat traced HDPE lines. The pipes will be fusion welded on site with flanged couplings at bends and valves.
- *Discharge pipes:* The discharge pipe from the barge to the pump house will be a 405 mm (16”) HDPE line, and the gravity pipeline from the pump house to the decant point will be a 355 mm (14”) HDPE line. Both pipes will be fusion welded on site with flanged couplings at bends and valves.
- *HDPE liner:* Primary containment in the emergency dump catch basins will be provided by a 57 mil HDPE liner sandwiched between two non-woven geotextiles.
- *Floating barge:* A floating barge will be required to house the reclaim and discharge pumps on the TCA. Access to the floating barge will be via floating walkway.

8.2 Construction Equipment

Construction of the dams can be achieved, in most part, using conventional earthworks equipment. Trucks, loaders, graders, excavators, smooth roller compactors and track-mounted bulldozers would not require special modifications other than to be capable of operating continuously in extremely cold weather.

Conventional rock quarry development equipment will be used to produce the construction fill material. This includes track-mounted air drills, crushers and screening equipment.

The placement of the core material will require some special attention. Since the material has to be placed under arctic winter conditions and with a sufficiently high moisture content that it freezes to a nearly ice-saturated condition, a mobile heating plant, capable of heating the crushed rock and a mobile mixer capable of controlling the proper dosage of water, would be required.

Placement of the GCL and the geotextile poses no special problems, whilst the seaming of the HDPE liners and fusion welding of the HDPE pipelines will require heating tents over the joint areas to ensure proper bonding.

8.3 Construction Quality Control and Quality Assurance

Complete details of the Quality Assurance and Quality Control (QA/QC) procedures to be followed for the construction activities are provided in the Technical Specifications (SRK 2007b). Quality Control will be the responsibility of the Contractor, and/or the equipment and materials manufacturer. The Engineer of Record, which will be a Registered Professional Engineer in the Nunavut Territory, will carry out Quality Assurance. Complete documentation of all QA/QC data will be provided in the relevant As-Built Reports.

8.4 Construction Quantities

Construction quantities have not been generated as part of this report. Quantities will form part of Schedule 1 to the Technical Specifications (SRK 2007b).

8.5 Construction Procedure

8.5.1 Further Field Characterization

SRK is satisfied that sufficient and appropriate field characterization has been done in support of the dam designs presented in this report. However, as is normal in dam construction, field verification of conditions will be done as construction starts by means of a program of test pits, and actual key trench excavation. There is however no unforeseen circumstances anticipated that would result in any significant deviation of the designs as presented in this report.

8.5.2 Dams

Most of the dam construction activities must occur during the winter months. Placement of the dam core material will require an ambient air temperature sufficiently cold to maintain frozen core conditions. A summary of the construction steps are provided below:

- Strip the organic and peat cover within the dam footprint.
- Excavate the key trench and prepare the abutments. The key-trench must be at least 2 m deep, or until competent bedrock or ice saturated permafrost is encountered. Key-trench excavation is likely to require drill and blast excavation techniques, combined with conventional mechanical excavation. Final cleaning of the key-trench will be done with compressed air and hand-cleaning. The excavated material will be stockpiled in a suitable area for potential future use.

- The abutments will be cleared and stripped to assure a good bond between the core of the dam and the overburden or bedrock. Blasting in the bedrock may be required at the abutments to key in the core.
- Six looped thermosyphons will be installed along the base of the key-trench. Three loops will be located on either side of the dam alignment, extending from the lowest point in the key-trench. Each pair of looped pipes would cover opposing halves over the entire length of the dams, spaced at 1.5 m intervals. Each loop shall be connected to a vertical radiator, such that there will be three radiators on each dam abutment.
- The GCL will be placed on the thermosyphon loops and the construction of the core will start. Care will be taken to advance the GCL with the construction of the core material.
- The core material must be placed at sufficiently high moisture content that complete ice saturation can be achieved, and that the layers can be compacted to at least 90% of its maximum dry density. Care must be taken to ensure that the compacted layer is ice-free at the time of placement. Lift thickness and compaction details will be determined on site based on site specific trials as stipulated in the Technical Specifications (SRK 2007b). Subsequent lifts may only be constructed once the previous lift has completely frozen and the surface is cleared of snow, ice and loose material.
- The transition and shell materials can be placed using material directly from stockpiles, with no special pre-conditioning. The lift thickness and compaction methods will again be determined on site based on material specific compaction trials.
- Monitoring instrumentation will be installed as part of the bulk earthworks. These instrument clusters will have to be protected from damage at all times, and must be continuously tested to ensure that they are operational. Special hand compacting equipment may be required to work in close vicinity to these instrument clusters.

It should be reiterated that although complete design details have been provided for the South Dam, the actual construction of the South Dam should be delayed until water balance predictions suggest that the dam construction is required. Once construction of the South Dam is commissioned, the procedures will be identical to what has been described above.

8.5.3 Spillway

As has been described previously, there is a real possibility that the spillway may never be put to use, and certainly, for at least 5½ years, the spillway will not be required. Therefore, although this report and all supporting documents provide complete detailed design information for the spillway, the spillway should not be constructed at the outset. Spillway construction should be delayed until such time that water balance prediction suggest that it would become necessary.

Irrespective, of when the spillway will be constructed, the construction should be carried out as follows:

- Clear, strip and stockpile the overburden soils to expose the bedrock along the spillway alignment.
- Drill and blast the bedrock to the required line and level for the spillway invert.
- Over-excavate and fill areas of the spillway that is not bedrock controlled, and clad with erosion resistant armouring.
- Over-excavate and fill exposed overburden cut slopes of the spillway to prevent permafrost degradation.

8.5.4 Pipelines

The installation of all pipelines must be done according the manufacturers specification and the Technical Specifications (SRK 2007b). Specialist contractors must be used to carry out pipe welds as well as seaming of the emergency dump catch basin liner.

8.5.5 Shoreline Erosion Protection

Placement of the shoreline erosion protection should be done as follows:

- Stake out the areas that require shoreline erosion protection and clear the snow from those areas.
- Place the geotextile directly onto the tundra, taking care to allow at least a 0.5 m overlap between layers.
- End dump and spread the run of quarry material, such that the finished surface is about 0.5 m thick, and has a smooth top surface that mimics the natural ground topography. The upslope perimeter of the erosion protection material must be shaped such that no natural runoff would be ponded behind the erosion protection layer.

9 TCA Operation

9.1 Dams

The dams have been designed to be fully operational for at least a 25-year design life, provided all the appropriate monitoring and maintenance are carried out (see next sections of this report). Since there are no active elements to the dams, there are no special operational procedures associated with the dams.

9.2 Spillway

If the TCA operates the way it is intended, the spillway will hopefully never have to be constructed. However, if it is constructed it has been designed as a passive spillway with no special operational requirements other than ensuring that the opening is not blocked as result of an ice jam. There is however, more than sufficient storage capacity in the TCA that even if an ice jam does occur, and zero outflow is allowed for the design flood, then the water level in the TCA will still be below the normal freeboard height of the North Dam, i.e. there will still be 100% containment with no risk to the dam integrity.

9.3 Tailings Deposition

Tailings deposition will be a continuous operation. The tailings slurry will be pumped from the mill to the TCA and deposition into the TCA will be via one of five discharge points along the eastern boundary of the TCA. A pre-planned deposition plan will be followed to ensure that the tailings surface remain as horizontal as practical, with manual changeover of the tailings deposition point as required.

When the TCA water surface starts to freeze, the discharge lines will be pushed through the ice such that tailings under no circumstances are deposited on the ice.

If for any reason tailings production stops, the tailings discharge line will be kept operational by recirculating reclaim water. This will prevent the need to dump the tailings line content into the emergency dump catch basins. If the tailings line must be decommissioned for any reason, the line will be flushed with reclaim water followed by fresh water. Once the pipe is filled with fresh water only, the line can be dumped, but the emergency dump catch basins can be bypassed.

When tailings deposition stops and the emergency dump catch basins are used to dump either tailings slurry or reclaim water, the emergency dump catch basins must be emptied out as soon as possible, preferably before the spill freezes. If the spill does freeze, a heating tent must be placed over the emergency dump catch basin, and the spill must be thawed and either pumped out or mechanically loaded onto a dump truck for disposal back to the TCA. Even though the emergency dump catch basins have been designed with sufficient capacity to contain at least two dumps plus an additional freeboard volume, cleanup should commence immediately after every spill.

9.4 Reclaim and Fresh Water Make-Up

The intent is to recover all reclaim water from the TCA. Therefore, for as long as there is sufficient water depth and the TCA supernatant at the reclaim pump inlet is sufficiently clear of suspended solids the reclaim pumps will remain active. If at any point the amount of reclaim from the TCA is no longer sufficient, the rest of the make-up water will be pumped from the fresh water make-up line in Doris Lake.

Whenever either one of these lines are decommissioned, they will be completely flushed with fresh water before being drained. Once flushed with fresh water the reclaim line does not have to be drained into the emergency dump catch basins, but can be diverted directly onto the tundra.

If for any reason tailings deposition stops, the reclaim line pumping will continue such that the reclaim water can be re-circulated through the tailings discharge line. This will prevent unnecessary dumping into the emergency dump catch basins.

9.5 Discharge (Water Management Strategy)

The discharge (primary water management) strategy has been developed based on the results of a detailed water quality model for the TCA as documented in SRK (2007c). This section summarises the basic principles of the water management strategy. The primary objective of the TCA water management strategy will be to meet CCME guidelines for parameters of concern to protect freshwater aquatic life in Doris Creek, with the possible exception of nitrite. The key management and control components of the proposed discharge strategy will comprise:

- Real-time monitoring of flows in Doris Creek.
- Monitoring of water quality in Doris Creek and the TCA on a frequent basis.
- Managing the decant intake in the TCA to minimise suspended solids release.
- Use of the water quality results to determine allowable discharge rates.
- Controlling the discharge flow rate on a real-time basis.

The discharge strategy will be implemented as follows:

- Prior to commencement of milling, a low detection environmental laboratory will be set-up and analytical procedures developed, documented and verified. Sampling protocols will also be documented and verified.
- Two weeks prior to commencement of operations (assuming a spring start-up), water quality in the TCA and Doris Creek will be monitored every second day to establish baseline conditions.

- Real-time monitoring of the flows in Doris Creek will commence as soon as practical during the open water season. The pressure transducer would be connected to a programmable logic controller (PLC) that would record flows in Doris Creek and be used to control the discharge flow rate.
- During periods of active discharge, the flow level in Doris Creek will be monitored visually on a daily basis and checked against the real time monitoring results. For this purpose, a staff gauge will be installed at the location where the pressure transducer is located. The area will also be inspected on a daily basis for ice and any debris and cleared as required to ensure accurate monitoring of flows.
- Commencing with the start of tailings deposition, the TCA will be monitored for an additional two weeks every second day. As the dynamics of the system, i.e. rate of change in water quality, becomes better understood, the frequency of monitoring could be reduced.
- Before any discharge would commence, the TCA water would be submitted for toxicity testing and metals analysis. Only if the water meets MMER criteria will discharge from the TCA commence. The flow ratio would be calculated for each sampling event and adjusted as necessary. The discharge flow would be controlled by the automated flow control system which would use the real time flow monitoring in Doris Creek to control the discharge flow rate. Flow rates would automatically be logged by the flow control system.
- In subsequent years, it is anticipated that at the start of the open water season the analytical turnaround time will likely prevent discharge for the first few days. The downstream together with the upstream and TCA water quality monitoring results will be used to verify the performance of the discharge system at regular intervals and to make flow control adjustments as appropriate.
- As part of the control strategy, the actual water quality in the TCA will regularly be compared with the predicted water quality to assess the accuracy of the model. If necessary, the model may be recalibrated to the actual water quality observed in the TCA. The model would then be rerun to assess potential implications on the discharge strategy and to determine future operational requirements.

9.6 Operations Manuals

MHBL will prepare an Operation, Maintenance and Surveillance (OMS) Manual and an Emergency Preparedness Plan (EPP) prior to the start of operations at the TCA. These will be prepared in general accordance with the Dam Safety Guidelines published by the Canadian Dam Association (1999). The intent of the OMS Manual will be to provide mine personnel with descriptions of the equipment, tailings disposal, water management, equipment operation and monitoring procedures for the TCA.

The purpose of the EPP is to identify and evaluate potential emergencies in order to determine adequate preventive or remedial actions. The EPP will contain a notification process in case of an emergency situation and will incorporate preventive measures for situations or conditions that could be repaired or reduce the potential damage. The EPP will describe the actions to be taken in an emergency and will identify the various parties responsible for the actions and the agencies to be notified.

Copies of these plans will be issued to all parties that have responsibilities under the plan or that may be affected by the emergency situation. The EPP will be revised on annual basis to ensure that the site conditions still apply to the plan and that the contact information of the various parties is still valid.

10 TCA Maintenance

10.1 Dams

Dam maintenance will be determined each year after completion of the Dam Safety Inspection; however, it is likely that the following maintenance items would have to be completed every year:

- Some of the dam monitoring instrumentation may get damaged, either through natural wear and tear or perhaps as a result of animal damage, vandalism or accidents. Any damaged instrumentation will have to be repaired or replaced. Some of the instrumentation may also have to be recalibrated annually.
- Thermal modeling for the dams has shown that although the dam core and its foundation will remain well frozen, the upstream and downstream foundations will gradually thaw, and lead to settlement of those sections of the dam. Based on the findings of the Dam Safety Inspection, areas that have undergone settlement may have to be repaired by adding more fill.
- Snow drifts on the downstream toe of the dam will result in an insulating effect on the downstream toe, which may lead to more rapid thaw of the downstream foundation. If the snow is continuously cleared from this area, the dam may perform much better than the thermal modeling suggests, and therefore regular clearing of snow in this area is recommended.
- Every year the thermosyphons must be inspected and tested, and if necessary, the thermosyphon recharged with CO_2 gas. Any damaged radiator fins must also be repaired or replaced.

10.2 Spillway

Spillway maintenance is limited to the following:

- During the freshet, regular inspections must be conducted to ensure that there is not an ice jam in the spillway entrance. If an ice jam is detected it must be cleared using appropriate equipment.
- The spillway invert must be inspected every year and any areas where there may be some subsidence must be in-filled and re-armoured.
- Immediately after a significant flood has passed through the spillway, the spillway must be inspected and any areas where the erosion protection material has been exposed, new material must be put in place.
- The overburden cut slopes must be inspected for signs of permafrost thaw. If such signs are detected, those areas must be repaired by whatever means are appropriate.

10.3 Pipelines

Routine pipeline maintenance tasks are as follows:

- *Pipeline:* At least once every six months all pipelines must be completely flushed with fresh water. Once every year, every pipeline must be pressure tested with fresh water to determine if there are any leaks.
- *Drain outlets:* The drain outlet pipes should be monitored during pipe drainage. Should a drop off in the flow rate be detected the pipe should be flushed out. This should be carried out using hydraulic cleaning equipment.
- *Pumps:* Maintenance of the pumps, seals, controls, instrumentation and electrics is to be carried in accordance with manufacturer's specifications.
- *Valves:* Maintenance of the isolating and check valves should be performed in accordance with manufacturer's specifications.
- *Flow- and hourmeters:* All flow and hourmeters must be annually serviced and recalibrated according to manufacturer's specifications.

10.4 Emergency Dump Catch Basins

The maintenance tasks associated with the emergency dump catch basins are as follows:

- When tailings deposition stops and the emergency dump catch basins are used to dump either tailings slurry or reclaim water, the emergency dump catch basins must be emptied out as soon as possible, preferably before the spill freezes. This must be done using a trash pump.
- If the spill does freeze, a heating tent must be placed over the emergency dump catch basin, and the spill must be thawed and either pumped out or mechanically loaded onto a dump truck for disposal back to the TCA.
- Immediately after the emergency dump catch basins have been cleaned, they must be inspected to ensure that the liner integrity has not been compromised. Should any damage be observed, the appropriate repairs must be carried out as soon as practically possible.

10.5 Shoreline Erosion Protection

Maintenance tasks associated with the shoreline erosion protection works are as follows:

- Every year immediately before freeze-up the entire TCA shoreline must be inspected and areas where shoreline erosion is detected must be marked such that those areas can be clad when the ground is frozen.
- At any time during the summer months, if areas are detected along the TCA shoreline that are undergoing active shoreline erosion, silt curtains must be deployed such that any suspended matter will be contained. Physical erosion protection works will be carried out in those areas as soon possible.

- The six monitoring transects must be inspected, surveyed and routine maintenance must be carried out on the instrumentation to ensure that the instruments are in working order.
- If at any point during the life of the TCA, the suspended sediment concentration in the TCA exceeds the MMER value of 15 mg/L at the point of discharge, a silt curtain will be installed around the discharge uptake. At such time, water quality monitoring will be done both upstream and downstream of the silt curtain to ensure the success and integrity of the curtain.
- If the TSS concentration downstream of the silt curtain (if required) exceeds the MMER value before the TCA reaches FSL, or would not meet CCME Guideline values at the designated monitoring point downstream of the waterfall in Doris Creek during discharge, no water will be discharged.

11 TCA Monitoring

11.1 Dams

11.1.1 Routine Visual Inspections

Mine operations staff must carry out daily visual inspections of both dams, taking note of any signs of settlement, unaccounted for drops in water levels, signs of seepage, or any signs of damage or vandalism to instrument clusters. Records of these daily inspections must be documented in a site dairy, to be completed by the person carrying out the inspection. This inspection should also trigger actions such as snow clearing of the downstream dam abutments.

11.1.2 Annual Dam Safety Inspection

Annually, a suitably qualified Professional Engineer registered in the Nunavut Territory must undertake a personal physical inspection of the dams. This inspection must be carried out in the summer and must culminate in a detailed Dam Safety Inspection Report. The report must include findings and recommendations on the dam performance taking into account the personal inspection observations, interviews with mine operations staff responsible for the dams, as well as a review and analysis of all detailed monitoring data for the dams. This report must be delivered in a timely manner so that, if required, mitigation measures can be carried out to address any areas of concerns regarding the dams.

11.1.3 Instrumentation

Detailed monitoring instrumentation has been included in the dam design, as indicated in Dwg. T-09 and T-10. This equipment is used to monitor the thermal and deformation regime of the dams, and include the following:

- *Survey monitoring points:* 28 locations on the North Dam, along 7 transects, and 22 locations on the South Dam, along 6 transects. These monitoring points will consist of vertical settlement plates installed along the up- and downstream dam slopes to monitor the toe settlements, as well as along the dam crests to monitor core settlement. These stations must be manually read at least once a month.
- *Vertical thermistors:* 12 locations on the North Dam, along 4 transects, and 9 locations on the South Dam, along 3 transects. These thermistors will monitor the temperature of the foundation up- and downstream of the core. Manual reading of these strings will be required at least once a month. A continuous data logger recording daily values will be required along the most critical transect.
- *Horizontal thermistors:* 4 transects on the North Dam and 3 transects on the South Dam. These thermistors will monitor the temperature across the entire base of the core. Manual reading of

these strings will be required at least once a month. A continuous data logger recording daily values will be required along the most critical transect.

Other instrumentation, not directly linked to the dam, but that would provide necessary data to allow evaluation of the dam performance will include:

- *Weather station:* Detailed climatic data will be continuously recorded with a site specific weather station, which is already part of the monitoring program for the Doris North site.
- *Water level monitor:* The water level in the TCA will be continuously monitored by means of at least two submerged pressure transducers.
- *Bathymetric surveys:* Annually, a bathymetric survey will be completed to confirm the status of the deposited tailings surface.

11.2 Spillway

There will be no formal instrumentation in the spillway, since the spillway is only expected to be used in emergency events. Visual inspections of the facility will be undertaken as part of the daily operational inspections by the mine operations staff, and the facility will also be part of the annual Dam Safety Inspection.

11.3 Pipelines

Mine operations staff must carry out daily visual inspections of all pipelines, pump stations and emergency dump catch basins. The following information must be recorded in dedicated site logbooks, using a consistent format:

- *Pump stations:* Document which pumps are operational, how many hours each pump has operated and note the discharge and suction pressures of operational pumps. Carry out checks for leaks and spillages, and confirm oil levels for all pumps, and seals on water pumps for the tailings pumps. Take note of any alarms and messages.
- *Pipelines:* Record which pipelines are operational, and for how long they have been operating. Record the flowmeter data and the operating pressures along the pipelines. Check pipelines for any leaks and blockages and take note of any hazards along the pipeline route. Check the system for any alarms and messages (such as mal-function of the electric heat tracing cable inside the pipeline during freezing temperatures). Also record where actual tailings deposition has taken place in the previous 24-hour period.
- *Emergency dump catch basins:* Record the status of each emergency dump catch basin, and trigger any maintenance works such as snow clearing or emptying of a spill. Damage to the liner must also be noted and instructions to carry out repairs must be documented.

11.4 Water Quality

Detailed water quality monitoring will be required for to ensure adequate operation of the discharge strategy. Complete details of this monitoring are provided in SRK (2007c); however, for completeness these requirements are summarized as follows:

- *TCA:* The intake to the discharge pipeline will be located on a floating barge within the northern part of the TCA, about 1.5 m below the water surface. Three water samples will be obtained from the barge at depths of 1 m, 1.5 m and at 2 m to represent the intake water quality. The monitoring will initially be undertaken every second day, but may be reduced to weekly or less should the data indicate that the rate of change in water quality is small. Similarly, if the samples taken at different depths are shown to vary little, then sampling may be reduced to duplicate samples at the pipe intake depth.
- *End of pipe discharge:* The frequency of sampling and analysis is specified in the MMER to be weekly, at least initially, for regulated parameters. However, there is provision to reduce the frequency of analysis for some parameters based on the results obtained. These results will be correlated with the intake water quality results for further confirmation that the intake monitoring results reasonably reflect actual discharge water quality.
- *Doris Creek upstream of weir:* The upstream water quality samples for Doris Creek will be obtained upstream of the flow monitoring weir, as dictated by site conditions. Sampling will initially be undertaken every second day to coincide with the intake monitoring samples. As for the intake sampling, the frequency may be reduced to weekly should the data indicate that the rate of change in water quality is small.
- *Doris Creek downstream of waterfall:* Doris Creek downstream of the waterfall will be monitored only during periods of active discharge. The sample location will be established approximately 30 to 50 m downstream of the waterfall, as dictated by site conditions, to ensure that complete mixing of the TCA discharge and Doris Creek had occurred. Sampling will initially be undertaken every second day. As the discharge control strategy is refined and proven to meet discharge objectives, the frequency of sampling may be reduced.
- *Embankment seepage:* If evident, toe seepage at the North and South Dams will be sampled and monitored on a weekly basis. If flows become significant, the seepage will be collected and pumped back to the TCA.
- *Mill effluent:* Mill tailings discharge water will be monitored at a location after all of the effluent streams have been combined into a single flow. Initially the water quality will be sampled daily and composited over a two day period. Depending on the variability in the tailings effluent water quality, the composite period may be increased and the frequency of analysis reduced.
- *Lake Ice Thickness:* At least once a month, while there is ice on Tail Lake the ice thickness must be recorded through physical coring, at a minimum of five locations, spanning the surface area of the lake.

11.5 Shoreline Erosion Protection

Mine operations staff must carry out daily visual inspections along the entire TCA shoreline taking note of, and recording any signs of shoreline erosion. In addition, the following permanent instrumentation (as illustrated in Dwg. T-14) will be installed around the TCA perimeter (note that some of this instrumentation has already been put in place):

- *Thermistors:* 6 vertical thermistors have been installed along 6 important transects of the TCA perimeter, between elevations 28.3 m and 33.5 m. These thermistors will monitor the temperature profile in the overburden soils as the water level rises, giving advance warning of thaw. This data must be collected manually at least once a month.
- *Survey transects:* 6 detailed strip surveys along the same six transects where the thermistors are installed will be monitored such that the extent of shoreline erosion can be determined. Each strip will be 50 m wide. Strip surveys should be redone annually, including a bathymetry survey of the underwater section of the strip down to the original TCA water level of 28.3 m (this should continue for five years after the FSL is reached or till there is no discernable profile difference in any two consecutive years). This will provide information of the slope morphology as time progresses, as well as allow for calculation of a sediment balance.

The annual Dam Safety Inspection Report must include a detailed section that discusses and addresses the status of any shoreline erosion processes, and must recommend if necessary any remedial action that must be carried out.

12 TCA Closure Methodology

12.1 General

The TCA closure plan is described as part of the Doris North Project Abandonment and Closure Plan (MHBL 2006). A summary of some of the underlying closure principles as it relates to the closure of the TCA is provided here. The main closure components relating to the TCA include:

- Continued active water management until such time as the water quality in the TCA returns to acceptable discharge standards;
- Establish a suitable water cover over the tailings; and
- Breach the North Dam to allow the TCA catchment to revert back to its pre-deposition hydrologic cycle.
- The South Dam will remain in place.

12.2 Water Management

Active water management of the TCA will continue, either through active discharge via the discharge pump system, or through natural discharge via the spillway. Once the water quality in the TCA has returned to background water quality, containment of the TCA is no longer required, and the North Dam can be breached.

12.3 Water Cover

The final TCA closure require a permanent water cover of at least 3 m above the highest tailings elevation in the impoundment. Research has shown that a minimum stagnant water cover of 0.3 m is sufficient to prevent oxidization of tailings. Tailings can however be re-suspended due to wave action induced by environmental factors, and therefore the rule of thumb is to design a water cover of at least 1 m thick. Based on the orientation of the TCA, the predominant wind direction, maximum wind speeds, and the particle size of the tailings, the actual minimum water cover depth for the TCA has been calculated to be at least 2 m thick. A 3 m thick water cover was subsequently selected as the design criteria, which constitutes a factor of safety of 1.5, which seems reasonable to account for uncertainty. Complete details of the minimum water cover thickness design calculations are presented in Appendix C.

The maximum tailings surface in the TCA is expected to be below 24.3 m, which implies that the minimum final water elevation in the TCA must be at 27.3 m to ensure compliance with the design criteria. In actual fact, since the water level in the TCA will return to its pre-deposition elevation of 28.3 m once the North Dam is breached, the water cover will be at least 4 m thick offering an overall factor of safety of 2.

12.4 North Dam Breach

Prior to breaching the North Dam, any water in the TCA above elevation 28.3 m will be pumped via the discharge system into Doris Creek. Discharge of the excess water will be done using the same basic criteria as that used during the active discharge phase. Depending on the volume of water that has to be discharged, this draw-down period could be more than one discharge season.

Once the water level in the TCA is at 28.3 m, the North Dam will be breached by cutting a slot through the North Dam, down to the original pre-construction elevation. The slot will measure about 20 m wide, with 4H:1V side slopes on either side. The cut slopes will be covered with a 2.5 m thick layer of run of quarry material to ensure physical and thermal stability.

Tail Lake outflow will be re-established along the base of the slot cut, and suitable bedding material will be put in place to ensure erosional stability of the channel.

This report, "**Design of the Tailings Containment Area, Doris North Project, Hope Bay, Nunavut, Canada**", was prepared by SRK Consulting (Canada) Inc.

REGISTERED PROFESSIONAL ENGINEER
4/11/07
E.M. RYKAART
LICENSEE
NWT/NU

Prepared by

Maritz Rykaart, Ph.D., P.Eng.
Principal Engineer

13 References

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SRK Consulting (Canada) Inc. 2006c. *Geochemical Characterization of Quarry Rock, Nunavut, Canada*. Report submitted to Miramar Hope Bay Ltd., October 2006.

Appendix A

SRK Technical Memorandum re: Doris North Project Tailings Properties

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Engineers and Scientists

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Technical Memorandum

To:	Brian Labadie	**Date:**	July 15, 2005
cc:	Project File	**From:**	Maritz Rykaart
Subject:	Doris North Project Tailings Properties	**Project #:**	1CM014.006

The tailings physical characteristics are documented in the following two reports;

SRK Consulting (Canada) Inc. (2003). *Tailings Impoundment Preliminary Design, Doris North Project, Nunavut, Canada – Volume I, Report*. Technical Report prepared for Miramar Hope Bay Limited. Project No. 1CM014.01, October 2003.

SRK Consulting (Canada) Inc. (2003). *Tailings Impoundment Preliminary Design, Doris North Project, Nunavut, Canada – Volume II, Appendixes*. Technical Report prepared for Miramar Hope Bay Limited. Project No. 1CM014.01, October 2003.

This technical memorandum contains an extract of the relevant sections of the above two reports relating to tailings properties. Please note that we have retained the original report numbering of the source report.

6.3 Tailings Properties

A sample of total combined mill tailings from a pilot metallurgical tests conducted by Bateman Engineering was sent to AMEC Earth Engineering Pty Limited in Perth Australia (AMEC 2003). Representative samples of the tailings were extracted for determination of the tests listed in Table 6.1. The Australian Standards listed in Table 6.1 all have similar or equivalent ASTM procedures. The complete laboratory data sheets for all these tests are presented in Appendixes 6A through 6F.

Table 6.1: Laboratory Tests Conducted on Final Combined Mill Tailings

Test	Test Method (Australian Standards)	Number of Tests
Grain Size Distribution	Sieve and Hydrometer (AS 1289.3.6.2)	1
Plastic Properties	Casagrande Method (AS 1289.3.1.1,.3.2.1,.3.3.1,.3.4.1,.2.1.1)	1
Particle Density	AS 1289.3.5.1	1
Triaxial Test	Consolidated Undrained Triaxial Test with Pore Pressure Measurement (AS 1289.6.4.2)	1
Consolidation Test	One-Dimensional Consolidation (AS 1289.6.6.1)	1
Undrained Settling Test	SRC-WI-4.8.3	1
Drained Settling Test	SRC-WI-4.8.2	1

6.3.1 Index Properties

The total tailings are composed of sandy fine to coarse silt with 56% passing the No. 200 sieve (75 micron). The percent by weight of clay sized particles (less than 2 microns) in the tailings sample was approximately 11%. The tailings were found to be non-plastic and the measured tailings particle density was 2.74 g/cm^3.

6.3.2 Deposited Tailings Densities

The deposited tailings density is dependent on both the specific gravity of the particles and the void ratio of the resulting deposited material. Representative tailings densities deposited by different methods are given in Table 6.2. Void ratios for each method are conservatively estimated from SRK's experience at other mines. The measured dry density for the Doris North Project is summarized in Table 6.3.

Table 6.2: Typical In-Place Dry Densities for Tailings

Deposition Method	Void Ratio e	Dry Density (tonnes/m^3)
Tailings hydraulically deposited above water	1.0	1.50
Tailings hydraulically deposited underwater	1.2	1.36

Table 6.3: Measured Void Ratio & Dry-Density for the Doris North Project Tailings

Data Source	Void Ratio e	Dry Density (tonnes/m^3)
Triaxial Test	0.839	1.49
Consolidation Test - Initial	0.839	1.49
Consolidation Test – Final	0.797	1.54
Undrained Settling Test @ 33.8% solids	-	1.19
Drained Settling Test @ 33.8% solids	-	1.43

The most realistic design value to use for the subaqueous tailings deposition in Tail Lake is 1.19 tonnes/m^3, based on the measured properties. For the preliminary design presented in this report, we have assumed a tailings solids specific gravity of 2.7 and an in-place void ratio of 1.2, which results in an in-situ dry density of 1.23 tonnes/m^3.

6.3.3 Permeability and Strength Parameters

The permeability of the total tailings was measured in the laboratory by using one triaxial and one consolidation test. The results of these tests are summarized in Tables 6.4 and 6.5 respectively.

Table 6.4: Summary of Triaxial Compression Test Data

Stage - Confining Stress, σ_3 (kPa)	Coefficient of Consolidation, C_v (m^2/year)	Coeff. of Volume Compressibility, M_v, (m^2/kN)	Hydraulic Conductivity, K (cm/sec)	Cohesion, c (kPa)
1 (250 kPa)	113,890	0.153	5.4×10^{-5}	0.825
2 (300 kPa)	3,388	0.108	1.1×10^{-5}	0.010
3 (400 kPa)	1,822	0.040	2.2×10^{-6}	0.007

Table 6.5: Summary of One-Dimensional Consolidation Properties of Tailings

Pressure (kPa)	Void Ratio e	Coefficient of Consolidation, C_v (m^2/year)	Coeff. of Volume Compressibility, M_v, (m^2/kN)	Hydraulic Conductivity, K (cm/sec)
0	0.839	-	-	-
20	0.836	0.648	8.170×10^{-5}	1.69×10^{-9}
40	0.830	0.529	8.183×10^{-5}	1.37×10^{-9}
100	0.833	0.488	2.732×10^{-5}	0.42×10^{-9}
200	0.824	0.455	3.289×10^{-5}	0.47×10^{-9}
300	0.811	0.439	7.178×10^{-5}	9.98×10^{-10}
400	0.797	0.441	7.791×10^{-5}	1.11×10^{-10}
200	0.798	-	-	-
40	0.803	-	-	-

Following the measurement of the hydraulic conductivity at 400 kPa confining stress, the tailings sample in the triaxial cell was axially loaded to failure under undrained conditions to measure the frictional strength. The results of this test are summarized in Table 6.6. A value of 43.2° was obtained for the angle of internal friction, which is considered high for tailings. This value may have to be confirmed with further testing if it is required for the final design.

Table 6.6: Summary of Shear Strength Properties for Tailings

Parameter	Stage 1	Stage 2	Stage 3
Confining Stress, σ_3 (kPa)	250	300	400
Porewater Pressure, U (kPa)	222	137	134
Effective Confining Stress, (σ_3 – U) (kPa)	28	163	266
Deviator Stress, (σ_1 - σ_3) (kPa)	128	706	1,163
Shear Stress (σ_1 - σ_3)/2 (kPa)	64	353	582
Internal Friction, Φ (degrees)	43.2		
Cohesion, c (kPa)	1		

6.3.4 Tailings Settling Properties

The tailings settling tests results are summarized in Table 6.7. These results confirm that due to the coarse nature of the tailings they settle out quickly and the recovery of clarified water should, therefore, be relatively simple. The bench scale tests suggest that under undrained conditions, similar to subaqueous deposition in Tail Lake, the maximum settling time is in the order of 2 hours.

Table 6.7: Tailings Settlement Time

Test	Supernatant Suspension (%)	Dry Density (tonnes/m^3)	Elapsed Time (minutes)
Undrained	74.44	1.112 (1.190 maximum)	120 (2,880 minutes to reach maximum dry density)
Drained	84.16	1.412 (1.430 maximum)	75 (150 minutes to reach maximum dry density)

The Appendixes referred to in the text above are appended.

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TEST CERTIFICATE

Client: MIRAMAR HOPE BAY LIMITED
Project: DORIS NORTH COMBINED FINAL MILL TAILING

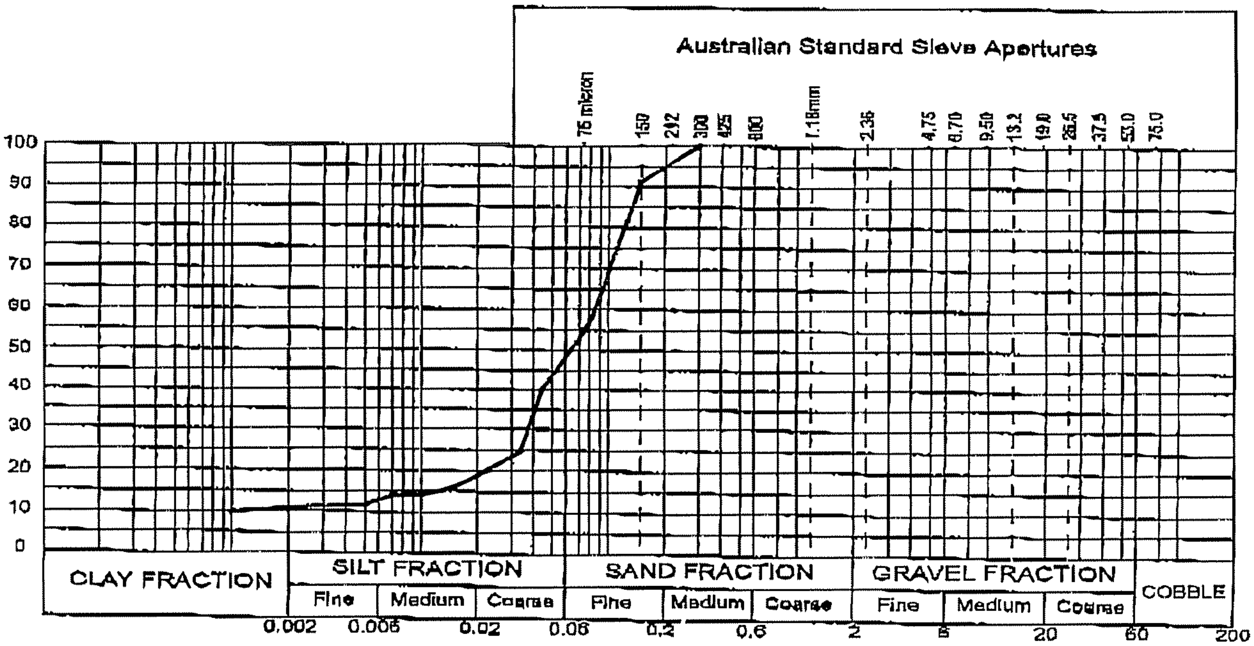
Sheet No.: 2 OF 10
Job No.: S9645
Date Tested: 25.08.03

Sample ID: TAILS

Particle Size Distribution of a Soil

AS 1289.3.6.2: Sieving with Hydrometer

Sieving				Hydrometer			
Sieve Size	% Passing	Sieve Size	% Passing	Diameter	% Passing	Diameter	% Passing
75.0mm		1.18 mm		67 micron	49	10 micron	14
37.5 mm		600 micron		49 micron	40	7 micron	14
19.0 mm		425 micron		35 micron	34	5 micron	12
9.50 mm		300 micron	100	26 micron	25	1 micron	10
4.75 mm		150 micron	92	18 micron	21		
2.36mm		75 micron	56	13 micron	18		



Remarks: Sampling Method/s - Submitted by Client.

This laboratory is accredited by the National Association of Testing Authorities, Australia. The test(s) reported herein have been performed in accordance with its terms of accreditation. This document shall not be reproduced except in full.

Approved:

W Rozmianiec

Date: 17.09.03

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TEST CERTIFICATE

Client: MIRAMAR HOPE BAY LIMITED
Project: DORIS NORTH COMBINED FINAL MILL TAILING

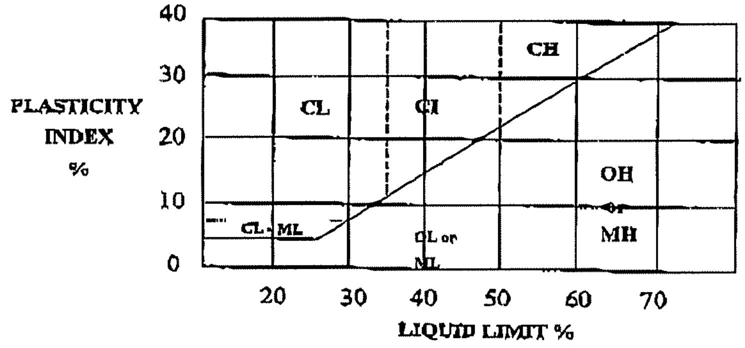
Sheet No.: 3 OF 10
Job No.: S9645
Date Tested: 11.09.03

Plastic Properties - Casagrande Method

AS 1289.3.1.1, .3.2.1, .3.3.1, .3.4.1, .2.1.1

Test No.		1	2	3	4
Sample ID		TAILS			
Liquid Limit	%	Not Obtainable			
Plastic Limit	%	Non Plastic			
Plasticity Index	%	Non Plastic			
Linear Shrinkage	%	0.5			

PLASTICITY CHART: AS 1726



History of Sample: Cool Oven Dried
Method of Preparation: Dry Sieved
Remarks: Sampling Method/s - Submitted by client.

Length of Linear Shrinkage Mould: 250 mm
Nature of Shrinkage: Normal

NATA

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Approved:

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Date: 17.09.03

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TEST CERTIFICATE

Client: MIRAMAR HOPE BAY LIMITED
Project: DORIS NORTH COMBINED FINAL MILL TAILING

Sheet No.: 4 OF 10
Job No.: S9645
Date Tested: 28.08.03

Determination of the Soil Particle Density of a Soil

AS 1289.3.5.1

Sample ID	Temperature of Test	Average Soil Particle Density (-2.36mm)	Average Soil Particle Density (+2.36mm)	Soil Particle Density Total Soil Sample
	°C	g/cm³	g/cm³	g/cm³
TAILS	20.5	2.74	N/A	2.74

Remarks: Sampling Method/s - Submited by client

NATA

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Approved:

W Rozmianiec

Date: 17.09.03

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TEST CERTIFICATE

Client: MIRAMAR HOPE BAY LIMITED
Project: DORIS NORTH COMBINED FINAL MILL TAILING

Sheet No.: 2 OF 11
Job No.: S 9645
Testing period: 07.09.03 16.09.03

Sample ID: TAILS

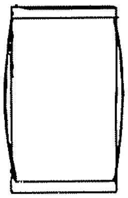
Triaxial Compression Test

AS 1289.6.4.2

Triaxial Test Conditions: Consolidated Undrained Multistage with Pore Pressure Measurement

Specimen Data		Initial
Diameter	mm	50.0
Length	mm	99.9
Volume	mm^3	196153.2
Moisture Content	%	22.5
Dry Density	t/m^3	1.49
Soil Particle Density	t/m^3	2.74
Volume of Solids	m^3	1.067E-04
Volume of Voids	m^3	8.949E-05
Void Ratio		0.839
Volume of Water	m^3	6.566E-05
Saturation	%	73

MODE OF FAILURE



Stage	Cv (m^2/ year)	Mvl (m^2/MN)	k (m/sec)	c	Drainage
1	11389	0.153	5.4E-07	0.825	One End only
2	3388	0.108	1.1E-07	0.010	One End only
3	1822	0.040	2.2E-08	0.007	One End only

Remarks: A 200kPa back pressure was applied during the saturation/consolidation phases

Authorised: R . Deznan

Date: 16.09.03

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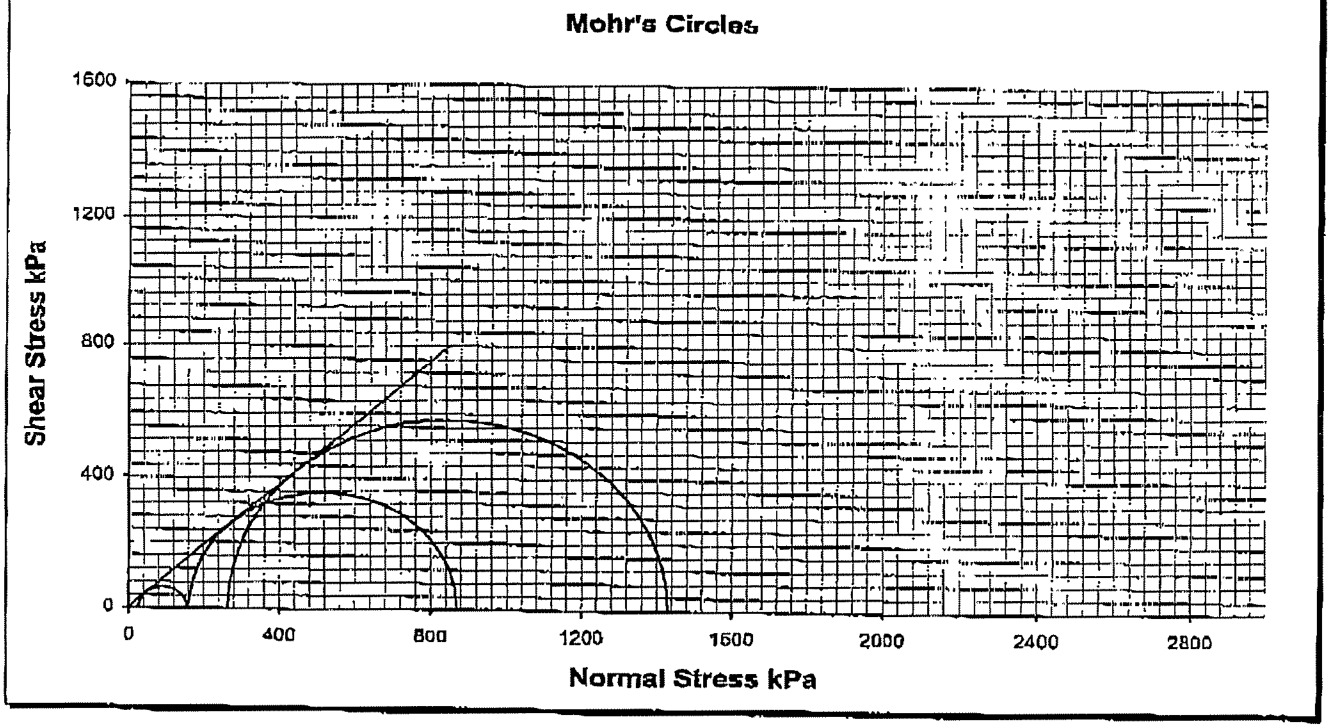
TEST CERTIFICATE

Client: MIRAMAR HOPE BAY LIMITTED
Project: DORIS NORTH COMBINED FINAL MILL TAILING
Sheet No.: 3 OF 11
Job No.: S 9645
Testing Period: 07.09.03 16.09.03

Sample ID: TAILS

Triaxial Compression Test

Confining Stress (σ_3)	kPa	250	300	400
Porewater Pressure (U)	kPa	222	137	134
Effective Confining Stress (σ_3-U)	kPa	28	163	266
Deviator Stress ($\sigma_1-\sigma_3$)	kPa	128	706	1169
Shear Stress ($\sigma_1-\sigma_3$) /2	kPa	64	353	582
Internal Friction (Φ)	Degrees	43.2		
Cohesion (c)	kPa	1		



Remarks: A 200kPa back pressure was applied during the saturation/consolidation phases

Authorised By: R. Deznan

Date : 16.09.03

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TEST CERTIFICATE

Client: MIRAMAR HOPE BAY LIMITTED
Project: DORIS NORTH COMBINED FINAL MILL TAILING
Sample ID: TAILS

Sheet No.: 4 OF 11
Job No.: S 9645
Date Tested: 07.09.03
16.09.03

One Dimensional Consolidation Properties of a Soil

AS 1289.6.6.1

Data:		Initial	Final	Pressure (kPa)	T_{50} minutes
Sample Preparation:	Insitu				
Initial Dry Density:	t/m^3	1.49	1.54	20	13.667
Initial Moisture Content:	%	22.6	13.4	40	16.667
Soil Particle Density (AS 1289.3.5.1):	t/m^3	2.74	2.74	100	18.000
Initial % Saturation:	%	73.8	47.3	200	19.167
Test Condition:	Inundated			300	19.667
Cycle :	24 Hours			400	19.167

PRESSURE kPa	VOID RATIO	COEFFICIENT OF CONSOLIDATION Cv (m^2/year)	COEFFICIENT OF VOLUME COMPRESSIBILITY Mv (m^2/kN)	COEFFICIENT OF PERMEABILITY k (m/sec)
0	0.839	-	-	-
20	0.836	0.648	8.170×10^{-5}	1.69×10^{-11}
40	0.833	0.529	8.183×10^{-5}	1.37×10^{-11}
100	0.830	0.488	2.732×10^{-4}	0.42×10^{-11}
200	0.824	0.455	3.289×10^{-6}	0.47×10^{-11}
300	0.811	0.439	7.178×10^{-5}	9.98×10^{-12}
400	0.797	0.441	7.791×10^{-5}	1.11×10^{-12}
200	0.798	-	-	-
40	0.803	-	-	-

Remarks: Sampling Method/s - Submitted by client. Soil Particle Density value was assume.

Approved: R Deznan

Date: 16.09.03

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TEST CERTIFICATE

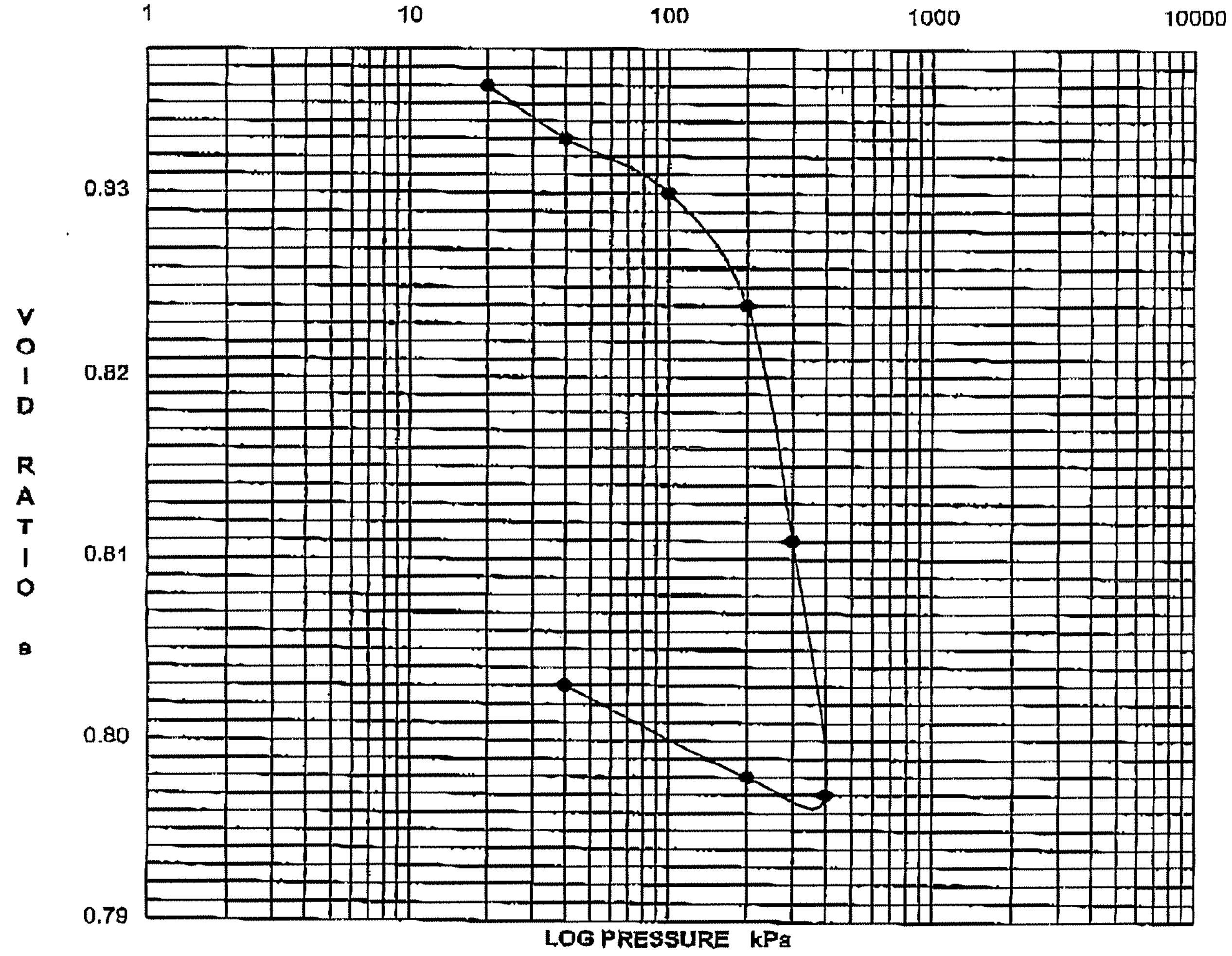
Client: MIRAMAR HOPE BAY LIMITTED
Project: DORIS NORTH COMBINED FINAL MILL TAILING

Sheet No.: 5 OF 11
Job No.: S 9645
Date Tested: 07.09.03
16.09.03

Sample ID: TAILS

One Dimensional Consolidation Properties of a Soil

AS 1289.6.6.1



Remarks: Sampling Method/s- Submitted by client.

Approved: R. Dezman

Date: 16.09.03

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TEST CERTIFICATE

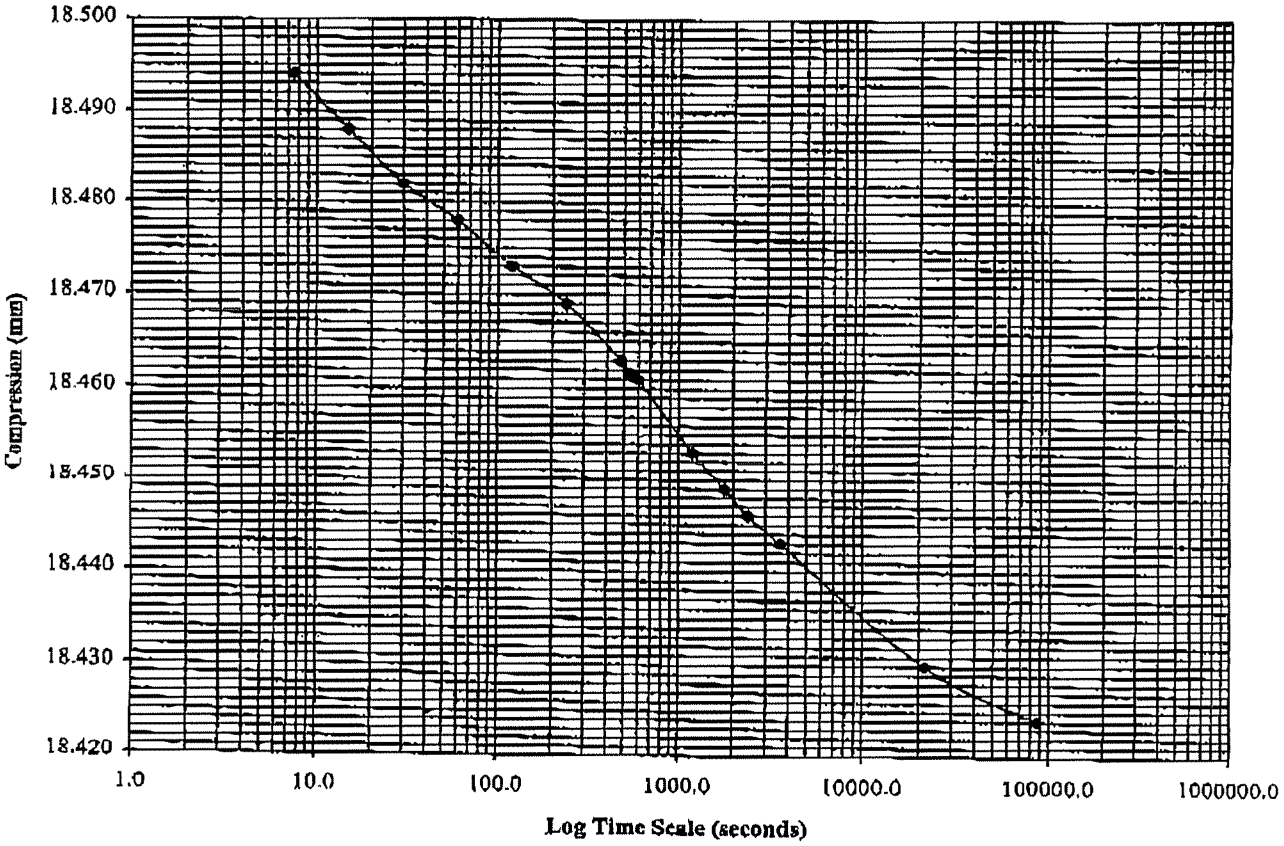
Client: MIRAMAR HOPE BAY LIMITTED
Project: DORIS NORTH COMBINED FINAL MILL TAILING
Sample ID: TAILS
Sheet No.: 6 OF 11
Job No.: S 9645
Date Tested: 07.09.03
19.09.03

CONSOLIDATION TEST

AS 1289.6.6.1

LOAD:	20	kPa	0 % Consolidation :	18.485 mm
INITIAL HEIGHT:	18.500	mm	50 % Consolidation :	18.455 mm
FINAL HEIGHT:	18.424	mm	100 % Consolidation :	18.424 mm
Cα:		m²/yr	t 50 :	16.667 min

Compression vs Time (Log Scale)



Remarks: Sampling Method/s - Submitted by client

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TEST CERTIFICATE

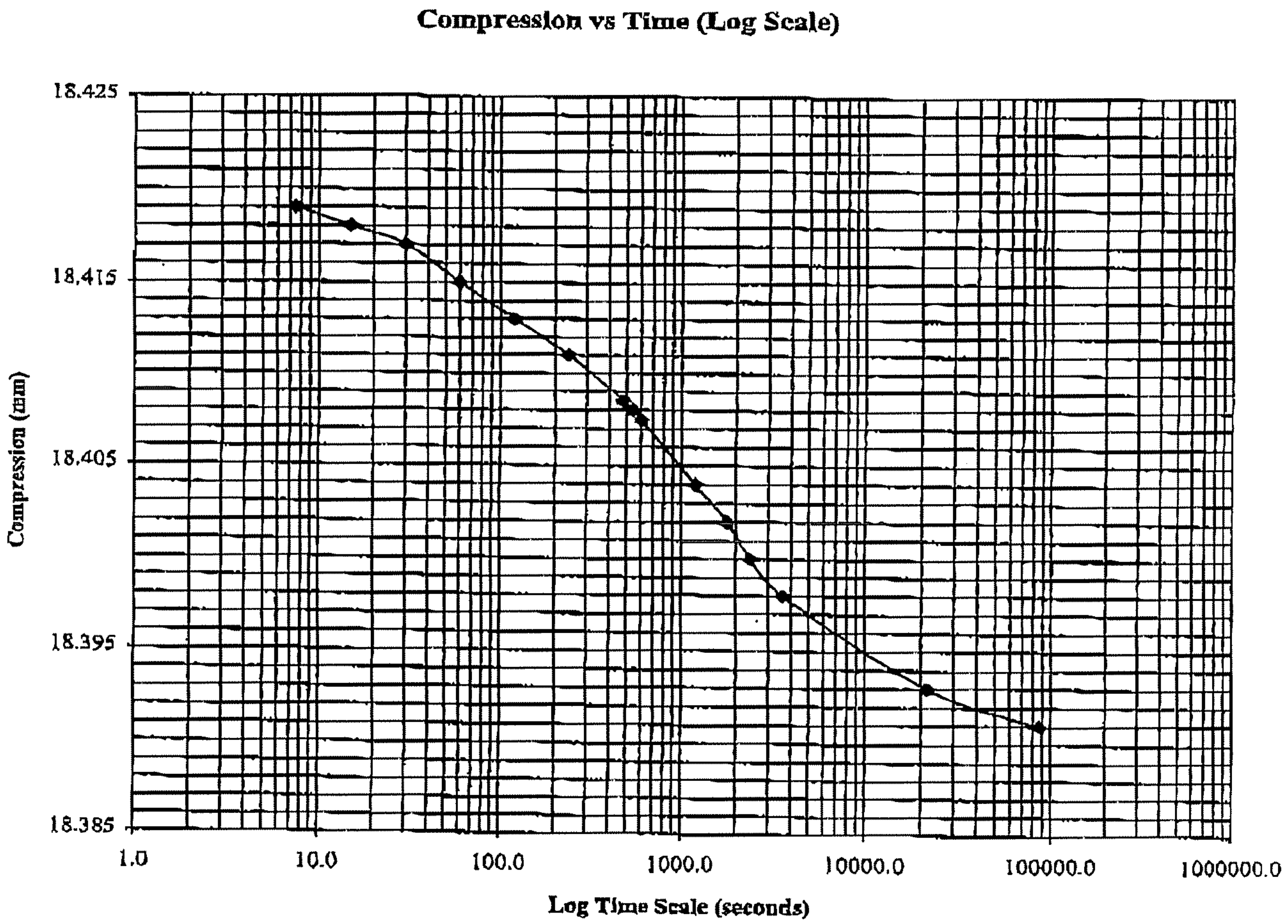
Client: MIRAMAR HOPE BAY LIMITED
Project: DORIS NORTH COMBINED FINAL MILL TAILING
Sample ID: TAILS

Sheet No.: 7 OF 11
Job No.: S 9645
Date Tested: 07.09.03
19.09.03

CONSOLIDATION TEST

AS 1289.6.6.1

LOAD:	40	kPa	0 % Consolidation :	18.421 mm
INITIAL HEIGHT:	18.424	mm	50 % Consolidation :	18.406 mm
FINAL HEIGHT:	18.391	mm	100 % Consolidation :	18.391 mm
Cα:		m²/yr	t 50 :	13.667 min



Remarks: Sampling Method/s - Submitted by client

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TEST CERTIFICATE

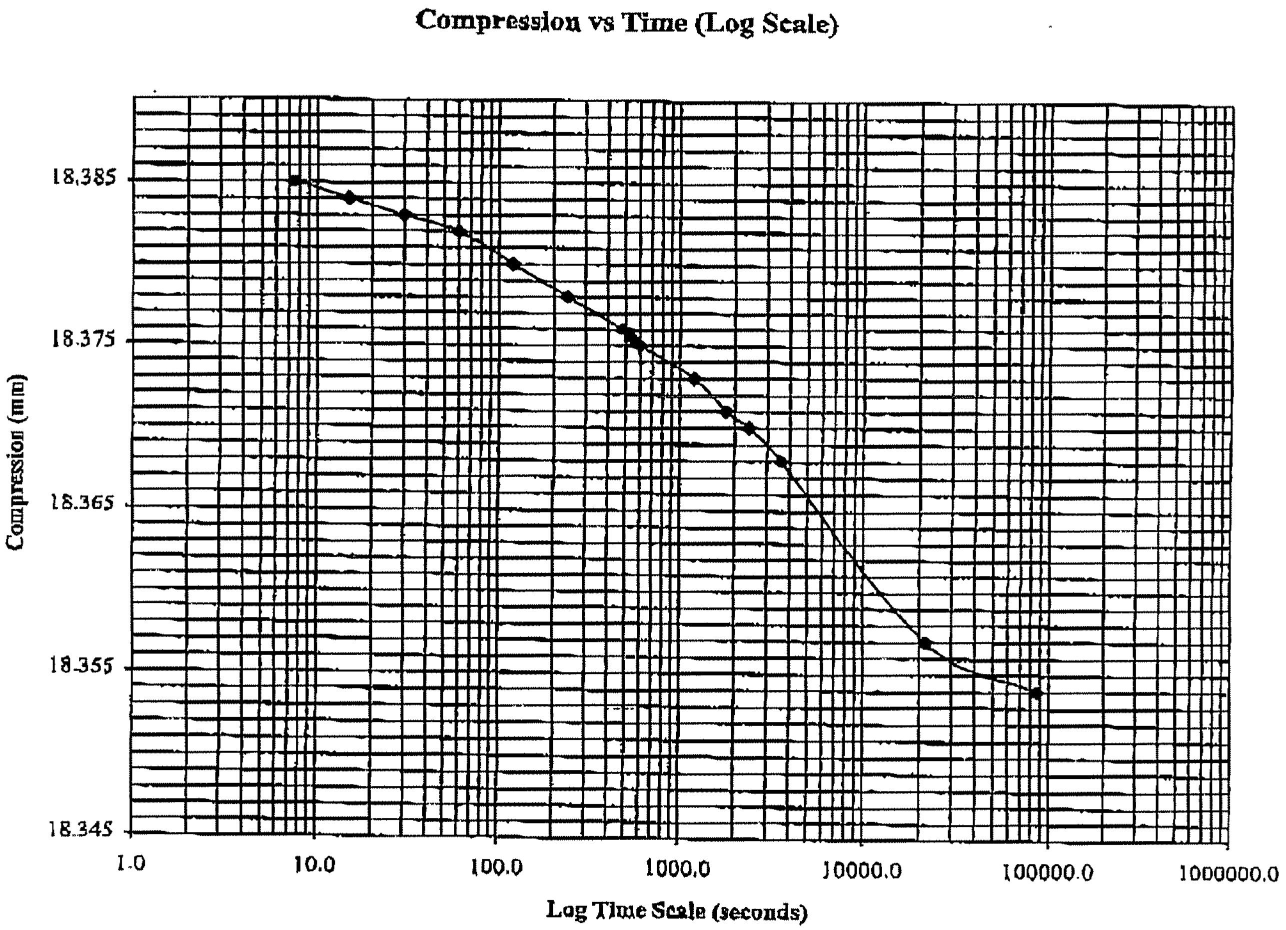
Client: MIRAMAR HOPE BAY LIMITED
Project: DORIS NORTH COMBINED FINAL MILL TAILING
Sample ID: TAILS

Sheet No.: 8 OF 11
Job No.: S 9645
Date Tested: 07.09.03
19.09.03

CONSOLIDATION TEST

AS 1289.6.6.1

LOAD:	100	kPa	0 % Consolidation :	18.387 mm
INITIAL HEIGHT:	18.391	mm	50 % Consolidation :	18.371 mm
FINAL HEIGHT:	18.354	mm	100 % Consolidation :	18.354 mm
Cα:		m^2/yr	t 50 :	18.00 min



Remarks: Sampling Method/s - Submitted by client

Approved: R. Dz
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SRC RF-037

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TEST CERTIFICATE

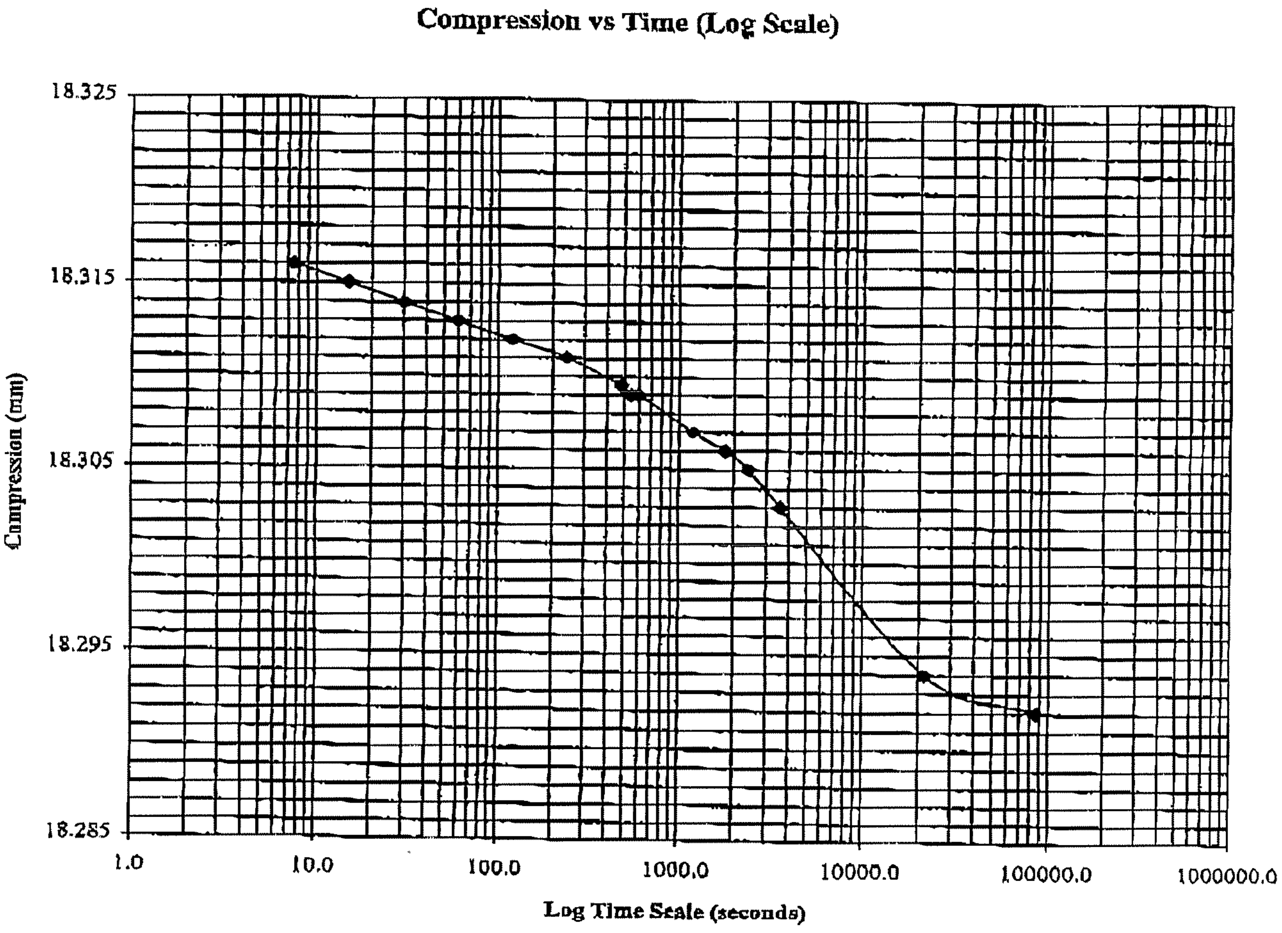
Client: MIRAMAR HOPE BAY LIMITTED
Project: DORIS NORTH COMBINED FINAL MILL TAILING
Sample ID: TAILS

Sheet No.: 9 OF 11
Job No.: S 9645
Date Tested: 07.09.03
19.09.03

CONSOLIDATION TEST

AS 1289.6.6.1

LOAD:	200	kPa	0 %	Consolidation :	18.318 mm
INITIAL HEIGHT:	18.354	mm	50 %	Consolidation :	18.305 mm
FINAL HEIGHT:	18.292	mm	100 %	Consolidation :	18.292 mm
Cα:		m²/yr	t 50 :	19.167 min	



Remarks: Sampling Method/s - Submitted by client

Approved: R. Dz
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TEST CERTIFICATE

Client: MIRAMAR HOPE BAY LIMITTED
Project: DORIS NORTH COMBINED FINAL MILL TAILING
Sample ID: TAILS

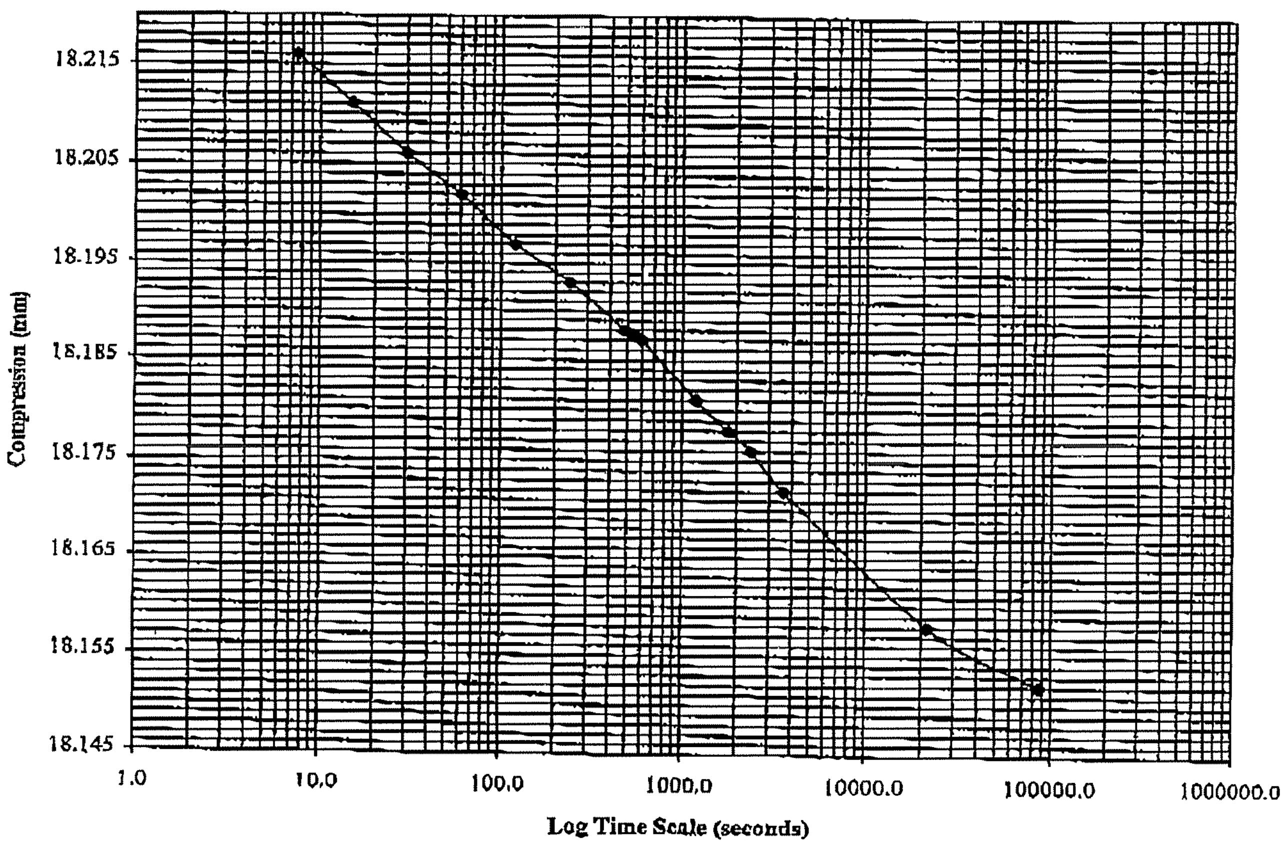
Sheet No.: 10 OF 11
Job No.: S 9645
Date Tested: 07.09.03
19.09.03

CONSOLIDATION TEST

AS 1289.6.6.1

LOAD:	300	kPa	0 %	Consolidation :	18.198 mm
INITIAL HEIGHT:	18.292	mm	50 %	Consolidation :	18.175 mm
FINAL HEIGHT:	18.152	mm	100 %	Consolidation :	18.152 mm
Cα:		m²/yr	t 50 :	19.667 min	

Compression vs Time (Log Scale)



Remarks: Sampling Method/s - Submitted by client

Approved: R. Dz
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Date: 16.09.03

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TEST CERTIFICATE

Client: MIRAMAR HOPE BAY LIMITTED
Project: DORIS NORTH COMBINED FINAL MILL TAILING
Sample ID: TAILS

Sheet No.: 11 OF 11
Job No.: S 9645
Date Tested: 07.09.03
19.09.03

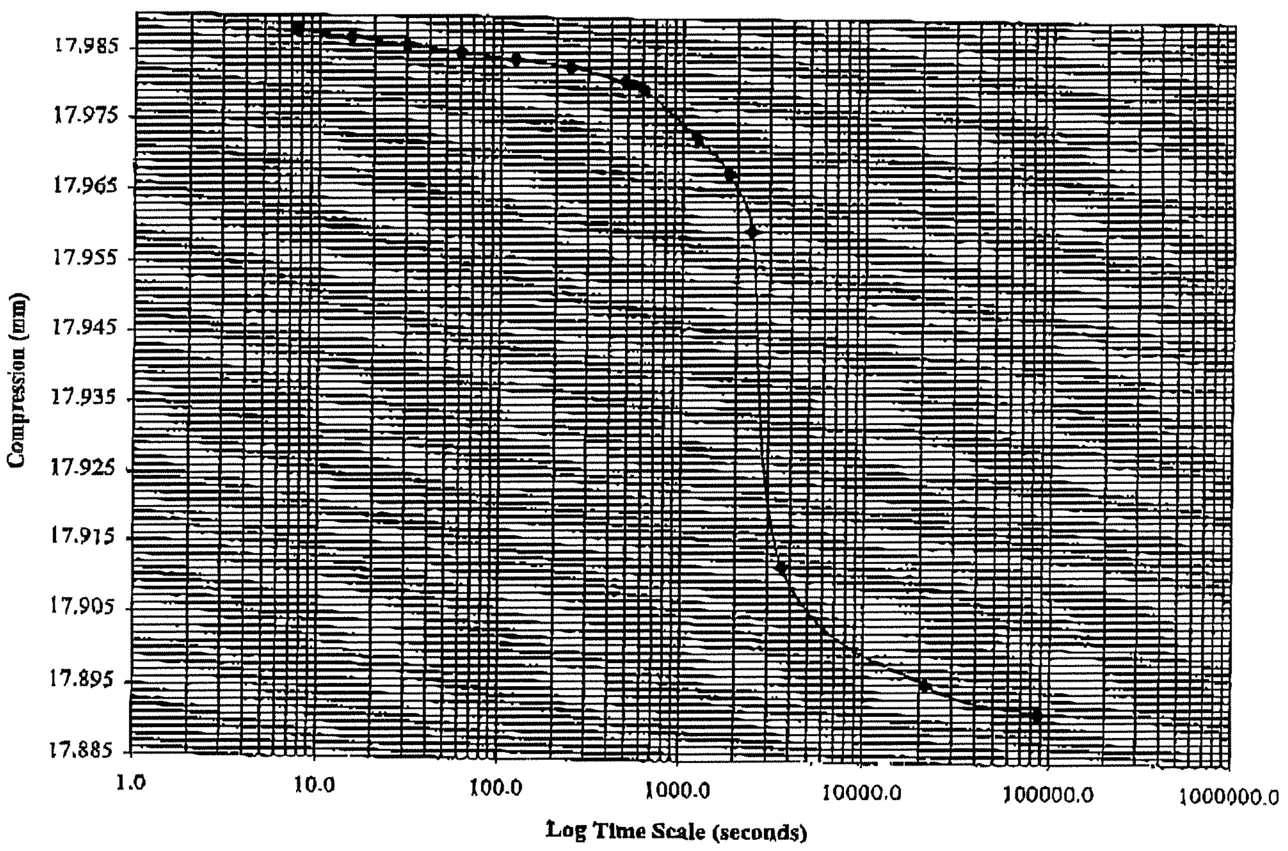
CONSOLIDATION TEST

AS 1289.6.6.1

LOAD: 400 kPa
INITIAL HEIGHT: 18.152 mm
FINAL HEIGHT: 17.892 mm
Cα: m²/yr

0 % Consolidation : 18.012 mm
50 % Consolidation : 17.952 mm
100 % Consolidation : 17.892 mm
t 50 : 19.167 min

Compression vs Time (Log Scale)



Remarks: Sampling Method/s - Submitted by client

Approved: R. Dekman

Date: 16.09.03

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TEST CERTIFICATE

Client: MIRAMAR HOPE BAY LIMITED
Project: DORIS NORTH COMBINED FINAL MILL TAILINGS

Sheet No.: 5 OF 10
Job No.: S9645
Date Tested: 25.08.03-28.08.03

Sample ID: TAILS

Undrained Settling Test
SRC-WI-4.8.3

% SOLIDS:	**33.8**	
TEST CYLINDER		
Diameter of Cylinder	60.9	mm
Area of Cylinder	2913.3	mm²
Mass of Cylinder	833.42	g
Mass Cylinder & Tailings	1714.67	g
Mass of Tailings Wet	881.25	g
Mass of Tailings Dry	298.10	g

MOISTURE CONTENT CHECK

Container No.	6	
Mass Cont. & Tailings Wet	257.48	g
Mass Cont. & Tailings Dry	153.29	g
Mass Container	100.03	g
Moisture Content	**195.63**	%

Amount of Water in Sample **200.17** mm

Date & Time Test Commenced: 25.08.03 @ 0930HRS

					With Respect to Initial Volume of Water				
Elapsed Time min	Height of Water mm			Height of Tailings mm	Cumulative Supernatan %				Dry Density t/m3
0	0			239	0.00				0.428
15	37			202	18.48				0.507
30	75			166	37.47				0.616
45	106			133	52.95				0.769
60	129			111	64.44				0.922
75	142			96	70.94				1.066
90	146			94	72.94				1.089
120	149			92	74.44				1.112
150	149			89	74.44				1.150
180	149			89	74.44				1.150
240	150			88	74.94				1.163
1440	151			87	75.44				1.176
2880	152			86	75.93				1.190
4320	152			86	75.93				1.190

Remarks: Sampling method/s - Submitted by client

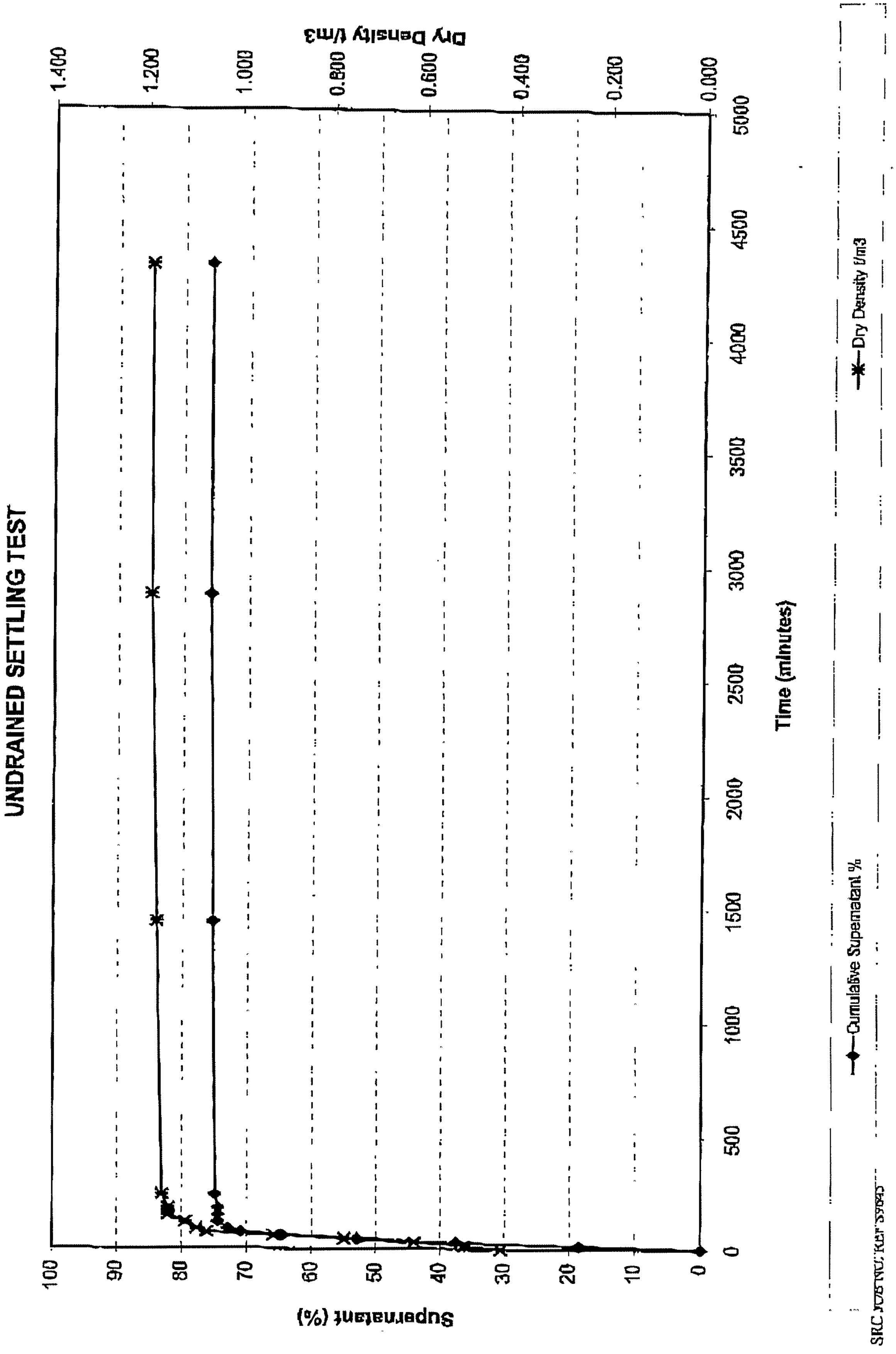
Approved:

W Rozmianiec

Date: 17.09.03

SHEET NO.:6 OF 10

DORIS NORTH COMBINED FINAL MILL TAILINGS

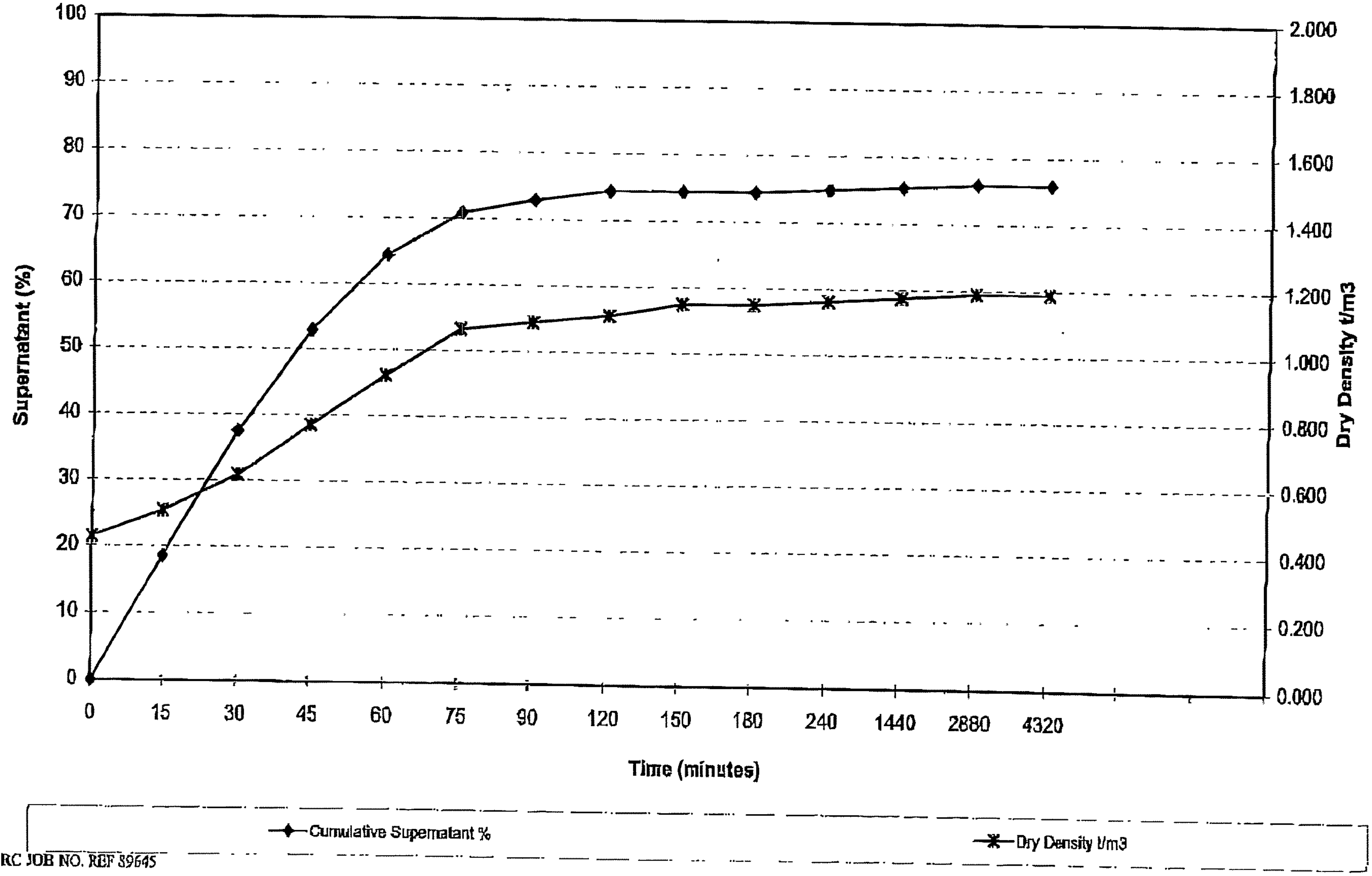


SRC JOB NO./REF S9045

DORIS NORTH COMBINED FINAL MILL TAILINGS

SHEET NO.:7 OF 10

UNDRAINED SETTLING TEST



SRC JOB NO. REF 89645

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TEST CERTIFICATE

Client: MIRAMAR HOPE BAY LIMITED
Project: DORIS NORTH COMBINED FINAL MILL TAILINGS
Sheet No.: 8 OF 10
Job No.: S9645
Date Tested: 25.08.03-28.08.03

Sample ID: TAILS

Drained Settling Test
SRC-WI-4.8.2

% SOLIDS:	**33.8**	
TEST CYLINDER		
Diameter of Cylinder	57.3	mm
Area of Cylinder	2579.0	mm^2
Mass of Cylinder	1064	g
Mass Cylinder & Tailings	1914.30	g
Mass of Tailings Wet	850.30	g
Mass of Tailings Dry	287.63	g
AFTER TEST		
Mass Cylinder & Tailings	1433.42	g
Mass of Tailings Wet	369.42	g
Mass of Tailings Dry	287.63	g
Amount of Water in Sample	**218.17**	mm
Amount of Water Drained	**88.79**	mm
Amount of Water Removed	**0.00**	mm
Remaining Water in Sample	**129.38**	mm

MOISTURE CONTENT CHECK

Container No.	6	
Mass Cont. & Tailings Wet	257.48	g
Mass Cont. & Tailings Dry	153.29	g
Mass Container	100.03	g
Moisture Content	**195.63**	%

AFTER TEST

Final Moisture Content	**28.44**	%

Date & Time Test Commenced: 25.08.03 @ 1000HRS

check: **120.51** 97.66

					With Respect to Initial Volume of Water					Moisture
Elapsed Time min	Height of Water mm	Water Drained ml	Water Drained mm	Height of Tailings mm	Supernatan %	Drainage %	Cumulative Underdrain %	Total Recovery %	Dry Density t/m3	Content of Slurry %
0	0	0	0	251	0.00	0.00	0.00	0.00	0.444	195.63
15	39	100	39	187	17.88	17.77	17.77	35.65	0.596	125.89
30	75	20	8	149	34.38	3.55	21.33	55.70	0.748	86.66
45	105	10	4	108	48.13	1.78	23.10	71.23	1.033	56.28
60	123	8	3	86	56.38	1.42	24.53	80.90	1.297	37.36
75	127	8	3	79	58.21	1.42	25.95	84.16	1.412	30.99
90	124	6	2	79	56.84	1.07	27.01	83.85	1.412	31.59
120	120	10	4	79	55.00	1.78	28.79	83.79	1.412	31.70
150	115	10	4	78	52.71	1.78	30.57	83.28	1.430	32.71
180	111	12	5	78	50.88	2.13	32.70	83.58	1.430	32.13
240	10	20	8	78	4.58	3.55	36.26	40.84	1.430	115.73
1440	0	25	10	78	0.00	4.44	40.70	40.70	1.430	116.01
2880	0	0	0	78	0.00	0.00	40.70	40.70	1.430	116.01
4320	0	0	0	78	0.00	0.00	40.70	40.70	1.430	116.01

Remarks: Sampling method/s - Submitted by client

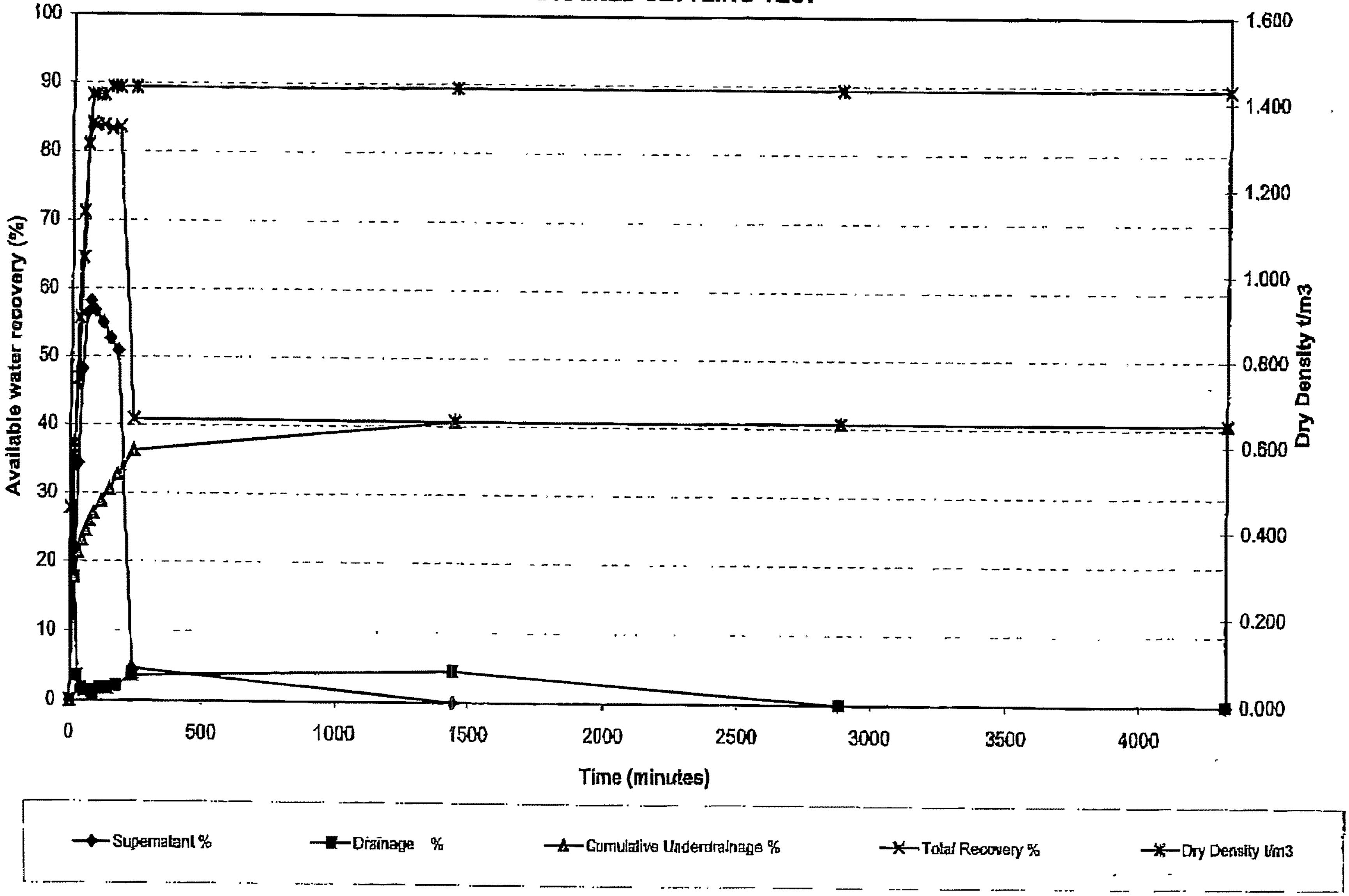
Approved:
W Rozmianiec

Date: 17.09.03

DORIS NORTH COMBINED FINAL MILL TAILINGS

SHEET NO. 9 OF 10

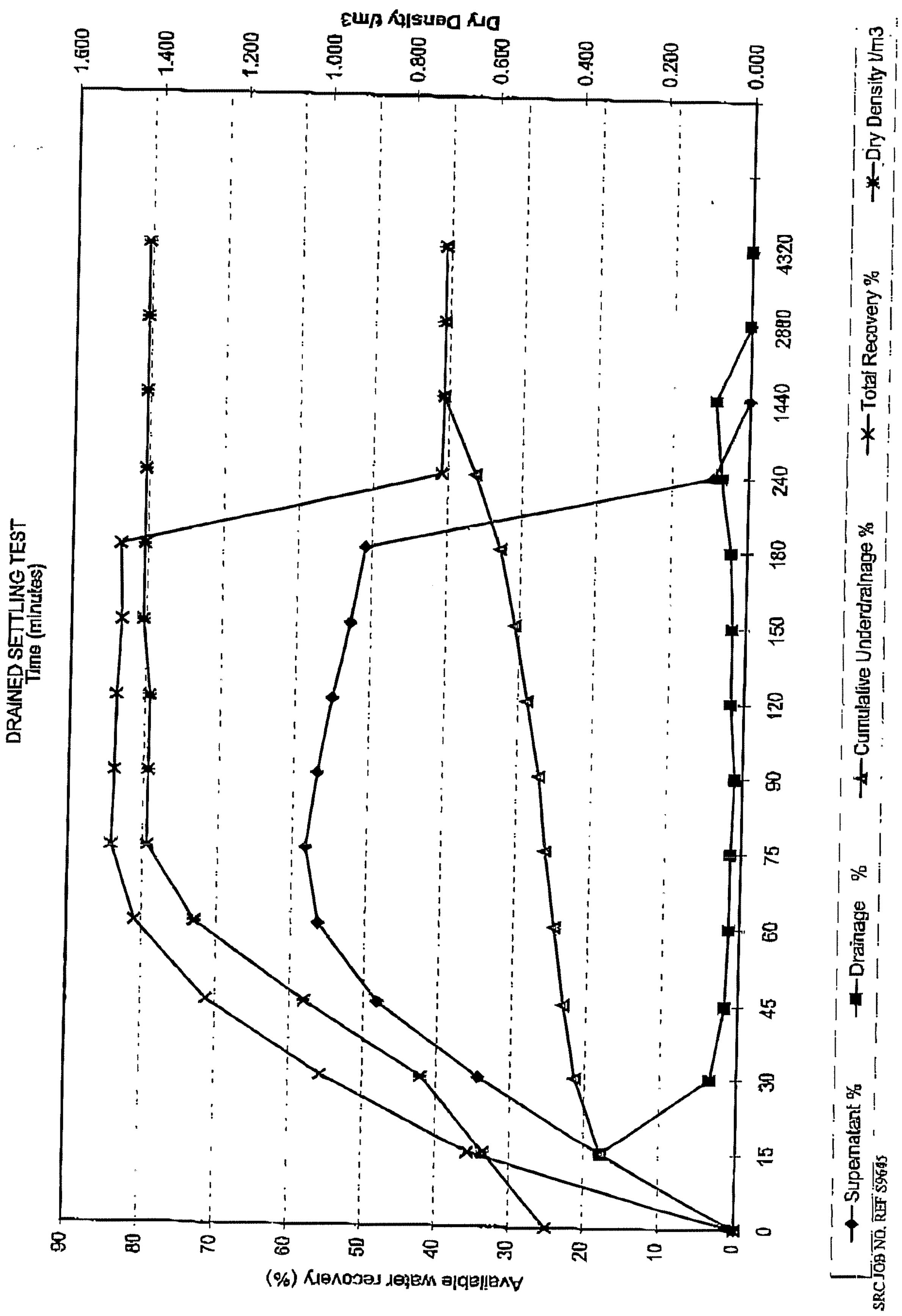
DRAINED SETTLING TEST



SRC JOB NO. REF S9645

SHEET NO.:10 OF 10

DORIS NORTH FINAL MILL TAILING



Appendix B

Thermal Design of Tailings Dams (EBA 2006)

CREATING AND DELIVERING BETTER SOLUTIONS

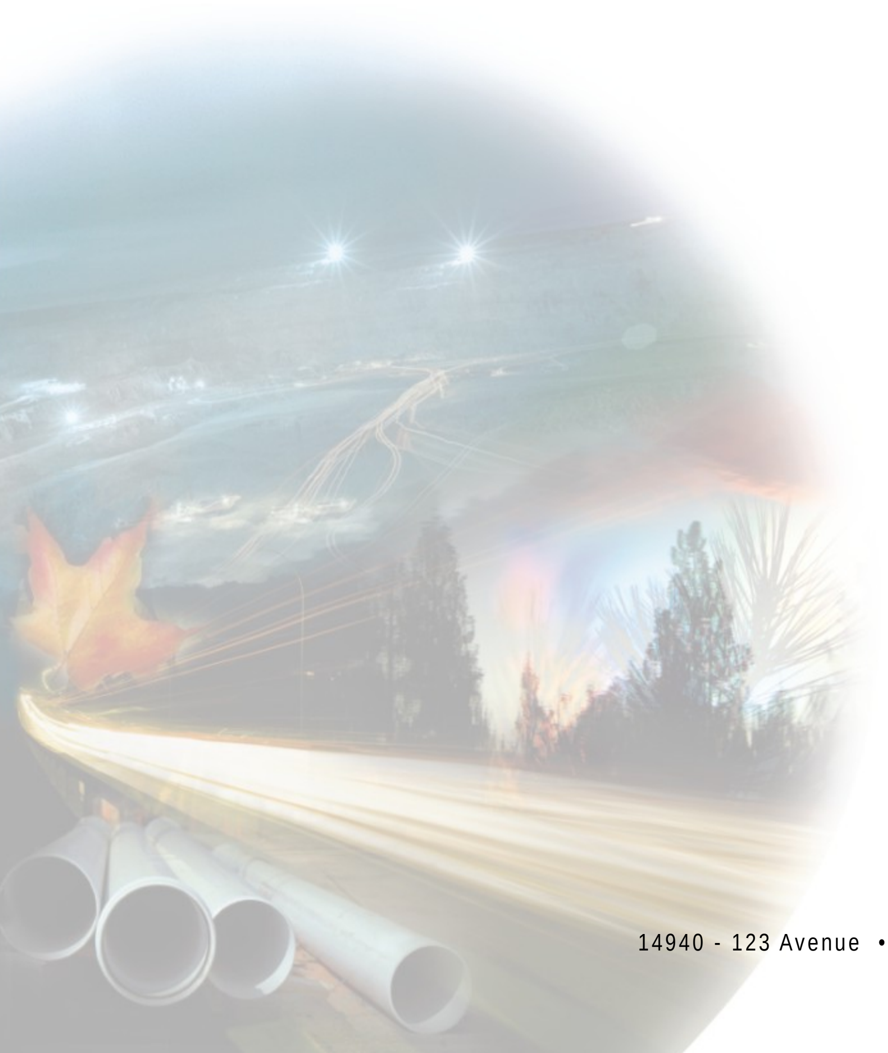
www.eba.ca

SRK Consulting (Canada) Inc.

THERMAL DESIGN OF TAILINGS DAMS
DORIS NORTH PROJECT, NU

1100126

September 2006



EBA Engineering Consultants Ltd.
p. 780.451.2121 • f. 780.454.5688
14940 - 123 Avenue • Edmonton, Alberta T5V 1B4 • CANADA

EXECUTIVE SUMMARY

The tailings water management plan for the Doris North Project requires the construction of two dams, the North Dam and the South Dam. The two dams have been designed as frozen core dams within an environment where cold permafrost soils persist. The dams are intended to have a 25-year design life. This report summarizes the analyses carried out in support of final dam design.

The North Dam is up to approximately 11.4 m high, has a 190 m crest length and is designed to retain up to 7.4 m of water at the full supply level. The South Dam is up to approximately 7.2 m high, has a 350 m crest length, and is designed to retain up to 2.7 m of water at the full supply level.

The North Dam foundation consists of predominantly ice-poor frozen sand below approximately two-thirds of the dam alignment, with the remaining portion comprising saline, marine clayey silt. The South Dam foundation is predominantly thick saline marine clays and silts overlying gravel. At both dam alignments, bedrock is relatively shallow in the abutment areas.

Each dam consists of a frozen core of nearly-saturated crushed quarry rock with a well-graded sand and gravel texture. The core is surrounded by a transition zone of processed 200 mm minus material. A geosynthetic clay liner (GCL) will be placed upstream of the core to provide seepage control if cracks were ever to develop in the frozen core. A series of horizontal thermosyphon loops will be installed within the key trench to ensure that the core and foundation soils beneath the core are sufficiently cold to limit creep-induced deformations and to be impervious to seepage. The dam shell will be constructed of run-of-quarry rock. The dam will be constructed during the winter in such a manner that the core and permafrost foundation are frozen as the dam is built and will remain in a permafrost condition throughout the dam's design life. The design has appropriate precedence from 10 years of performance history at the EKATI Diamond Mine where similar structures have been in service.

Finite element thermal analyses were carried out to predict future ground temperature within the dam and foundation. The analyses accommodate potential global warming trends by assuming that future air temperatures will rise at a rate of approximately 0.6°C per decade over the design life of the dams. The results indicate that frozen conditions are maintained with much of the core remaining colder than -2°C and the permafrost foundation soils beneath the core remaining colder than -8°C in order to enhance stability of the saline clay soils. Sensitivity thermal analyses indicated that thermosyphons installed in the key trench were required to sustain the core and underlying permafrost foundation sufficiently cold over the dams' design life. Two consecutive extreme warm years were also evaluated as an upset condition. The results indicate that the core and underlying foundation remain well-frozen even under this upset condition. Overburden soils are predicted to thaw under the upstream shell below the water impoundment. Thaw settlement will occur over time as a result of the thaw in areas where the foundation soils contain excess ground ice. The design has anticipated and accommodated predicted embankment settlement.

The portions of the dam underlain by saline marine clays and silts will undergo creep deformation due to the imposed loading from the dam embankment. Creep-deformation analyses were carried out using a two-dimensional, finite difference stress-deformation model. Relatively flat slopes (4H:1V to 6H:1V) were adopted in dam design to minimize the creep movements. The highest creep strains and stresses are located under the upstream shell. Minimal creep strains and shear stresses are predicted within the dam core and the foundation beneath the core. These strains and associated imposed stresses are viewed to be within tolerable limits and are expected to occur in a ductile manner over the life of the dam.

Total stress limit equilibrium analyses carried out for the design side-slopes indicate that the factor of safety against slope movement is greater than 1.5 and 1.1 for both the upstream and downstream slopes under static and earthquake loading, respectively. This result satisfies dam safety requirements.

The crest elevations of the dam and dam core have been designed to be over-built by 0.5 m and 1.0 m for the North and South Dams, respectively, to accommodate future settlements due to creep deformation. The predicted magnitude and rate of displacements due to permafrost creep and thaw are considered conservative. However, non-uniform displacements are anticipated because of variable soil conditions along each dam alignment. Therefore, it is recommended that a deformation monitoring program be implemented following construction to determine what measures, if any, will be required under the unlikely event that movements are greater than predicted.

TABLE OF CONTENTS

PAGE

EXECUTIVE SUMMARY i
1.0 INTRODUCTION 1
2.0 SITE DESCRIPTION 1
  2.1 Foundation Conditions 1
  2.2 Climatic Data for Thermal Modelling 2
3.0 DESIGN BASIS 4
  3.1 Physical Characteristics of Saline Frozen Soils 4
    3.1.1 Unfrozen Water in Frozen Soil 4
    3.1.2 Hydraulic Conductivity of Frozen Soils 4
    3.1.3 Mechanical Behaviour of Frozen Soils 5
  3.2 Rockfill Embankments Constructed on Saline Permafrost 7
  3.3 Frozen Core Dam Concept 8
  3.4 Dam Layout and Geometry 9
  3.5 Operating Intent 10
4.0 THERMAL EVALUATION 10
  4.1 Thermal Model 10
  4.2 Thermal Model Calibration 10
  4.3 Thermal Design Criteria 13
  4.4 Analyses Methodology 13
  4.5 Input Parameters 14
    4.5.1 Climate Parameters 14
      4.5.1.1 Global Warming 14
      4.5.1.2 Extreme Warm Year 15
      4.5.1.3 Design Climatic Conditions 15
    4.5.2 Soil Profile and Properties 16
  4.6 Initial Conditions 18
  4.7 Boundary Conditions Applied in Thermal Analyses 18
  4.8 Results of Thermal Analyses 20
    4.8.1 North Dam 20
    4.8.2 South Dam 21
    4.8.3 Summary 21

TABLE OF CONTENTS

PAGE

5.0 CREEP DEFORMATION EVALUATION 21
5.1 Creep Deformation Analysis Methodology 21
5.2 Creep Deformation Design Criteria 23
5.3 Input Parameters 23
5.3.1 Elastic Parameters 23
5.3.2 Shear Strength Parameters 24
5.3.3 Creep Parameters 24
5.4 Deformation Analyses Results 25
5.4.1 North Dam 26
5.4.2 South Dam 26
5.4.3 Discussion 27
6.0 STABILITY EVALUATION 27
6.1 Analysis Methodology 27
6.2 Design Criteria 28
6.3 Material Properties 28
6.4 Results of Stability Analyses 29
6.5 Liquefaction Potential 29
7.0 EMBANKMENT SETTLEMENT EVALUATION 29
8.0 CLOSURE 31

TABLES

Table 1 Estimated Mean (1971-2000) Climatic Conditions, Doris North Project
Table 2 Comparison of Physical Properties, Doris North Marine Clay/Silt and Svea Clay
Table 3 Physical Properties Used in Thermal Model Calibration Analyses
Table 4 Thermal Properties Used in Thermal Model Calibration Analyses
Table 5 Estimated Monthly Air Temperatures at Doris North (2004-2005)
Table 6 Design Air Temperatures Used in Thermal Analyses
Table 7 Physical Properties Used in Thermal Analyses
Table 8 Thermal Properties Used in Thermal Analyses

Table 9 Physical Properties Used to Initialize Stresses

Table 10 Shear Strength Properties Used in Deformation Analyses

Table 11 Creep Properties Used in Deformation Analyses

Table 12 Material Properties Used in Stability Analyses

Table 13 Summary of Stability Analyses Results

FIGURES

Figure 1 Historical Air Temperature Record for Kugluktuk, Cambridge Bay and Contwoyto Lake/Lupin (1959-2005).

Figure 2 Hydraulic Conductivity of Frozen, Fine-Grained Soils

Figure 3 Typical Creep Curve for Frozen Soil.

Figure 4 Relationship of the Creep Parameter B with Salinity and Temperature.

Figure 5 Generalized Cross Sections

Figure 6 Thermal Model Calibration, 2004/2005, North Dam Alignment, Boreholes SRK-15 and SRK-51

Figure 7 Thermal Model Calibration, 2004/2005, South Dam Alignment, Borehole SRK-33

Figure 8 Unfrozen Water Content Curves Used in Dam Design Thermal Analyses

Figure 9 Water Temperatures in Northern Lakes

Figure 10 Predicted Temperature Distribution, One Year After Construction, North Dam

Figure 11 Predicted Temperature Distribution, Two Years After Construction, North Dam

Figure 12 Predicted Temperature Distribution, Five Years After Construction, North Dam

Figure 13 Predicted Temperature Distribution, Ten Years After Construction, North Dam

Figure 14 Predicted Temperature Distribution, Twenty-Five Years After Construction, North Dam

Figure 15 Predicted Temperature Distribution, Two Extreme Warm Years After Construction, North Dam

Figure 16 Predicted Temperature Distribution, Ten Years After Construction, No Thermosyphons, North Dam

Figure 17 Predicted Temperature Distribution, One Year After Construction, South Dam

Figure 18 Predicted Temperature Distribution, Two Years After Construction, South Dam

Figure 19 Predicted Temperature Distribution, Five Years After Construction, South Dam

Figure 20 Predicted Temperature Distribution, Ten Years After Construction, South Dam

Figure 21 Predicted Temperature Distribution, Twenty-Five Years After Construction, South Dam

Figure 22 Predicted Temperature Distribution, Two Extreme Warm Years After Construction, South Dam

Figure 23 Predicted Temperature Distribution, Ten Years After Construction, No Thermosyphons, South Dam

Figure 24 Predicted Vertical Settlement of Crest of Frozen Core, Doris North Dams

Figure 25 Predicted Lateral Creep Deformations Ten Years After Construction, North Dam

Figure 26 Predicted Vertical Creep Deformations Ten Years After Construction, North Dam

Figure 27 Predicted Maximum Shear Strain Ten Years After Construction, North Dam

Figure 28 Predicted Maximum Shear Strain Rate Ten Years After Construction, North Dam

Figure 29 Predicted Shear Stress Distribution Ten Years After Construction, North Dam

Figure 30 Predicted Lateral Creep Deformations Ten Years After Construction, South Dam

Figure 31 Predicted Vertical Creep Deformations Ten Years After Construction, South Dam

Figure 32 Predicted Maximum Shear Strain Ten Years After Construction, South Dam

Figure 33 Predicted Maximum Shear Strain Rate Ten Years After Construction, South Dam

Figure 34 Predicted Shear Stress Distribution Ten Years After Construction, South Dam

1.0 INTRODUCTION

EBA Engineering Consultants Ltd. (EBA) was subcontracted by SRK Consulting (Canada) Inc. (SRK) on behalf of Miramar Hope Bay Limited (MHBL) to conduct thermal and creep deformation analyses and provide advice for the design of two new tailings dams proposed for the Doris North Project. This report describes the thermal and mechanical behaviour of the local overburden soils and presents the methodology and results of thermal analyses, creep deformation analyses, and slope stability analyses carried out in support of dam design.

2.0 SITE DESCRIPTION

2.1 FOUNDATION CONDITIONS

This section summarizes the interpretation of soil conditions at the North and South Dam alignments (SRK, 2005).

The North Dam alignment is within a relatively narrow valley within which subsurface conditions are characterized by two distinct zones. About two-thirds of the dam longitudinal section is dominated by an ice-saturated sand deposit that is approximately 10 m to 15 m thick. The sand deposit is overlain by a silt and clay layer that is less than 3 m thick. The remaining one-third portion is dominated by marine clayey silt that is up to 15 m thick. The fine-grained materials are also ice-saturated and contain excess ground ice. The overburden soils are up to 20 m thick at the base of the valley and thin out at the dam abutments. Bedrock is generally competent basalt.

The South Dam alignment is along a flat valley section at the watershed boundary that separates Tail Lake and Ogama Lake to the south. Soil conditions along this alignment typically consist of marine silt and clay overlying gravel. The marine deposit, which is up to 20 m thick at the base of the valley, is ice-saturated and contains excess ground ice. The gravel layer is up to 15 m thick. The overburden soils thin out at the dam abutments. Bedrock consists of basalt and argillite and is generally competent.

The Doris North Project site is located within the zone of continuous permafrost. Mean ground surface temperatures typically range from -9°C to -7°C. Permafrost thickness has been estimated to be approximately 550 m deep.

Pore water salinity measured from selected soil samples typically ranged from 30 parts per thousand (ppt) to 50 ppt. As 30 to 35 ppt is the typical salinity for sea water (Nixon and Neukirchner, 1984), the salinity measurements indicate that the marine deposit is highly saline. The pore water salinity of the sand deposit is typically 4 ppt or less and is considered non-saline.

2.2 CLIMATIC DATA FOR THERMAL MODELLING

Thermal modelling of the Doris North Project dams requires the input of climate data, including air temperature, wind speed, snow cover, and solar radiation.

A meteorological station was installed on the northern shore of Doris Lake in May 2003. Longer-term site specific data are not available. The closest meteorological station with a longer monitoring period is the Boston Camp site, located approximately 50 km south of Doris North. Climatic data have been collected at the Boston Camp site since 1993, although there are several gaps in the data set (AMEC, 2003). The closest Environment Canada meteorological stations (and the year when the station began collecting climatic data) with a long-term climatic record are as follows:

- Coppermine/Kugluktuk (operating since 1931), located approximately 320 km west of Doris North;
- Cambridge Bay (operating since 1929), located approximately 180 km northeast of Doris North;
- Contwoyto Lake/Lupin (operating since 1959), located approximately 320 km southwest of Doris North; and
- Lady Franklin Point (operating from 1957 to 1993), located approximately 260 km northwest of Doris North.

Assuming that the climate at Boston Camp is representative of Doris North, AMEC (2003) found that from the available air temperature data, the best correlation was obtained by multiple regression of the Boston data with the data from the Kugluktuk, Cambridge Bay, and Lupin stations as follows:

$$T_{Boston} = 0.3200 \cdot T_{Kugluktuk} + 0.3326 \cdot T_{CambridgeBay} + 0.3512 \cdot T_{Lupin} \quad [1]$$

where T is the daily air temperature (°C) at the respective stations.

Figure 1 compares the historical air temperature records since 1959 for Kugluktuk, Cambridge Bay, and Lupin/Contwoyto Lake. The Environment Canada stations show remarkably similar long-term trends, which implies that Doris North has likely experienced a similar warming trend. The annual air temperature history since 1998 for Doris North, computed using Equation 1 and the monthly Environment Canada station temperatures over this period, is also shown. The computed annual air temperatures at Doris North are approximately 2°C warmer than Cambridge Bay and 1°C colder than Lupin/Contwoyto Lake and Kugluktuk. The temperature comparison with Cambridge Bay agrees with the available air temperature data from Doris North since 2003 (Golder, 2005). Therefore, Equation [1] is considered reasonable for estimating long-term air temperatures. Long-term monthly air temperatures at Doris North were estimated using Equation [1] and the mean monthly air temperatures at the three Environment Canada stations, based on the

1971-2000 climate normal period. The mean annual air temperature at Doris North is estimated to be -12.1°C.

Long-term monthly mean wind speeds (1971-2000 climate normal period) are available from the Kugluktuk, Cambridge Bay and Lady Franklin Point stations. The average annual wind speed ranges from 16.1 km/h at Kugluktuk to 21.2 km/h at Cambridge Bay. Monthly wind speeds at Doris North were estimated to be the same as at Lady Franklin Point, which has an average annual wind speed of 20.1 km/h.

Long-term snow cover data (1971-2000 climate normal period) are available from Kugluktuk, Lady Franklin Point and Cambridge Bay. On average, Kugluktuk gets approximately 50 percent more snow cover than Lady Franklin Point and Cambridge Bay. The long-term monthly snow cover at Doris North was estimated to be the same as the monthly average of Lady Franklin Point and Cambridge Bay.

Daily solar radiation data are available for only a limited number of sites in the arctic. The closest meteorological station with solar radiation data is Cambridge Bay. Daily solar radiation at Doris North was assumed to be the same as that at Cambridge Bay (based on the climate normal period of 1951-1980, Environment Canada 1982).

Table 1 summarizes the long-term climatic data used in the thermal analyses.

TABLE 1: ESTIMATED MEAN (1971-2000) CLIMATIC CONDITIONS, DORIS NORTH PROJECT

	Air Temperature[(a)] (°C)	Wind Speed [(b)] (km/h)	Snow Cover[(c)] (cm)	Daily Solar Radiation[(d)] (W/m^2)
January	-31.6	19.5	21	2.2
February	-31.0	20.5	24	21.1
March	-28.0	19.6	28	87.6
April	-18.9	20.0	28	185.5
May	-7.1	20.4	22	251.9
June	3.8	20.0	0	268.1
July	9.5	18.7	0	214.6
August	7.6	19.4	0	138.5
September	1.1	21.4	0	71.8
October	-9.2	22.3	7	33.0
November	-21.5	19.9	14	4.5
December	-27.6	19.3	18	0.0

Notes: (a) Based on Equation [1] and monthly air temperatures from 1971-2000 climate normal period for Kugluktuk, Contwoyto Lake/Lupin, and Cambridge Bay.

(b) Assumed equal to monthly wind speeds from 1971-2000 climate normal period for Lady Franklin Point.

(c) Averaged from monthly snow cover from 1971-2000 climate normal period for Lady Franklin Point and Cambridge Bay.

(d) Assumed equal to daily solar radiation data from 1951-1980 climate normal period for Cambridge Bay (Environment Canada, 1982).

3.0 DESIGN BASIS

3.1 PHYSICAL CHARACTERISTICS OF SALINE FROZEN SOILS

The marine fine-grained soils beneath the dam alignments are in a permafrost condition and are saline. High pore water salt content increases the freezing point depression (i.e., lowers the temperature at which the pore water freezes) and increases the unfrozen water content (Andersland and Ladanyi, 2004). The lower ice content results in reduced frozen soil strength, increased hydraulic conductivity, and higher creep rate at a given temperature. Furthermore, it increases the potential for the frozen soil to behave as an unfrozen cohesive soil that may compress and consolidate upon loading.

3.1.1 Unfrozen Water in Frozen Soil

The water-ice phase composition in a soil or rock varies with particle mineral composition, specific surface area of the particles, salt content, and temperature (Andersland and Ladanyi, 2004).

According to Anderson and Tice (1972), the percentage of unfrozen moisture content (by dry weight of the soil) at a given temperature is given by:

$$w_{u\theta} = A \cdot \theta^{B} \qquad [2]$$

where: $A = 1.2993\ S_a^{0.5519}$

$B = -1.4495\ S_a^{-0.264}$

θ = temperature in degrees Centigrade below zero

and S_a = specific surface area of soil (m^2/g)

At a given temperature, the percentage of unfrozen moisture is greater for soils with a high specific surface area (typically fine-grained, plastic soils).

The relationship between freezing point and pore water salinity can be calculated using the following equation (Ono, 1966):

$$\theta_f = \frac{-54.11 \cdot S}{(1000 - S)} \qquad [3]$$

where θ_f is the freezing point (°C) and S is the pore water salinity (parts per thousand, or ppt). The effect of salinity is to shift the unfrozen water content curve by an amount equal to the freezing point depression of the pore water.

3.1.2 Hydraulic Conductivity of Frozen Soils

Nixon (1991) reviewed the hydraulic conductivity of frozen, fine-grained soils. Figure 2 summarizes the review of the literature. The reduction in hydraulic conductivity with colder temperatures strongly correlates with the reduction in unfrozen water content (and corresponding decrease in voids available for water percolation). To EBA's knowledge, there are no published data relating hydraulic conductivity with temperature for saline, frozen soils. However, if we assume that the saline fine-grained foundation soils at the Doris North dam alignments are at an average temperature of -8°C and have an average pore water salinity of 50 ppt (corresponding to a freezing point depression of approximately -3°C), and if we apply a temperature adjustment of 3°C as a safety factor, its hydraulic conductivity can be considered equivalent to freshwater clay/silt at a temperature of approximately -2°C. From Figure 2, the hydraulic conductivity of the saline clay and silt foundation is estimated to range from 10^{-12} cm/s to 10^{-10} cm/s, which is equivalent to unfractured rock or unweathered marine clay (Freeze and Cherry, 1979).

3.1.3 Mechanical Behaviour of Frozen Soils

Frozen soils and/or ice behave as visco-plastic materials and, as such, exhibit creep (Weaver and Morgenstern, 1981). Time-dependent deformation (creep) is composed of an instantaneous (elastic) response that is followed by decelerating (primary) creep, constant rate (secondary) creep, and at sufficient stress levels, tertiary creep (Mellor, 1979). Under moderate stress levels, the long-term deformation behaviour of ice or ice-rich soil is controlled by secondary creep, while the deformation behaviour of ice-poor soil is controlled by primary creep (Weaver and Morgenstern, 1981). Figure 3 illustrates a typical creep curve for ice-rich frozen soil or ice.

Power law models are used to model primary and secondary creep behaviour. The model not only depends on the soil characteristics, such as soil type, ice content, and temperature, but also the stress level in the soil. A model to describe the creep behaviour of frozen soils using power law relationships to represent the primary and secondary stages of creep under uniaxial compression was presented by Vyalov (1962). Sayles (1988) reviewed the available creep models and concluded that the Vyalov model provided the best approach to describe the primary phase of the creep curve. The creep law proposed by Vyalov can be expressed in the following form:

$$\varepsilon = \left[\frac{1}{w \cdot (\theta + 1)^k} \right]^n \cdot \sigma^n \cdot t^b \qquad [4]$$

where:

ε = primary strain;
w = a constant with dimensions of temperature;
θ = temperature below the freezing point of water, 0°C [C°];
σ = applied uniaxial stress [kPa];
t = elapsed time after application of the load [hours]; and
k, n, b = dimensionless constants that depend on the material properties.

Equation [4] can be expressed in a more generalized form as follows:

$$\varepsilon = B \cdot \sigma^n \cdot t^b \quad [5]$$

Differentiating Equation [5], a generalized expression for the primary strain rate, $\dot{\varepsilon}$, can be found:

$$\dot{\varepsilon} = b \cdot B \cdot \sigma^n \cdot t^{b-1} \quad [6]$$

The following flow law representing secondary creep is commonly used to describe the steady-state creep of polycrystalline ice and ice-rich frozen soils:

$$\dot{\varepsilon} = B \cdot \sigma^n \quad [7]$$

Equation [7] is a simple form of Equation [6], with b = 1. Nixon and Lem (1984) reported that this flow law is also applicable to frozen saline soils. Ladanyi (1972) described how these relationships could be modified to include the influence of confining pressure. In a two- or three-dimensional deformation analysis, the applied effective stresses and resulting effective strains and strain rates can be expressed in deviatoric terms (Odqvist, 1966).

The creep response of frozen soils generally requires specialized testing equipment and a well-controlled temperature environment over an extended period. For the vast majority of permafrost engineering design applications, creep tests are rarely conducted and secondary creep behaviour is typically characterized by data published by Morgenstern et al. (1980) and McRoberts (1988) for ice and freshwater frozen soils, and by Nixon and Lem (1984) for saline frozen soils. Laboratory creep tests of frozen, saline marine clays from Svalbard, Norway, have also been reported (e.g., Berggren, 1983; Gregersen et al., 1983; Furuberg and Berggren, 1988). Figure 4 compares the relationship of creep parameter B with salinity and temperature.

While creep is often considered the main source of deformation of frozen soils, deformations due to consolidation may also be important, particularly for frozen soils with high unfrozen water content (Andersland and Ladanyi, 2004). Currently, very little is known about the laws governing the consolidation of frozen soils under load and their temperature dependence, which makes it difficult to separate settlements due to consolidation from those due to creep (Andersland and Ladanyi, 2004).

As the hydraulic conductivity of the partially frozen marine clay and silt is expected to be relatively low (see Section 3.1.2) and the drainage boundaries for the cold permafrost foundation are expected to be large, deformations due to consolidation are anticipated to be negligible over the design life of the structures, particularly compared with creep-induced deformations. The partially-frozen marine silt and clay is expected to behave similarly to an unfrozen, low-permeability cohesive soil. At the end of construction, the foundation soil will be in a virtually undrained condition and excess pore pressures will be generated.

The physical properties of the Doris North marine clay and silt are very similar to those reported for a Norwegian clay known as Svea Clay (Berggren, 1983). Table 2 below compares some of the physical properties. It is noted that these are typical values, and that there is considerable natural variability in the recorded measurements.

TABLE 2: COMPARISON OF PHYSICAL PROPERTIES, DORIS NORTH CLAY/SILT AND SVEA CLAY

Parameter	Doris North Clay/Silt	Svea Clay (Berggren, 1983)
Natural Moisture Content	50	45
Natural Bulk Density	1.65	1.68
Clay Fraction (%)	45	40
Mineralogy	Illite, mica, chlorite, quartz, plagioclase	Chlorite, illite, albite, kaolin, quartz
Atterberg Limits (plastic limit, liquid limit, plasticity index) (% dry weight)	40, 22, 18	47, 28, 19
Natural Porewater Salinity (ppt)	45	34
Volumetric Fraction of Unfrozen Water at -5°C	0.60	0.33

Berggren (1983) reported a thawed undrained shear strength of 40 kPa for Svea Clay. This value was also considered its long-term frozen creep strength. Based on the similarity of physical properties, the undrained shear strength for the Doris North marine clay and silt has also been estimated to be 40 kPa.

3.2 ROCKFILL EMBANKMENTS CONSTRUCTED ON SALINE PERMAFROST

Rockfill embankments constructed in permafrost environments may alter the thermal regime of the foundation soils/rock and cause the permafrost subgrade to thaw. If the permafrost consists of coarse-grained materials and is ice-poor (i.e., contains less than 5 percent visible excess ground ice), permafrost thaw beneath the embankment will not generate excess pore pressures, thaw strains will be limited, and the embankment will remain stable. However, if the permafrost contains significant quantities of excess ground ice, permafrost thaw below the embankment may generate excess pore pressures, thaw strains may be significant, and the embankment may become unstable and ultimately fail.

As previously described, a large portion of the Doris North dams are founded on permafrost soils that are highly saline and contain excess ground ice. If the permafrost soils are ice-rich and/or highly saline, and the permafrost foundation is maintained in a frozen state, embankment loading may cause the permafrost to deform due to creep, which may lead to ruptures and cracks in the permafrost and embankment.

To EBA's knowledge, there are no reported case histories describing the long-term creep deformations of earthfill or rockfill embankments founded on cold, highly saline permafrost such as found at the Doris North Project site. The closest reported case histories are of road embankments founded on warm, freshwater permafrost in Alaska. Phukan (1983) reported the results from the Bonanza Creek test embankment. The 7.6 m high roadway embankment was founded on ice-rich permafrost soils that were at a temperature of -0.5°C.

Over a five-year period, the embankment crest settled at a rate of approximately 2 cm per year. McHattie and Esch (1988) reported the results from a road section near Fairbanks that was founded on ice-rich permafrost soils with a mean ground temperature of -0.4°C. Embankment settlements averaging 0.3 m per year were reported over a 10 year period for the roadway that was originally 10.5 m high. Embankment side slopes at both test sites ranged from approximately 1.5H:1V to 2H:1V. Based on the results from these case histories, it can be inferred that a large rockfill embankment founded on saline permafrost can be expected to experience similar, if not greater, magnitudes of creep deformation unless measures are taken to reduce levels of shear stress in the permafrost (by using relatively flat dam side slopes) or to thermally protect the frozen core and permafrost foundation (through the use of rockfill cover as a thermal insulator or by incorporating passive refrigeration devices known as thermosyphons), as part of dam design.

3.3 FROZEN CORE DAM CONCEPT

The nature of the foundation conditions beneath the Doris North dams is such that it is desirable to maintain the foundation soils and rock frozen rather than necessitating a massive excavation below the embankment. The frozen core design concept relies on the pores within the core and foundation soils and rock to be nearly ice-saturated to produce a well-bonded and impermeable mass.

Frozen core dams have been constructed in Canada—e.g., at the Lupin Mine (Dufour and Holubec, 1988); at the Polaris Mine (Cathro et al., 1992), at the EKATI Diamond Mine (Hayley et al., 2004), and at the Diavik Diamond Mine (Holubec et al., 2003). Frozen core dams have also been constructed in Alaska, Greenland, and Russia (Johnston and MacPherson, 1981).

Frozen core dams are particularly effective in areas of continuous permafrost, where winters are very cold and ground temperatures are typically several degrees Centigrade below freezing. What is technically challenging about the design of the Doris North dams is that significant portions of the dams are founded over thick layers of saline permafrost soils that not only contain excess ground ice but also have a high unfrozen water content. This creates a high potential for creep-deformation of the dam embankment. Therefore, alternative design concepts, such as a conventional lined dam, would not be practical under such conditions because any movement below the liner could rupture the liner. Furthermore, a frozen core dam is a more robust structure. It is essentially a rockfill structure with an impervious frozen core. A secondary seepage barrier is provided by installing a geosynthetic clay liner (GCL) against the upstream side of the core. Finally, a combination of relatively flat dam side slopes and horizontal thermosyphons installed within the key trench will minimize lateral creep displacements, vertical thaw settlements beneath the dam, and deformation of the frozen core over the design life of these structures. Thermosyphons have been installed in many of the dams at the EKATI Diamond Mine and have performed effectively to date (Hayley et al., 2004).

3.4 DAM LAYOUT AND GEOMETRY

The selected dam geometry and layout have been designed to minimize material costs, protect the integrity of the frozen core, and satisfy thermal, stability, and settlement design criteria.

As described by SRK (2005), the dams have been designed with a minimum 1.0 m freeboard for the full supply water elevation of 33.5 m. As will be discussed in Section 5, the core is predicted to settle over time due to creep deformation of the saline permafrost foundation. Therefore, the design elevations of the crest of the core have been raised by 0.5 m and 1.0 m for the North and South Dams, respectively, to accommodate future settlement of the core while maintaining minimum freeboard requirements for the first several years following dam construction.

Figure 5 shows generalized cross sections of the North and South Dams. The North Dam is approximately190 m long and has a crest width of 13 m. The crest width is required for constructability and thermal protection of the frozen core. The maximum dam height is approximately 11.4 m. The South Dam is approximately 350 m long and also has a crest width of 13 m. The maximum dam height is approximately 7.2 m.

The impervious core of the dams will be constructed of frozen, nearly-saturated well graded gravel prepared by crushing quarried rock (processed 20 mm minus). A key trench will be excavated beneath the core to 2.0 m (typical) below original grade.

Transition material (processed 200 mm minus rockfill) will be placed over the face of the frozen core to provide increased protection of the core from thermal and mechanical degradation. The transition material downstream of the core is designed to act as a filter material in the unlikely event of thaw of the core. The top of the frozen core (defined as the crest of the internal GCL within the zone of processed 20 mm minus material), will be covered with 300 mm of 20 mm minus material and 700 mm of transition material. A 1.0 m thick blanket of transition material is provided over the upstream dam foundation to provide thermal protection of the foundation soils and upstream toe of the core against convective heat transfer within the pores of the submerged shell rockfill.

Shell material has been provided for stability and thermal protection of the frozen core. The shell will consist of run-of-quarry rockfill. The upstream and downstream slopes of the North Dam will be 6H:1V and 4H:1V, respectively. Both the upstream and downstream slopes of the South Dam will be 6H:1V. The slopes are designed to maintain long-term stability by limiting potential creep deformations.

There is 2.5 m thick rockfill cover over the crest of the frozen core to prevent summer thaw of the frozen core. The core is positioned just downstream of the centre of the dam to increase the distance of the core from the impounded water, which acts as a heat source.

3.5 OPERATING INTENT

As detailed in SRK (2005), the tailings impoundment requires the construction of two dams at the north and south ends of Tail Lake. The tailings impoundment is sized to operate as a zero discharge facility during the two years of mine operation. As the water management plan is based on a water quality model, analyses by SRK indicate that Tail Lake would take between 5 and 22 years to reach the design full supply level of 33.5 m. Therefore, the dams have been designed for a minimum operational design life of 25 years.

4.0 THERMAL EVALUATION

4.1 THERMAL MODEL

Thermal analyses were carried out using EBA's proprietary finite element computer program, GEOTHERM. The model simulates transient, two-dimensional heat conduction. The program has been verified by comparison with closed form solutions and numerous field observations. The model has been the design basis for a large number of projects in the arctic and subarctic, including the design of frozen core dams at the Polaris Mine in Resolute, NU (Cathro et al., 1992) and at the EKATI Diamond Mine in Lac de Gras, NT (Hayley et al., 2004).

4.2 THERMAL MODEL CALIBRATION

The thermal model was calibrated against measured temperatures from Boreholes SRK-15 and SRK-51 at the North Dam alignment, and Borehole SRK-33 at the South Dam alignment. These locations were selected because they showed the thickest sections of saline, marine clay and silt at each alignment and were thus considered the most sensitive locations to creep-induced deformations. Ground temperatures have been collected at Boreholes SRK-15 and SRK-33 since April 2003 and at Borehole SRK-51 since April 2005. Data from both Boreholes SRK-15 and SRK-51 were used to calibrate the thermal model at the North Dam alignment because Borehole SRK-51 provided detailed temperatures only up to 5.0 m depth while Borehole SRK-15 provided deep ground temperature data (to 19.5 m depth), but with only one measurement within the top 5 m depth.

Simplified soil profiles were developed for each dam alignment, based on the soil descriptions in the borehole logs. Table 3 summarizes the soil profile and properties used at each dam alignment. The freezing point depression was calculated using Equation [3].

TABLE 3: PHYSICAL PROPERTIES USED IN THERMAL MODEL CALIBRATION ANALYSES

Material	Depth Interval (m)	Water Content (% dry weight)	Bulk Density (Mg/m³)	Pore Water Salinity (ppt)	Freezing Point Depression (°C)
North Dam (Borehole SRK-51)					
Peat/Organic Soil	0.0 – 0.4	100	1.00	0	0.0
Icy Silt	0.4 – 1.3	60	1.60	33	-1.9
Silt	1.3 – 2.1	35	1.80	33	-1.9
Silt and Clay	2.1 – 12.9	45	1.77	50	-2.9
Basalt Bedrock	12.9 – 50.0	1	2.63	0	0.0
South Dam (Borehole SRK-33)					
Peat/Organic Soil	0.0 – 0.08	100	1.00	0	0.0
Silt No. 1	0.08 – 5.6	50	1.69	40	-2.3
Clay No. 1	5.6 – 6.2	50	1.71	50	-2.9
Silt No. 2	6.2 –11.0	37	1.85	45	-2.6
Clay No. 2	11.0 – 21.3	60	1.65	45	-2.6
Silt No. 3	21.3 – 23.1	30	1.94	45	-2.6
Gravel	23.1 – 30.0	18	2.11	5	-0.3
Basalt Bedrock	30.0 – 50.0	1	2.63	0	0.0

Thermal properties were determined directly from well-established correlations with soil index properties (Farouki, 1986; Johnston, 1981). Table 4 summarizes the material properties used in the calibration thermal analyses.

TABLE 4: THERMAL PROPERTIES USED IN THERMAL MODEL CALIBRATION ANALYSES

Material	Thermal Conductivity (W/m°C)		Specific Heat (kJ/kg°C)		Latent Heat (MJ/m³)
	Frozen	Unfrozen	Frozen	Unfrozen	
Native Foundation Materials (North Dam, Borehole SRK-51)					
Peat/Organic Soil	1.45	0.62	1.41	2.46	167
Icy Silt	2.32	0.98	1.25	2.03	198
Silt	1.52	1.20	1.10	1.63	152
Silt and Clay	1.92	1.35	1.21	1.80	167
Basalt Bedrock	2.20	2.20	0.75	0.77	9
Native Foundation Materials (South Dam, Borehole SRK-33)					
Peat/Organic Soil	1.45	0.62	1.41	2.46	167
Silt No. 1	2.40	1.07	1.20	1.88	186
Clay No. 1	2.25	1.08	1.24	1.88	175
Silt No. 2	2.51	1.25	1.11	1.67	163
Clay No. 2	2.27	1.01	1.30	2.03	193
Silt No. 3	2.54	1.37	1.06	1.53	146
Gravel	2.60	1.64	0.94	1.26	108
Basalt Bedrock	2.20	2.20	0.75	0.77	9

Unfrozen water content curves were estimated for each soil layer based on the methodology described in Section 3.1.1. The estimated unfrozen water content curves were in good agreement with the measured data reported by SRK (2005).

One-dimensional thermal analyses were initially carried out in order to calibrate the model with measured ground temperatures. For simplification, the thermal model was calibrated against only the 2004/2005 measured ground temperatures. By inspection, 2003 ground temperatures were comparatively warmer than the 2004/2005 ground temperatures. This was expected, since 2003 was a relatively warm year.

The mean climatic conditions described in Table 1 were applied to the ground surface of each soil profile for a period of thirty years, by which time ground temperatures were stable from year to year. Then, beginning January 1, 2004, mean monthly air temperatures from January 2004 to September 2005 (estimated from the Kugluktuk, Cambridge Bay, and Lupin data and Equation [1] and listed in Table 5) were used. All other climate parameters were assumed to be the same as for the mean conditions described in Table 1.

TABLE 5: ESTIMATED MONTHLY AIR TEMPERATURES (°C) AT DORIS NORTH (2004-2005)

Month	2004	2005
January	-32.6	-28.8
February	-32.3	-31.8
March	-31.2	-26.7
April	-20.7	-14.5
May	-11.4	-8.1
June	4.3	4.3
July	9.9	8.3
August	6.6	8.4
September	0.7	0.4
October	-10.6	-7.8
November	-22.0	-19.0
December	-29.7	-23.7

Figures 6 and 7 compare the predicted and measured ground temperatures at the North Dam and South Dam alignments, respectively. Both figures show excellent agreement between the measured and predicted ground temperatures below 5 m depth. Within the top 5 m depth, there is very good agreement between the predicted and measured data at the end of summer (August/September) and good agreement during the spring (April-May). Based on EBA's experience, the overall match between the measured and predicted ground temperatures is considered very good. Therefore, the selected input parameters are considered reasonable and the thermal model can be used for thermal design of the tailings dams.

4.3 THERMAL DESIGN CRITERIA

A frozen core dam constructed over a saline permafrost foundation must be designed to ensure that the core and its foundation will be perennially frozen and nearly ice-saturated throughout the life of the structure. It is intended that the frozen core will be nearly saturated with freshwater ice that freezes at 0°C.

The Doris North dams have been designed to maintain "critical sections" of the core and underlying saline permafrost foundation sufficiently cold over a wide enough section to be an impervious barrier to seepage. The critical section of the core is defined as the part of the core that is colder than -2°C during impoundment under normal operating conditions or colder than -1°C during impoundment under upset conditions. A warmer design temperature has been selected for the upset condition because the probability of its occurrence is considered relatively remote. The critical section of the saline permafrost foundation is defined as the portion of the saline permafrost layer that is colder than -8°C under normal or upset operating conditions. A colder design temperature has been selected for the saline permafrost foundation compared to the frozen core because of the unfrozen water content distribution with permafrost temperature. Furthermore, colder foundation temperatures will reduce the rate and magnitude of creep-induced deformations of the frozen core and permafrost foundation.

For design purposes, the Doris North Dams must satisfy the following conditions:

- the elevation of the top of the critical section of the core must remain higher than the maximum operating level (Elevation 33.5 m);
- the width of the critical section within the core must be at least twice the head of water impounded against the dam [e.g., for the North Dam, the maximum head is approximately 7.4 m; therefore, the width of the critical section of the core at its thickest section must be at least 14.8 m]; and
- the critical section in the marine clay/silt layer must extend to the base of this layer and have a minimum width equal to the critical section of the core (e.g., it must be at least 14.8 m wide at the thickest section of the North Dam).

4.4 ANALYSES METHODOLOGY

Thermal analyses were carried out for dam cross sections near the deepest sections for each of the North and South Dam alignments. The generalized foundation profiles at each dam alignment were simplified from those used in the respective thermal model calibrations. The selected soil profiles are considered the worst foundation conditions along each proposed dam alignment from a creep-deformation perspective.

Thermal analyses were carried out to model every step from dam construction through subsequent pond impoundment. It was assumed that both the North and South Dams would be constructed the same season. The thermal analyses conservatively assumed that the water level in Tail Lake will be at Elevations 31.0 m and 33.5 m by the first and second freshets (assumed to be June 1), respectively, following construction of the dam. It was also assumed that the water level would remain at full impoundment (Elevation 33.5 m) level over the remainder of its 25-year design life.

It is understood that the South Dam could be constructed at a later date than the North Dam. This fact is not expected to influence dam design since the dam configuration is governed more by long-term creep deformation than by the thermal influence of water impoundment against the dam. Furthermore, the modeled rate and level of impoundment are very conservative, based on SRK's water balance evaluation (SRK, 2005).

Figure 1 shows that air temperatures from the closest long-term meteorological stations to the Doris North project site have increased at an average rate of approximately 0.5°C per decade since 1959. Varying degrees of warming have also been observed on a national and global scale over this same period. Therefore, the Doris North dams have been designed assuming global warming over their 25-year design life as the normal environmental condition. The effects of two consecutive extreme warm years immediately following dam construction have also been evaluated as an upset condition.

4.5 INPUT PARAMETERS

4.5.1 Climate Parameters

4.5.1.1 Global Warming

According to the International Panel on Climate Change (IPCC), a working group jointly-established by the World Meteorological Organization (WMO) and the United Nations Environment Program (UNEP) to assess scientific information on climate change, the observed recent global warming is attributed to increasing concentration of greenhouse gases.

General Circulation Models (GCMs) are mathematical representations of the global climate system that are used to explore the effects on climate behaviour of changes in the composition of the atmosphere, and specifically, human-induced changes. GCMs are highly complex and include global representations of the atmosphere, oceans, and land surface. The globe is modelled on a grid with grid sizes ranging from 41,000 km^2 to 168,000 km^2, depending on the GCM.

The IPCC's working groups use several GCMs but there are five models that are highly regarded and well-reported:

- CGCM2 – Canadian Centre for Climate Modelling and Analysis – Canada;
- ECHAM – Max-Planck Institute of Meteorology – Germany;
- GFDL-R30 – Geophysical Fluid Dynamics Laboratory – United States; and
- HadCM3 – Hadley Centre for Climate Prediction and Research – United Kingdom.

The magnitude of climate change projected by GCMs depends on the amount of change in concentration of atmospheric gases, which depends, in turn, on the emissions of radiatively-active gases into the atmosphere, from both natural and anthropogenic sources. The IPCC has developed emission scenarios based on assumptions of "storylines" of the global socio-economic conditions. These are known as the SRES emissions scenarios after the IPCC's Special Report on Emissions Scenarios (IPCC, 2000). The SRES scenarios are divided into four main "families": A1, A2, B1 and B2. The A1 and A2 families have a more economic focus than the B1 and B2 families, which are more environmentally based. The A1 and B1 families are based on storylines that assume a high degree of global coordination and uniformity, whereas A2 and B2 maintain considerable regional diversity.

GCM output for Canada is available from the Canadian Institute for Climate Studies' Canadian Climate Impacts Scenarios (CCIS) Project website: http://www.cics.uvic.ca/scenarios. The CCIS web site provides GCM output for a baseline period (1961-1990) and three future intervals (2019-2039; 2040-2069; and 2070-2099). Different GCMs and SRES scenarios result in various temperature projections, with some GCM output predicting greater temperature changes during certain seasons (e.g., during the winter) than others.

For the grid cell encompassing the Doris North site, seasonal air temperature changes from a total of 20 GCM output, derived mainly from the A2 and B2 families, were obtained for the future interval 2019-2039. These values are considered equivalent to the change in temperature over a 30-year period. According to these models, annual air temperatures at Doris North are predicted to rise by between 1.3°C and 2.5°C (median value of 1.9°C) over a 30-year period. Seasonal changes in temperature over a thirty-year period corresponding to the median GCM output are estimated to be 1.9°C (December to February), 2.1°C (March to May), 1.7°C (June to August), and 2.3°C (September to November). The median annual projection (approximately 0.6°C per decade) is greater than the observed historical trends (0.5°C per decade) in the area and is thus considered conservative for long-term thermal design of the Doris North dams.

4.5.1.2 Extreme Warm Year

There is insufficient historical air temperature data at the Doris North site to carry out a probabilistic analysis of site air temperatures. Therefore, for design purposes, the monthly air temperatures at Doris North from 1998, the warmest year on record at the Kugluktuk, Cambridge Bay and Lupin stations, were estimated from the latter stations and taken to represent the design warm-year air temperatures.

4.5.1.3 Design Climatic Conditions

Assuming that air temperatures have been linearly increasing since 1971, the mean monthly air temperatures listed in Table 1 correspond to the mid-point of the 1971-2000 period, or year 1986. Future air temperatures at Doris North were projected by increasing the mean (1986) monthly air temperatures by the corresponding rate of seasonal temperature change multiplied by the number of years since 1986. For example, the December to February monthly air temperatures are predicted to rise at a rate of 1.9°C over thirty years. December 2007 monthly air temperatures were estimated by adding the rate of seasonal temperature change (1.9°C/30 years) multiplied by the number of years since 1986 (2007 – 1986 = 21 years) to the mean (1986) monthly air temperature listed in Table 1. The design monthly air temperatures used in the thermal analyses are listed in Table 6. The other climate parameters (wind speed, snow cover and solar radiation) were assumed to be the same as the mean conditions listed in Table 1.

TABLE 6: DESIGN AIR TEMPERATURES USED IN THERMAL ANALYSES

	Global Warming		Warm-Year
	2007	2032	
January	-29.1	-27.5	-31.4
February	-28.4	-26.8	-28.5
March	-25.2	-23.4	-23.5
April	-16.6	-14.9	-15.3
May	-5.2	-3.5	-3.9
June	6.0	7.4	6.3
July	11.5	12.9	13.5
August	9.3	10.7	10.2
September	3.1	5.0	3.3
October	-7.5	-5.5	-6.2
November	-19.5	-17.6	-15.3
December	-26.0	-24.4	-14.3
Mean Annual	-10.6	-9.0	-8.8

4.5.2 Soil Profile and Properties

The design foundation soil profile and parameters are listed in Table 7. The physical properties of the embankment materials were based on past experience with similar materials. The profile and physical properties of the foundation materials were simplified from the soil layers used in the thermal model calibration analyses for each dam alignment.

TABLE 7: PHYSICAL PROPERTIES USED IN DAM THERMAL ANALYSES

Material	Depth Interval (m)	Water Content (% dry weight)	Bulk Density (Mg/m³)	Pore Water Salinity (ppt)	Freezing Point Depression (°C)
Dam Rockfill					
20 mm Core	–	11	2.28	0	0.0
Saturated 200 mm Transition	–	10	2.31	0	0.0
Unsaturated 200 mm Transition	–	4	2.18	0	0.0
Saturated Shell	–	13	2.37	0	0.0
Unsaturated Shell	–	3	2.16	0	0.0
Native Foundation Materials (North Dam 2-D)					
Silt and Clay	0.0 –13.0	45	1.77	50	-2.9
Basalt Bedrock	13.0 – 75.0	1	2.63	1	0
Native Foundation Materials (South Dam 2-D)					
Silt	0.0 – 11.0	45	1.71	40	-2.3
Clay	11.0 – 23.0	50	1.65	45	-2.6
Gravel	23.0 – 30.0	18	2.11	5	-0.3
Basalt Bedrock	30.0 – 75.0	1	2.63	0	0.0

Table 8 lists the thermal properties used in the thermal analyses. These values were determined directly from well-established correlations with soil index properties (Farouki, 1986 ; Johnston, 1981).

TABLE 8: THERMAL PROPERTIES USED IN DAM THERMAL ANALYSES

Material	Thermal Conductivity (W/m°C)		Specific Heat (kJ/kg°C)		Latent Heat(MJ/m³)
	Frozen	Unfrozen	Frozen	Unfrozen	
Dam Rockfill					
20 mm Core	2.64	1.97	0.87	1.07	75
Saturated 200 mm Transition	2.70	2.05	0.86	1.05	70
Unsaturated 200 mm Transition	1.72	1.77	0.79	0.87	28
Saturated Shell	2.80	10.00/2.06(a)	0.89	1.13	91
Unsaturated Shell	1.56	1.68	0.77	0.83	21
Native Foundation Materials (North Dam)					
Silt and Clay	1.92	1.35	1.21	1.80	167
Basalt Bedrock	2.20	2.20	0.75	0.77	9
Native Foundation Materials (South Dam)					
Silt	2.34	1.09	1.17	1.80	174
Clay	2.27	1.01	1.30	2.03	193
Gravel	2.60	1.64	0.94	1.26	108
Basalt Bedrock	2.20	2.20	0.75	0.77	9

Note: (a) The thermal conductivity of the saturated shell was assumed to be 2.06 W/m-°C in the vertical direction and 10.0 W/m-°C in the horizontal direction. The larger horizontal thermal conductivity was used to account for potential heat transfer by convection through the open voids of the saturated shell material.

The unfrozen water content curves estimated for the permafrost foundation soils are shown on Figure 8. These curves compare well against the measured data reported by SRK (2005).

4.6 INITIAL CONDITIONS

It was assumed that construction of each dam would be carried out in the winter of 2007/08 and completed by April 2008.

Initial temperatures of the embankment and foundation were estimated based on monitoring records of similar dam height sections during construction of frozen core dams at EKATI, and from thermal calibration of measured ground temperatures from Boreholes SRK-51 and SRK-15 for the North Dam, and Borehole SRK-33 for the South Dam.

4.7 BOUNDARY CONDITIONS APPLIED IN THERMAL ANALYSES

Climatic conditions were applied at the surface of the dam and ground exposed to air. Mean snow cover was applied to the natural ground surface and over the slope face of the dam, except for the case of the downstream slope of the North Dam where, because of the relatively steep slope (4H:1V), snow drifting was assumed at the downstream toe. In this case, the snow cover was assumed to vary linearly from mean at a distance of 7 m on either side of the slope toe to three times the mean at the slope toe. Snow cover at the crest of both dams was reduced to 10 percent of the mean snow depth to account for wind-blown snow cover that is typical around topographic highs in the area.

Water from the initial impoundment will infiltrate the upstream dam shell as water rises against the dam. Prior to impoundment, the upstream shell and transition are generally frozen and unsaturated. It has been assumed that impounded water will seep through the shell and into the transition zone, thereby raising the temperature of the upstream shell and transition. Upon initial impoundment, the elements representing the saturated upstream shell and transition were assigned a temperature of 0.01°C.

There are limited published temperature measurements in northern lakes. Temperature data from two lakes, one located in northern Manitoba (P&N Lake) (Welch and Bergmann, 1986) and another located in the Mackenzie Delta, NT (Todd Lake) (Burn, 2002) are shown in Figure 9. Based on these two data sets, the temperature history of Tail Lake was estimated, as shown in Figure 9. The thermal influence of the water impounded against the dam was modeled as a temperature boundary for all nodal points along the dam slope and natural ground surface at and below the water level.

The thermosyphons were modelled as a convective boundary. The U.S. Army Cold Regions Research and Engineering Laboratory conducted laboratory tests of full-scale, two-phase thermosyphons to relate heat transfer to wind speed and evaporator slope angle (Haynes and Zarling, 1988). For a thermosyphon with a 6.5 m^2 radiator and a horizontal evaporator, heat transfer conductance, $\left[\frac{Q}{\Delta T}\right]_{6.5}$ (W/°C), is expressed by the following equation:

$$\left[\frac{Q}{\Delta T}\right]_{6.5} = 8.70 + 12.29 \cdot V_w^{0.83} \qquad [8]$$

where V_w is the wind velocity (m/s). The heat transfer conductance is a measure of the ability of a thermosyphon system to extract heat from the evaporator section and release it to the atmosphere. For different size radiators, Jardine et al. (1992) indicate that the heat transfer conductance, $\left[\frac{Q}{\Delta T}\right]_{area}$, can be determined by multiplying $\left[\frac{Q}{\Delta T}\right]_{6.5}$ by the ratio of the radiator area (m^2) to the reference radiator area (6.5 m^2).

The heat transfer coefficient, H_c (W/m^2), can then be computed from the following equation:

$$H_c = \frac{\left[\frac{Q}{\Delta T}\right]_{area}}{2\pi r \ell} \qquad [9]$$

where: r = evaporator pipe radius (m) and

ℓ = effective length of buried evaporator pipe (m).

Convective heat transfer, Q_h (W), is calculated by the following:

$$Q_h = H_c \cdot (T_s - T_a) \quad [10]$$

where T_s is the temperature of soil around the thermosyphon evaporator and T_a is the ambient air temperature.

As a safety factor, thermosyphon heat transfer conductance was calculated assuming a low wind speed -- 13 km/h, or approximately two-thirds of the mean winter wind speed. The thermosyphons provide cooling only when ambient air temperatures are colder than the ground temperatures surrounding the evaporator pipes.

4.8 RESULTS OF THERMAL ANALYSES

4.8.1 North Dam

Figures 10 to 14 show the ground temperature distribution during the month of December in Years 1, 2, 5, 10 and 25, respectively following dam construction. December is the time of year when temperatures below the frozen core are warmest. The results show that upstream of the frozen core, the permafrost foundation is progressively getting warmer over the years in response to the continued warming influence of the lake impoundment. If we assume that the marine clay and silt layer fully thaws at -3°C, much of the marine clay and silt upstream of the core is predicted to thaw after 25 years of full impoundment. Downstream of the frozen core, the permafrost foundation is predicted to progressively warm due to the long-term effects of snow drifting and the modelled climate warming trend. The degree of permafrost warming below the downstream shell is not expected to be as great as below the upstream shell. Permafrost thaw will result in settlement as excess ground ice melts and dissipates under loading.

Beneath the frozen core, permafrost temperatures are predicted to progressively cool with time over the first ten years or so due to the thermal influence of the thermosyphons but then become warmer because of long-term climate warming. The results show that the frozen core is well frozen (temperatures mainly colder than -5°C), as is the permafrost foundation beneath the core (temperatures colder than -8°C).

An upset condition was evaluated by assuming extreme warm years for the first two years following initial impoundment. The short-term, extreme warm year has a greater influence on dam embankment temperatures (e.g., active layer penetration) than on the permafrost foundation temperatures, which are largely controlled by long-term temperature trends. Figure 15 shows the predicted early-October temperature distribution the year following initial impoundment. October is the time of year when temperatures near the top of the frozen core are warmest. Figure 15 shows that the frozen core is colder than -1°C. This indicates that the 2.5 m rockfill cover over the core crest provides sufficient thermal insulation to maintain the core perennially frozen even under extreme temperatures.

As a sensitivity evaluation, analyses were conducted assuming that there were no thermosyphons installed within the core. Figure 16 presents the predicted December temperatures ten years after construction, assuming the same climate warming scenario adopted for the earlier analyses with thermosyphons. The permafrost foundation beneath the core is predicted to warm to a temperature of between -7°C and -8°C. This compares to the base case (with thermosyphons) in Figure 13, where temperatures in the permafrost foundation beneath the core ten years after construction are predicted to be colder than -9°C.

4.8.2 South Dam

Figures 17 to 21 show the ground temperature distribution during the month of December in Years 1, 2, 5, 10 and 25, respectively following dam construction. The long-term geothermal response of the dam embankment and its permafrost foundation is similar to that described for the North Dam above. Permafrost foundation temperatures beneath the core are predicted to remain colder than -8°C over the 25-year design life.

Figure 22 shows the predicted early-October temperature distribution the year following initial impoundment for the upset condition of two extreme warm years following dam construction. As with the North Dam, the frozen core and permafrost foundation are predicted to be colder than -1°C and -8°C, respectively.

Figure 23 shows the predicted December temperature distribution ten years after dam construction, assuming no thermosyphons were installed within the core. The marine silt and clay layers are predicted to warm to between -7°C and -8°C, compared with temperatures colder than -8°C for the base case (with thermosyphons) shown in Figure 20.

4.8.3 Summary

The results of the thermal analyses demonstrate that the Doris North dams are predicted to maintain the "critical sections" of the frozen core and underlying marine clay and silt permafrost foundation sufficiently cold and over a wide enough section, even under reasonable worst-case upset conditions. Therefore, the Doris North dams satisfy the thermal design criteria described in Section 4.3. The results also show the need for thermosyphons to be included in dam design.

5.0 CREEP DEFORMATION EVALUATION

5.1 CREEP DEFORMATION ANALYSIS METHODOLOGY

Creep deformation analyses were carried out for the same dam section and foundation conditions used in the thermal evaluation. Two-dimensional plane-strain analyses were carried out using FLAC (Fast Lagrangian Analysis of Continua), a commercial, two-dimensional, explicit finite difference stress analysis program developed by HCItasca.

The results of the thermal analyses show that temperatures below and within the dam vary seasonally and from year-to-year in response to variations in surface temperature and thermosyphon cooling. At the base of the frozen core, temperatures are predicted to initially become colder with time because of thermosyphon cooling but then gradually warm over time in response to long-term climate warming. Accurate predictions of creep deformations require a coupled thermal-creep deformation model since the creep behaviour of frozen soils containing excess ground ice is strongly temperature-dependent. There is currently no available commercial software that can implement this efficiently. Long-term creep displacements were therefore estimated from the predicted ground temperature distribution ten years after dam construction (Figures 13 and 20 for the North and South Dams, respectively). The temperature-dependent creep parameters were applied to the model element based on the predicted temperatures. This time interval (ten years) was selected because it is considered the most representative for long-term creep deformation predictions. The procedure used in the analyses is as follows:

- Initialize stresses in the dam embankment and permafrost foundation by assuming elastic parameters and turning gravity on.
- Change the dam shell and transition materials to Mohr-Coulomb parameters and bring the model again to equilibrium.
- Change the frozen core and permafrost soils to creeping materials. Assign creep properties to the model elements based on the predicted ten-year ground temperature distribution.
- Set the model to allow for large-strain deformations (i.e., deforming grid) and compute stresses and deformations for a 25-year period.

Roller boundaries were assigned to the left and right ends of the grid, and a fixed boundary was assigned to the base of the grid.

EBA has found that the FLAC creep model reasonably predicts the observed deformations of the Alder Creek Valley, Alaska road embankment constructed on warm, freshwater permafrost soils (McHattie and Esch, 1988). As described in Section 3.2, there is no documented case history describing the long-term performance of an embankment constructed on a saline permafrost foundation such as that found at the Doris North site. However, pile load tests by Nixon (1988) and Biggar and Kong (2001) in areas of saline permafrost have shown that the power law creep model described in Section 3.1.3 also applies to pile creep behaviour in saline permafrost. Therefore, the viscoelastic power law model used in FLAC is considered a realistic method to predict creep deformations of the dam embankment.

5.2 CREEP DEFORMATION DESIGN CRITERIA

Strains will develop within the dam embankment and underlying permafrost foundation in response to long-term creep deformations. Failure strains in frozen soils decrease with warmer temperatures and lower strain rates (Andersland and Ladanyi, 2004). Bragg and Andersland (1982) conducted laboratory creep tests on frozen sand and observed that at low strain rates (i.e., less than 10^{-5} s^{-1}), the frozen sand deformed elastically in the early stages of deformation followed by an initial yield or rapid change in slope. Beyond the peak stress, the frozen sand deforms in a ductile fashion with no visible cracking or formation of shear planes at strains. Minimum axial strains of approximately 8 percent were required for the frozen sand to fail at these low strain rates. At high strain rates (i.e., greater than 4 x 10^{-5} s^{-1}), the frozen sand exhibits brittle failure. The frozen core is anticipated to behave similarly to a frozen sand.

Sego and Morgenstern (1983) conducted constant strain rate experiments on polycrystalline ice and observed that at low strain rates (i.e., less than 8.3 x 10^{-5} s^{-1}), the ice did not collapse in brittle failure or deform to tertiary creep to strains of up to 15 percent. These findings indicate that ice undergoes ductile deformation at low strain rates, even up to 15 percent strain.

Berggren (1983) conducted constant stress tests on Svea clay, a saline marine clay from Svalbard, Norway. Test results indicated that the frozen saline clay undergoes ductile deformation at low strain rates (less than 10^{-5} s^{-1}), even up to 20 to 30 percent strain.

Based on these observations, and using engineering judgment about the risk of brittle rupture, the Doris North dams have been designed to maintain the long-term integrity of the frozen core and permafrost foundation by limiting the long term shear strains in these two areas to less than 2 and 10 percent, respectively. Localized zones of collapse or cracking away from the frozen core are tolerated, as this should not jeopardize the overall stability of the embankment or the ability of the frozen core and permafrost foundation soils to retain water.

5.3 INPUT PARAMETERS

5.3.1 Elastic Parameters

The stress state in the dam and foundation must be generated prior to the onset of creep. An elastic stress-strain analysis was carried out to determine the initial stress state. The material properties used in the stress initialization are given in Table 9. The elastic properties of the dam shell and transition were based on mean values for unfrozen, compact sand and gravel (Bowles, 1982). The elastic moduli for the frozen core and permafrost foundation soils were estimated from empirical equations for freshwater frozen soils (Johnston, 1981) and applying engineering judgment to consider reduced stiffness in saline frozen soils because of unfrozen water content. The values for Poisson's ratio were based on typical mean values for the materials considered (Bowles, 1982, Johnston, 1981).

The initial stress conditions were generated by virtue of a total stress analysis with no water impoundment.

TABLE 9: PHYSICAL PROPERTIES USED TO INITIALIZE STRESSES

Material	Elastic Modulus (kPa)	Poisson's Ratio ν	Unit Weight (kN/m³)
Bedrock	1.0 x 10^8	0.25	26
Shell	1.2 x 10^5	0.25	22
20 mm Core	1.0 x 10^5	0.35	22
200 mm Transition	1.0 x 10^5	0.30	21
North Dam			
Silt and Clay	5.0 x 10^4	0.35	17
South Dam			
Silt	5.0 x 10^4	0.35	17
Clay	5.0 x 10^4	0.35	17
Gravel	1.0 x 10^5	0.35	22

5.3.2 Shear Strength Parameters

The dam embankment materials were changed from elastic to Mohr-Coulomb materials after the stresses were initialized. The shear strength parameters are listed in Table 10. The shear strength parameters for the dam shell and transition were estimated based on engineering judgment and are believed to be conservative for these materials. The shear strength of the thawed marine clay/silt was based on published values for a similar saline marine clay, Svea Clay (Berggren, 1983), as described in Section 3.1.3

TABLE 10: SHEAR STRENGTH PROPERTIES USED IN THE DEFORMATION ANALYSES

Material	Cohesion (kPa)	Angle of Internal Friction (°)
Bedrock	1000	-
Shell	-	40
200 mm Transition	-	35
Thawed Marine Clay/Silt	40	-

5.3.3 Creep Parameters

The frozen core can be considered to behave like saturated, frozen sand. Creep properties for the frozen core were based on those for Ottawa Sand (Sayles, 1968).

The marine silt and clay behaves like saline frozen soil. Figure 4 shows the estimated temperature-dependent B parameter curve, extrapolated from published data for freshwater and saline frozen soils by McRoberts (1988) and Nixon and Lem (1984), respectively.

Table 11 summarizes the temperature-dependent creep properties used in the deformation analyses. The temperature-dependent elastic modulus for the marine clay/silt was estimated

Generally, creep deformation is considered to be a non-volume change phenomenon (i.e., Poisson's ration = 0.5).

TABLE 11: CREEP PROPERTIES USED IN DEFORMATION ANALYSES

Material	n	b	B (kPa^{-n} x yr^{-b})	Elastic Modulus (kPa)
Marine Clay/Silt[(a)]				
Thawed (> -3°C)		-	-	5.0×10^{3}
-4°C	3	1	4.0×10^{-5}	3.0×10^{4}
-5°C	3	1	7.0×10^{-6}	6.0×10^{4}
-6°C	3	1	2.0×10^{-6}	9.0×10^{4}
-7°C	3	1	7.0×10^{-7}	2.0×10^{5}
-8°C	3	1	2.0×10^{-7}	5.0×10^{5}
-9°C	3	1	1.0×10^{-7}	8.0×10^{5}
20 mm Core[(b)]				
Above Key Trench (-3°C)	1.32	0.26	5.9×10^{-7}	1.0×10^{5}
Within Key Trench (-5°C)	1.32	0.26	3.2×10^{-7}	1.8×10^{5}

Source: (a) Extrapolated, as shown in Figure 4
(b) Sayles (1968)

5.4 DEFORMATION ANALYSES RESULTS

Figure 24 presents the predicted vertical settlement histories at the crest of the frozen core for both the North and South Dams. These have been calculated based solely on creep deformation and ignore displacements due to other mechanisms (e.g., thaw settlement, consolidation, and compression). Substantial settlement of the cores of both dams is predicted. In fact, the predicted rates of settlement are such that the crests of the cores will reach the minimum allowable elevation that satisfies freeboard requirements (El. 34.5 m) by approximately 8 to 10 years following dam construction.

It should be noted that the predicted creep deformations are considered very conservative. First, they are based on geothermal predictions of a pessimistic scenario that assumes that Tail Lake will reach full supply level (El. 33.5 m) within two years of dam construction and be maintained at that level for the 25-year design life. A more realistic water level scenario—one in which the water level is well below the full supply level—would result in colder permafrost foundation temperatures, especially at the South Dam, where the water level against the dam will likely be sufficiently low that the water would freeze to the bottom each winter and the foundation soils should remain in a permafrost condition. Secondly, they are based on the predicted ground thermal regime ten years following dam construction. While this approach may best-represent long-term conditions, this approach will over-predict the rate of creep deformations for the first few years or so following dam construction.

The design approach adopted for establishing the minimum freeboard requirements of the Doris North dams has been to initially over-build the crest of the frozen core to accommodate some, but not all, of the predicted settlement over the 25-year design life. This approach is taken in part because the predicted dam deformations are believed to be conservative, and because additional embankment loading associated with raising the height of the dam to satisfy minimum freeboard requirements over the entire 25-year design life would increase the rate of creep deformations. It is intended that the dams will be regularly surveyed following construction. The predicted movements will be slow to develop; therefore, monitoring will provide additional assurance of satisfactory performance. In the unlikely event that movements are greater than predicted, the dam slopes could be flattened, berms constructed, or the foundation could be cooled with additional thermosyphons.

For the reasons described above, the results presented below are for the elapsed time of ten years after dam construction. The creep deformation analyses did not consider any future dam reconstruction activities, such as redistributing rockfill, that would affect the predicted rate or magnitude of deformations.

5.4.1 North Dam

Figures 25 and 26 show the predicted horizontal and vertical displacements, respectively, ten years after construction of the North Dam. The results show that the predicted deformations are largely away from the frozen core. Because the saline permafrost soils are warmer upstream and downstream of the frozen core, embankment loading causes the permafrost foundation soils beneath the core to "squeeze" laterally, causing the core to settle. Greater displacements are predicted upstream of the frozen core because of the predicted warmer foundation temperatures. The core itself is expected to behave rigidly.

Figures 27 and 28 show the predicted maximum shear strain and maximum shear strain rate, respectively, ten years after construction. Figure 27 shows that the most likely mechanisms of slope failure are deep-seated, rotational slips located upstream and downstream of the frozen core. Furthermore, the maximum shear strains within the frozen core and permafrost foundation beneath the core are predicted to be less than 2 percent and 10 percent, respectively. Figure 28 shows that the maximum shear strain rates within and beneath the frozen core are predicted to be less than $0.2 \times 10^{-6} \text{yr}^{-1}$ or $6.3 \times 10^{-14} \text{s}^{-1}$. At the given strains and strain rate, the frozen core and permafrost foundation are expected to remain ductile and brittle rupture is not expected.

Figure 29 shows the predicted shear stress distribution ten years after construction. The shear stresses are predicted to be generally less than 20 kPa. The strengths of the frozen core and underlying permafrost foundation soils are expected to well exceed these stress levels.

5.4.2 South Dam

Figures 30 and 31 show the predicted horizontal and vertical displacements, respectively, ten years after construction of the South Dam. Deformation trends at the South Dam are similar to those at the North Dam. Although the dam is thinner and is designed to retain a lower head of water than the North Dam, even greater creep deformations are predicted at the South Dam because the creep-susceptible marine clay and silt foundation is much thicker at this dam alignment than it is at the North Dam alignment.

Figures 32 and 33 show the predicted maximum shear strain and maximum shear strain rate, respectively, ten years after construction. Figure 32 shows that the most likely mechanism of slope failure is a deep-seated rotational slip surface upstream of the frozen core. The maximum shear strains developing within the frozen core and underlying permafrost foundation are predicted to be less than 1 percent and 8 percent, respectively. Furthermore, the maximum shear strain rate is predicted to be less than 1.5 x 10^{-5} yr^{-1} or 4.8 x 10^{-13} s^{-1}. At these strains and strain rates, the frozen core and underlying permafrost foundation are expected to remain ductile and brittle rupture is not expected.

Figure 34 shows the predicted shear stress distribution ten years after construction. The shear stresses are predicted to be generally less than 30 kPa. The strength of the frozen core and underlying permafrost foundation are expected to exceed these stress levels.

5.4.3 Discussion

The results of the creep deformation analyses indicate relatively high movement and strains in the foundation upstream and downstream of the frozen core, and relatively small movements and strains in the dam core and underlying foundation soils.

The strains are predicted to occur very slowly and in a ductile manner. The monitoring program will identify long term displacement and potential stability concerns in time to allow mitigating measures to be undertaken. With these safeguards in place, acceptable dam performance is anticipated.

6.0 STABILITY EVALUATION

6.1 ANALYSIS METHODOLOGY

Limit equilibrium analyses were conducted to determine the factor of safety against slope failure during construction and operation of the dam. All analyses were conducted using the commercially available two-dimensional, limit equilibrium software, SLOPE/W, developed by Geo-Slope International Ltd. The principles underlying the method of limit equilibrium analyses of slope stability are as follows (Morgenstern and Sangrey, 1978):

- A slip mechanism is postulated;
- The shear resistance required to equilibrate the assumed slip mechanism is calculated by means of statics;
- The calculated shear resistance required for equilibrium is compared with the available shear strength in terms of factor of safety; and

- The slip mechanism with the lowest factor of safety is determined through iteration.

Factor of safety is used to account for the uncertainty and variability in the strength and pore water pressure parameters, and to limit deformations.

Earthquake loading was modelled using a pseudo-static peak horizontal ground acceleration. According to SRK (2005), an earthquake with a 2,475 year return period coincides with a peak ground acceleration of 0.06 g; this value was used in the pseudo-static assessment.

Stability analyses were carried out for the dam sections near its maximum thickness. The same dam geometry and soil profile used in the thermal and creep deformation analyses were used in the stability evaluation. Analyses were carried out for two cases: one assuming full impoundment (i.e., water level = 33.5 m), and another assuming no water against the dam. A deep-seated slip surface and failure along the GCL liner were evaluated.

6.2 DESIGN CRITERIA

The slopes of the Doris North Dams were designed so that the dam, foundation and abutments are stable under all stages of construction, reservoir levels, and operating conditions, in accordance with the Dam Safety Guidelines (CDA, 1999). The minimum acceptable factors of safety specified in the Dam Safety Guidelines are 1.5 under static loading conditions and 1.1 under earthquake loading conditions.

6.3 MATERIAL PROPERTIES

The material properties chosen for the embankment and foundation materials in the stability analyses are presented in Table 12.

TABLE 12: MATERIAL PROPERTIES USED IN STABILITY ANALYSES

Material	Angle of Internal Friction (°)	Cohesion (kPa)	Unit Weight (kN/m^3)
Run-of-Quarry (shell)	40	--	20
200 mm Material (transition)	35	--	21
20 mm Material (core)	32	--	21
GCL	15	--	18
Marine Silt/Clay (Undrained)	--	40	17

Derivation of the shear strengths of the shell, transition, and marine silt/clay was previously described in Section 5.3.2.

The long-term strength of the frozen core (20 mm material) was represented by conventional effective stress strength parameters, based on studies by McRoberts and Morgenstern (1974), which showed that the long-term strength of ice-poor soil was frictional.

The friction angle for the GCL (15°) was determined from published residual shear strengths of reinforced GCL as measured in the laboratory (Gilbert et al., 1996; Richardson, 1997).

6.4 RESULTS OF STABILITY ANALYSES

Table 13 summarizes the computed minimum factors of safety for static and earthquake loading conditions. Minimum factors of safety were determined from cases where there was no water impounded against the dam. This is because in total stress analyses, the weight of the water acts as a resisting load against slope failures beneath the upstream slope.

TABLE 13: SUMMARY OF STABILITY ANALYSES RESULTS

Loading Condition	North Dam		South Dam	
	Upstream	**Downstream**	**Upstream**	**Downstream**
Static	1.6	1.5	1.7	1.7
Earthquake	1.1	1.1	1.1	1.1

The minimum factors of safety listed in Table 13 are for deep-seated, circular slip failures from the opposite slope (e.g., from the downstream dam slope for the upstream slip surface), through the marine silt/clay layer and daylighting beyond the slope toe. The calculated factors of safety satisfy dam safety requirements.

6.5 LIQUEFACTION POTENTIAL

The peak horizontal ground acceleration for the area is very low at 0.06 g and, as a consequence, liquefaction of the thawed marine silt and clay due to earthquake loading is not expected to be a concern.

7.0 EMBANKMENT SETTLEMENT EVALUATION

Thaw settlements were estimated based on the predicted thaw penetration into the typical cross section described in the thermal evaluation (Section 4). Large portions of the North and South Dams are founded on thick layers of frozen saline, fine-grained soils. In the extreme condition where the dams continuously retain water at its maximum operating level over a 25-year period, up to 6 m of the frozen saline fine-grained soils is predicted to thaw below the upstream shell; however, below the core, the permafrost is predicted to remain well-frozen. Assuming that the permafrost contains approximately 20 to 40 percent excess ground ice, thaw strains of up to 20 to 40 percent can be expected under the upstream shell and toe. Thaw settlements are not expected to affect the integrity of the frozen core.

Total deformations are predicted to occur due to permafrost creep and thaw and, to a lesser extent, consolidation of the marine clay and silt foundation soils. Given the variability in soil conditions along each dam alignment, there is a high potential for differential movements across the dam embankment. This is particularly true at the North Dam, where most of the dam is sited on non-saline, ice-poor frozen sand and displacements are expected to be relatively small, compared to the remaining portion of the dam, which is sited on saline clays and silts, over which relatively large displacements are predicted.

The predicted deformations are considered conservative and are expected to develop slowly. The dam monitoring data should be regularly reviewed to verify that the dams are behaving as predicted. Remediation measures, such as flattening slopes or installing thermosyphons, should be implemented should differential dam movements be seen to pose a risk of rupture of the frozen core and/or permafrost foundation.

8.0 CLOSURE

The information contained in this report is based on the best available data at the time of its preparation. Engineering judgment has been applied in developing the designs contained in this report.

This report has been prepared for the exclusive use of SRK and the owner, Miramar Hope Bay Limited, for the specific use at the Doris North site.

Respectfully submitted,
EBA Engineering Consultants Ltd.

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GEOLOGISTS and GEOPHYSICISTS
OF THE NORTHWEST TERRITORIES
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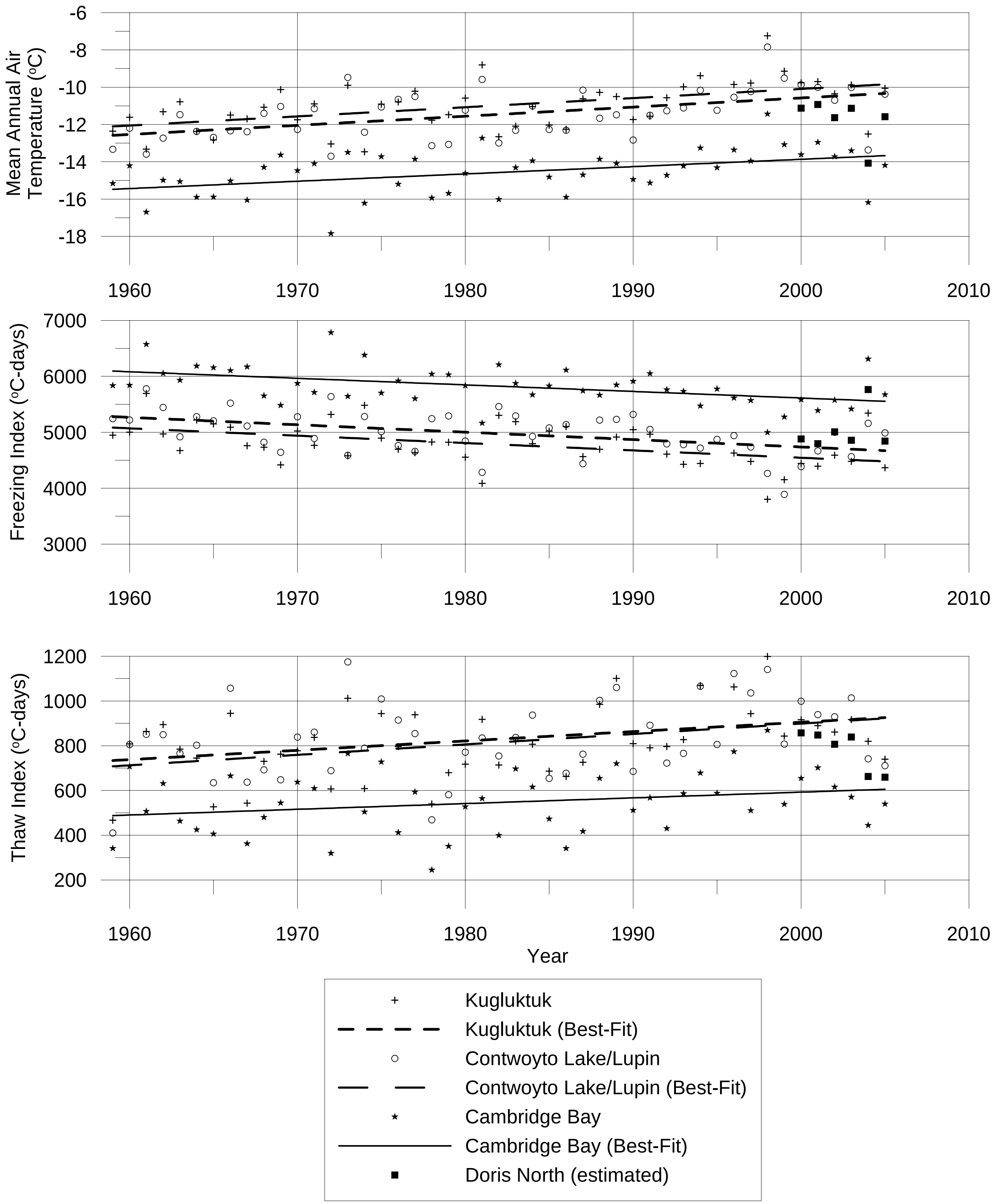
FIGURES



Notes:
1. Kugluktuk, Cambridge Bay, and Contwoyto Lake/Lupin data from Environment Canada's website.
2. Doris North air temperature estimated from above three meteorological stations.

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FILE: 1100126_AirTemps.grf

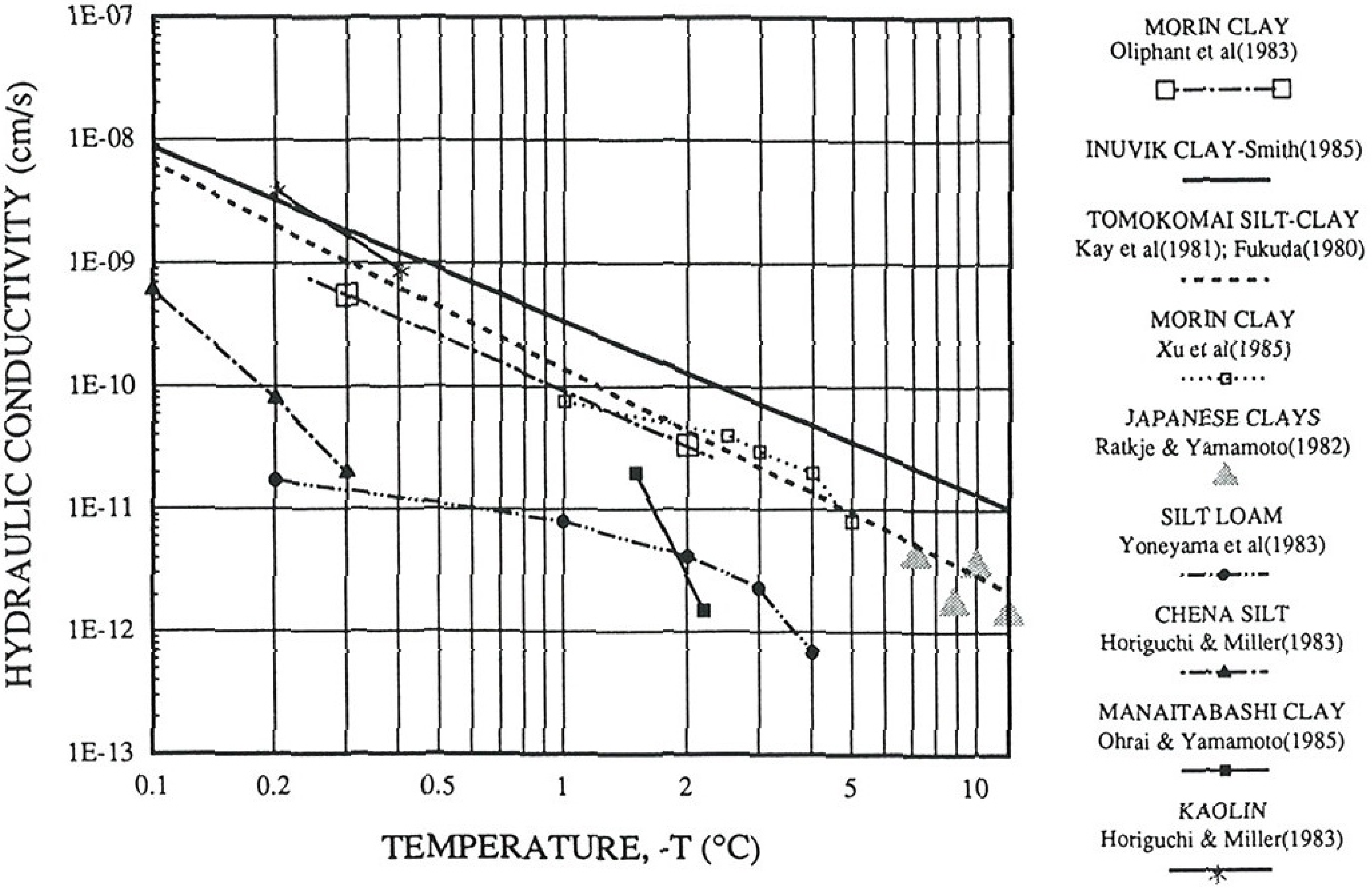
REVISION NO.:

PROJECT
Thermal Design of Tailings Dams
Doris North Project, NU

TITLE
Historical Air Temperature Record for Kugluktuk,
Cambridge Bay and Contwoyto Lake/Lupin (1959-2005)

DATE: September 2006

Figure 1



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Thermal Design of Tailing Dams
Doris North Project, NU

Hydraulic Conductivity of Frozen Fine-Grained Soils (After Nixon, 1991)

PROJECT NO./FILE NO.	DWN	CKD	REV
1100126 1100126J01.cdr	JLS	JS	1

OFFICE EBA-EDM

DATE September 1, 2006

Figure 2

NOTES

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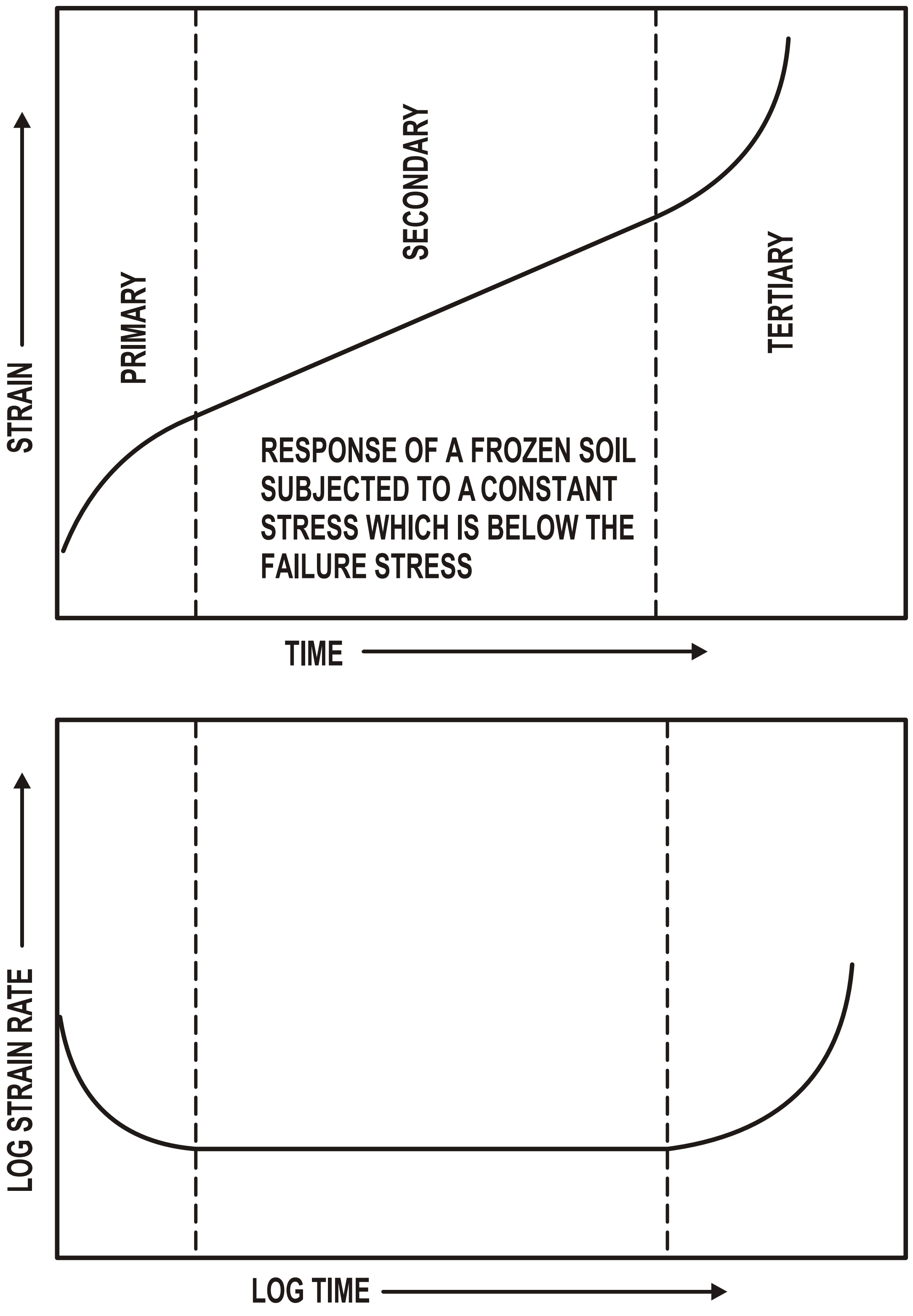

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Thermal Design of Tailings Dam
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Typical Creep Curve for Frozen Soil

PROJECT NO./FILE NO.	DWN	CKD	REV
1100126 1100126Q01a.cdr	DBD	JTCS	1

OFFICE	DATE
EBA-EDM	September 12, 2006

Figure 3

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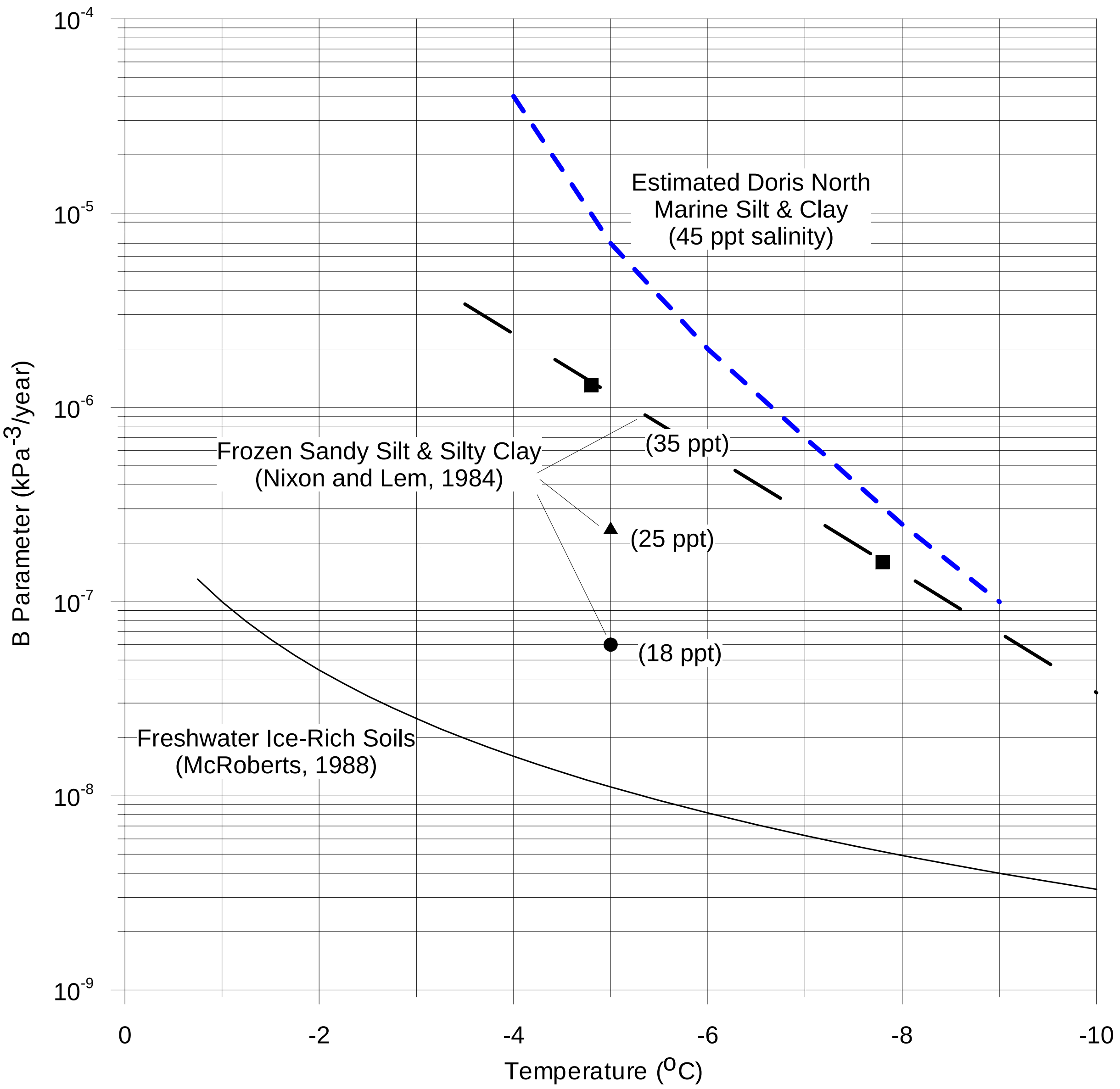

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Thermal Design of Tailings Dams
Doris North Project, NU

TITLE

Relationship of the Creep Parameter B with
Salinity and Temperature

DATE: September 2006

Figure 4

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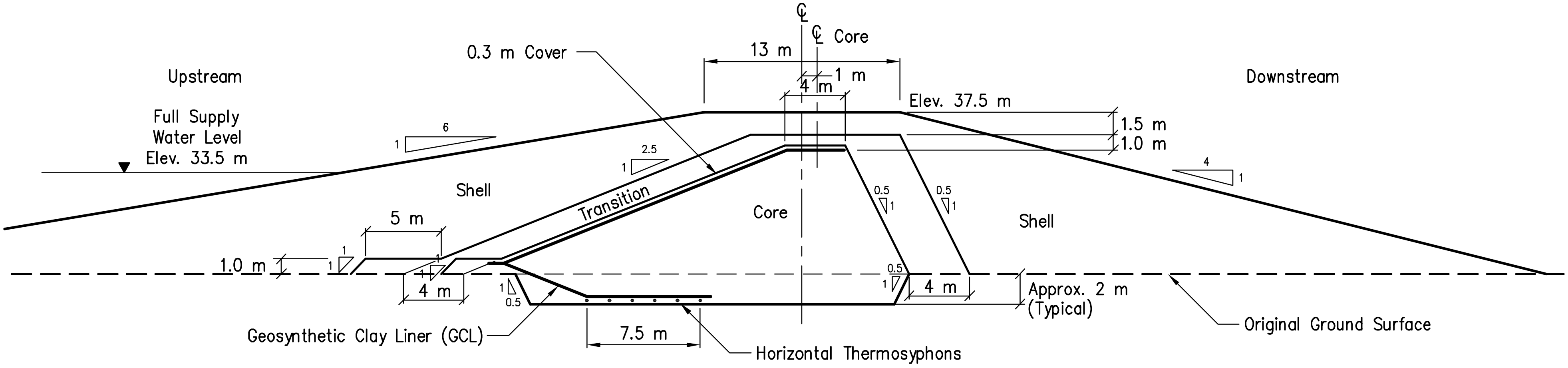
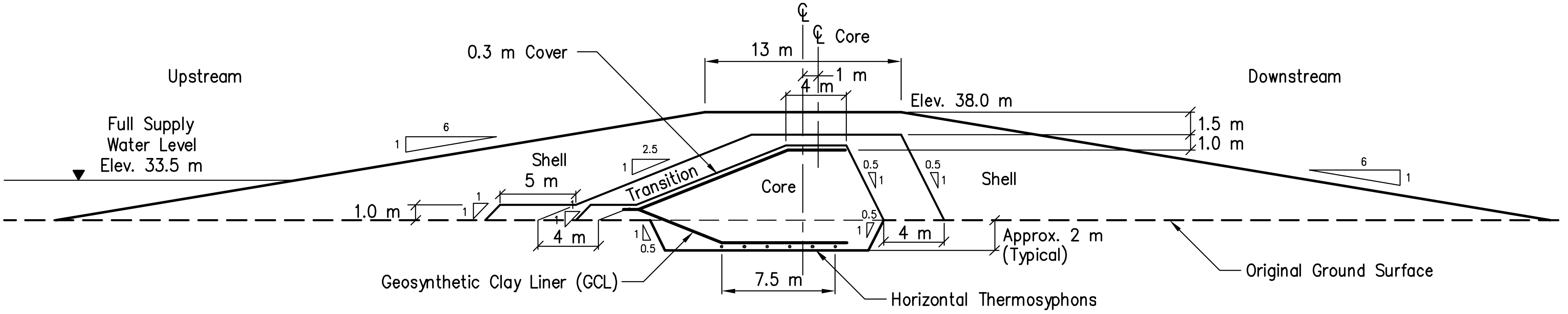


NORTH DAM



SOUTH DAM

0 20

Scale: 1: 400 (metres)

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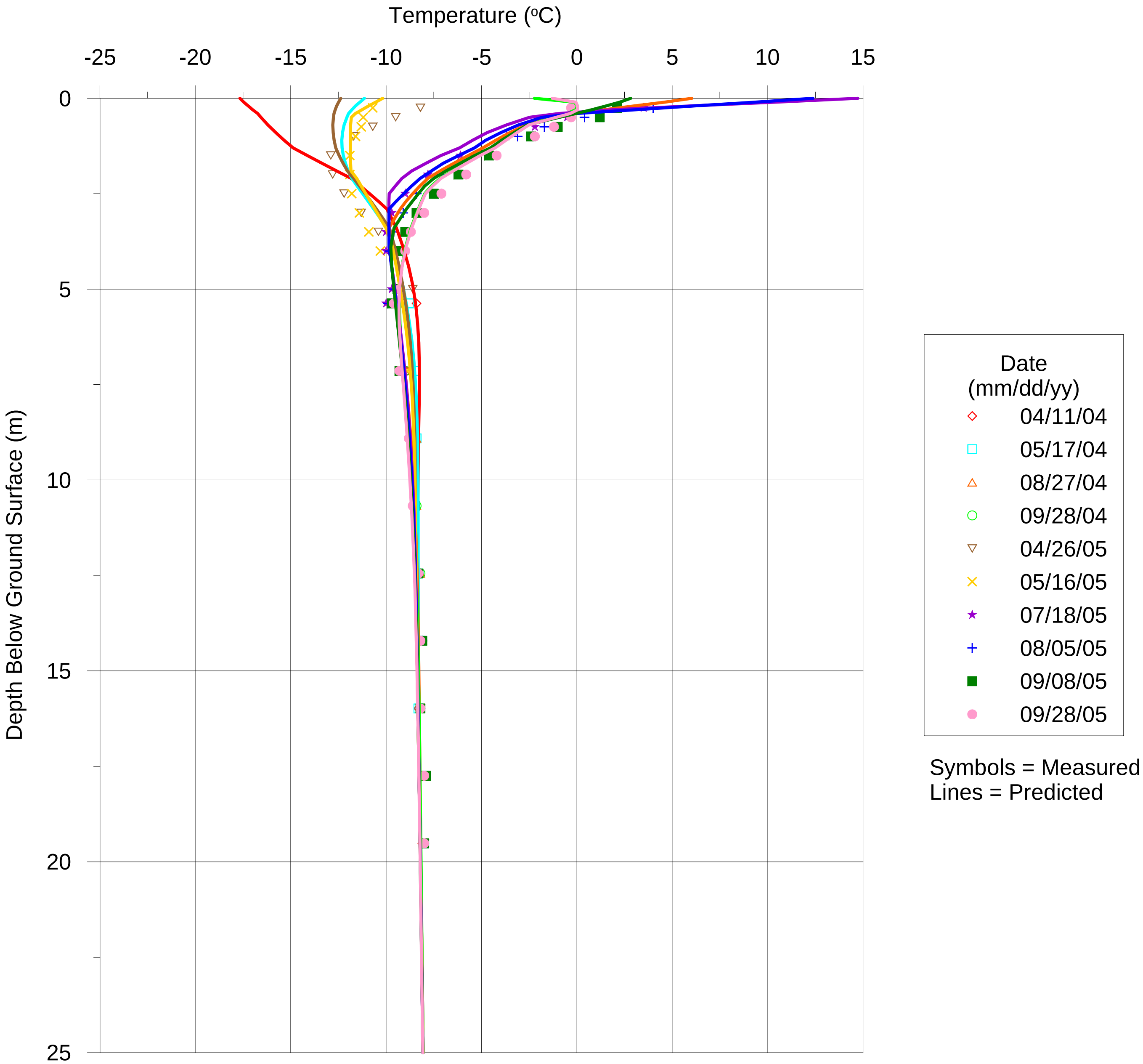
THERMAL DESIGN OF TAILING DAMS
DORIS NORTH PROJECT, NU

GENERALIZED DAM CROSS-SECTIONS

PROJECT NO.	DWN	CKD	REV
1100126	BR	JTCS	0
OFFICE	DATE		
EBA-EDMONTON	July 27, 2006		

Figure 5



Note: Measured Temperatures: Top 5 m based on BH SRK-51, below 5 m based on BH SRK-15

EBA Engineering Consultants Ltd.	CLIENT SRK Consulting *Engineers and Scientists*	PROJECT	Thermal Design of Tailings Dams Doris North Project, NU
DWN. JTCS	CHKD. JTCS	TITLE	Thermal Model Calibration, 2004/2005 North Dam Alignment, Boreholes SRK-15 & SRK-51
EBA JOB NO. 1100126	FILE: 1100126_SRK15_Cal.grf	REVISION NO.:	DATE: September 2006
			Figure 6

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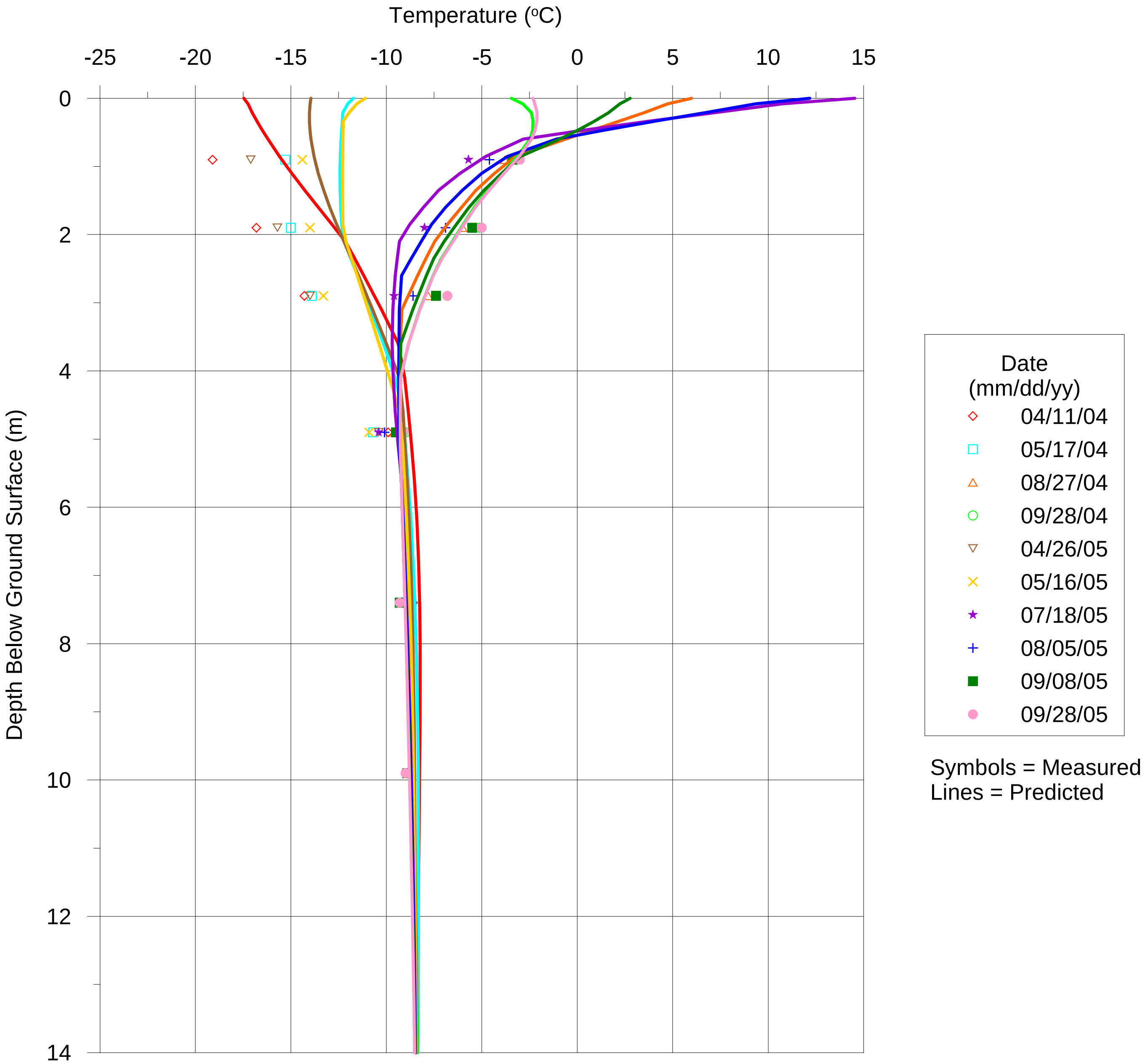

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PROJECT
Thermal Design of Tailings Dams
Doris North Project, NU

TITLE
Thermal Model Calibration, 2004/2005
South Dam Alignment, Boreholes SRK-33

DWN. JTCS	CHKD. JTCS		
EBA JOB NO. 1100126	FILE: 1100126_SRK33_Cal.grf	REVISION NO.:	DATE: September 2006

Figure 7

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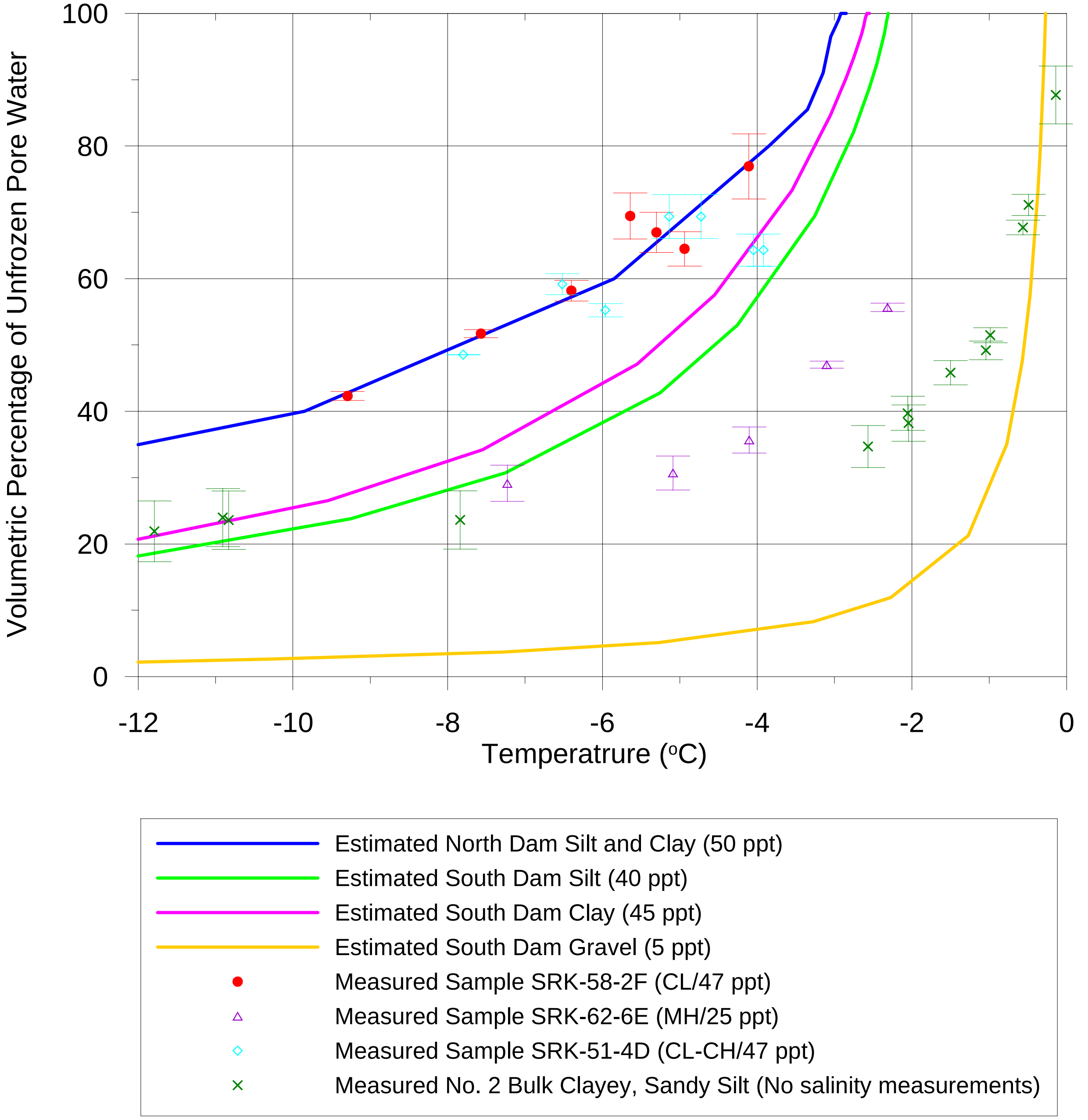

Note: Measured data reported in SRK's 2004 Summer and 2005 Winter Site Investigation reports.

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PROJECT
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Doris North Project, NU

TITLE
Unfrozen Water Content Curves
Used in Dam Design Thermal Analyses

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FILE: 1100126_UWC.grf

REVISION NO.:

DATE: September 2006

Figure 8

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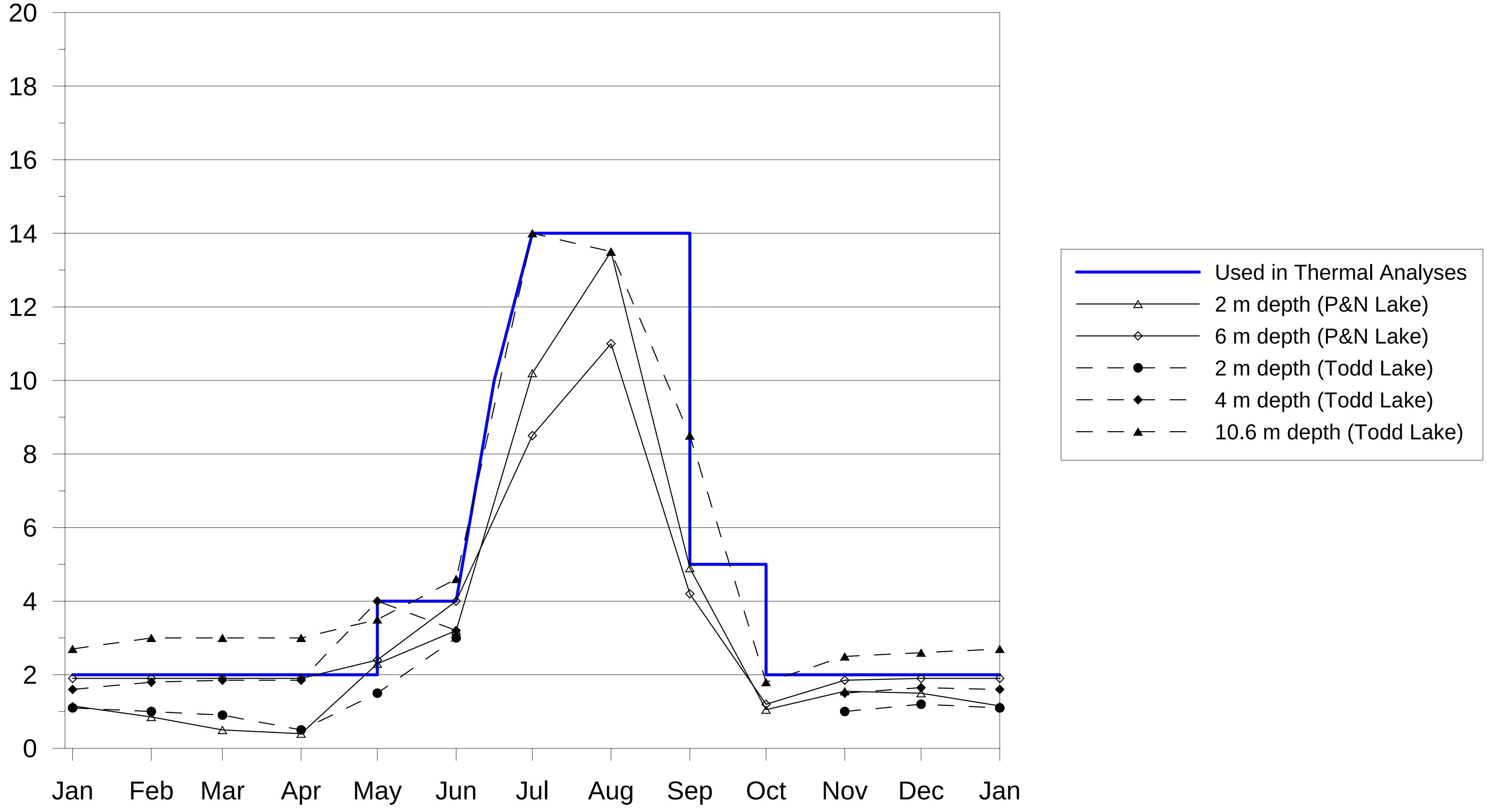

Notes:
1. P&N Lake data from Welch and Bergmann (1985)
2. Todd Lake data from Burn (2002)

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DWN. JTCS	CHKD. JTCS			TITLE	Lake Temperature Data Used in Thermal Analyses
EBA JOB NO. 1100126		FILE: 1100126wattemps.grf	REVISION NO.:	DATE: September 2006	Figure 9

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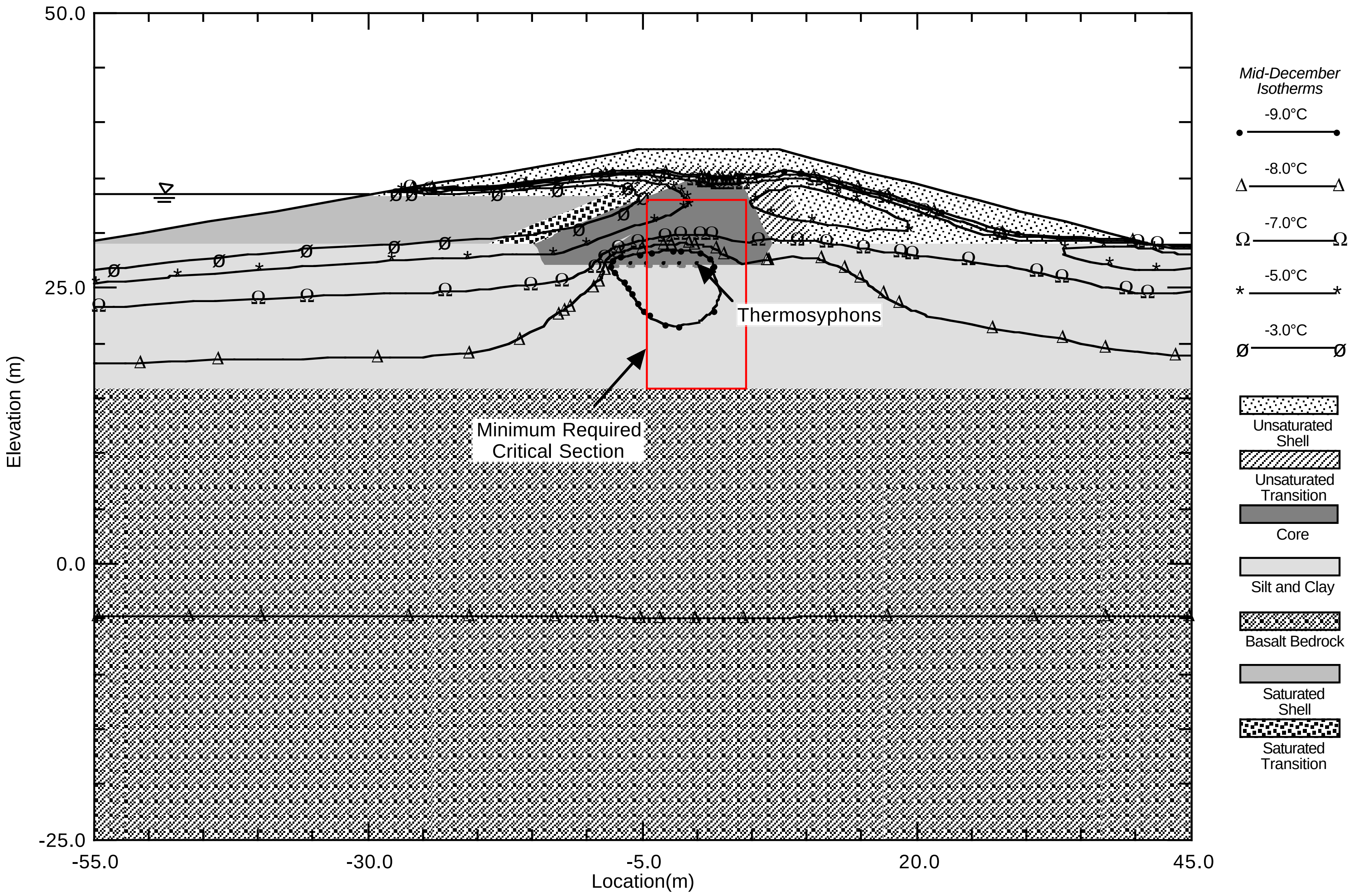

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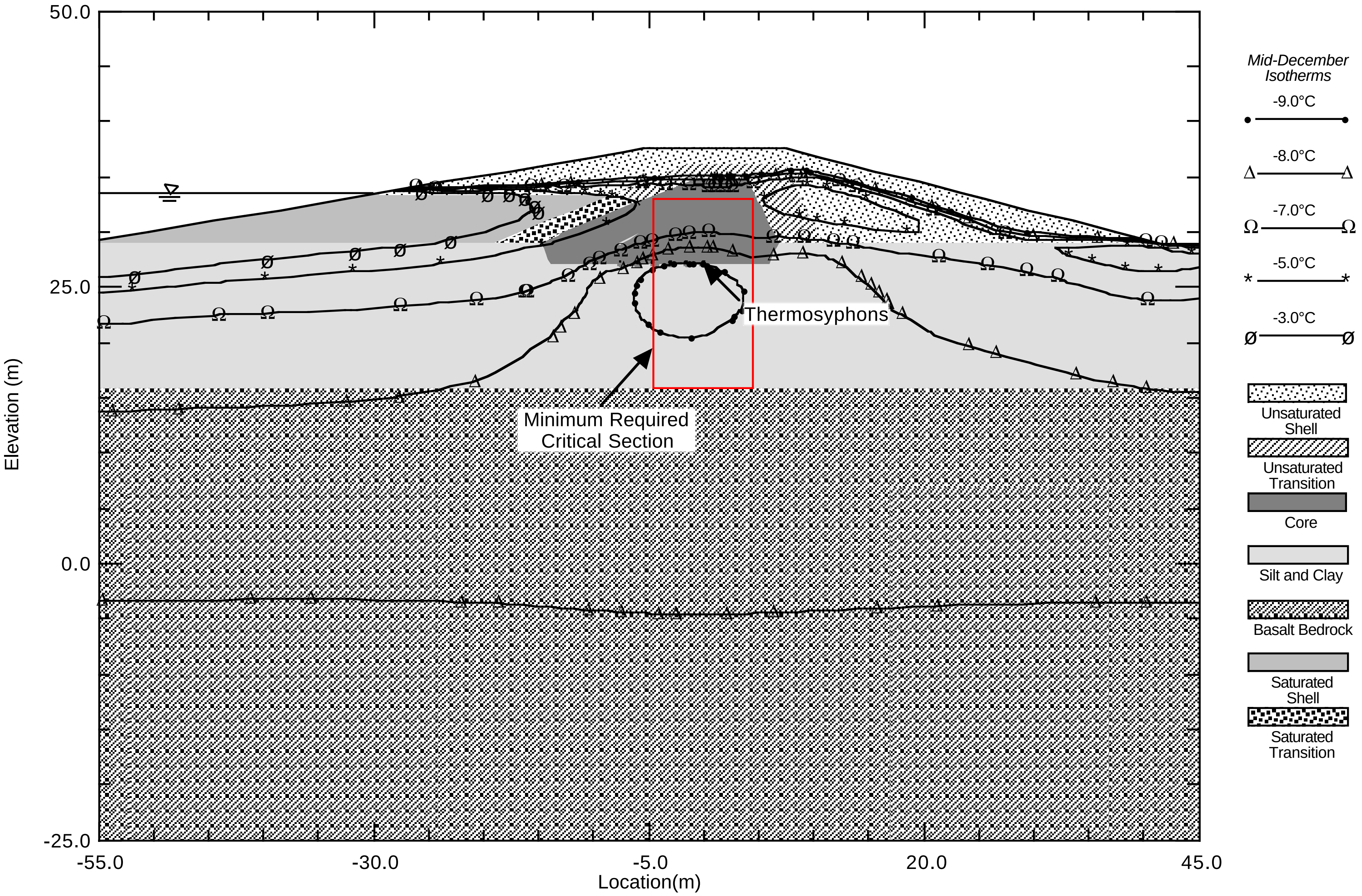
FILE: **1100126 Fig 10**
REVISION NO.:

PROJECT
Thermal Design of Tailings Dams
Doris North Project, NU

TITLE
Predicted Temperature Distribution One Year After Construction, North Dam

DATE: **September 2006**

Figure 10



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FILE: **1100126 Fig 11**

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PROJECT

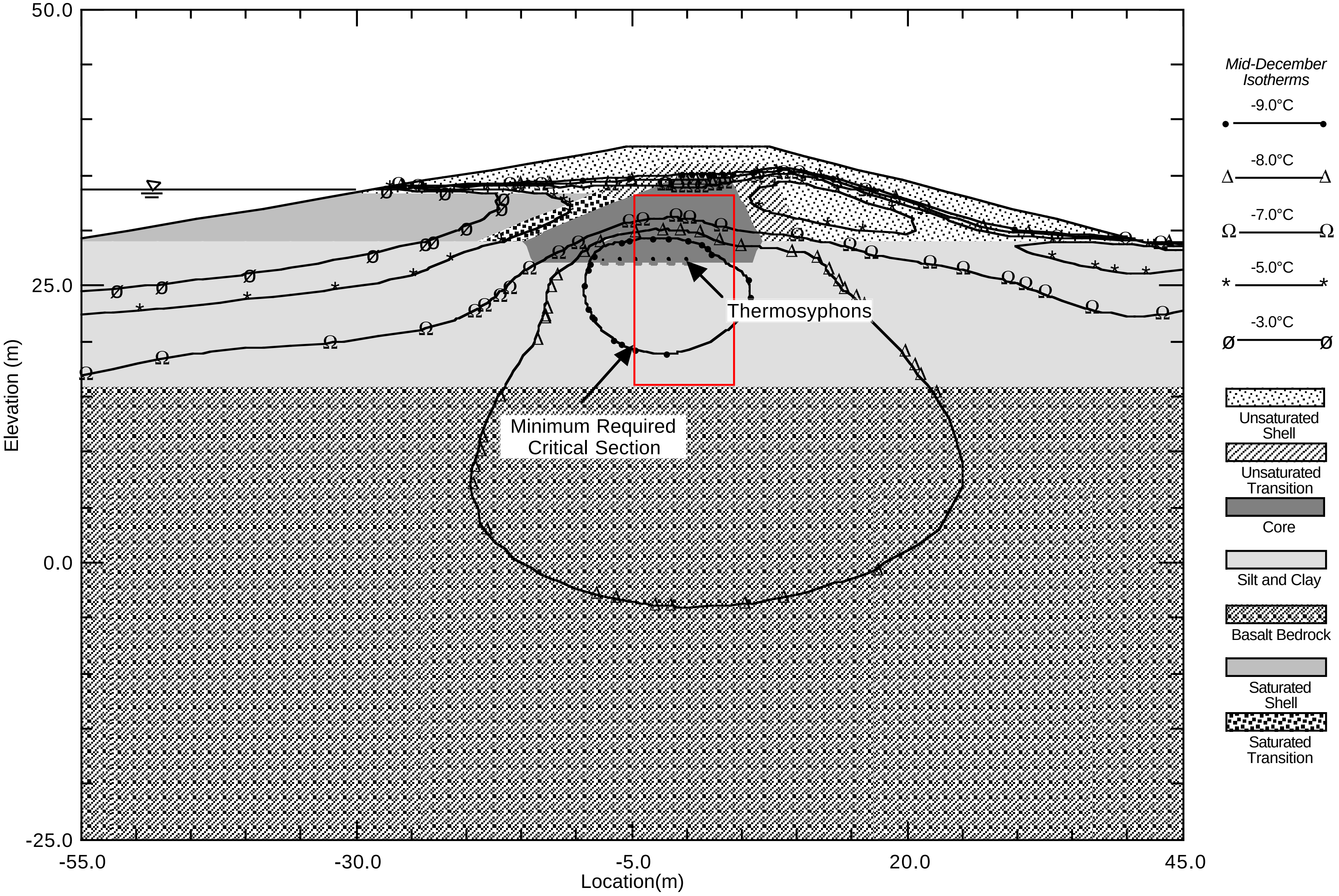
Thermal Design of Tailings Dams
Doris North Project, NU

TITLE

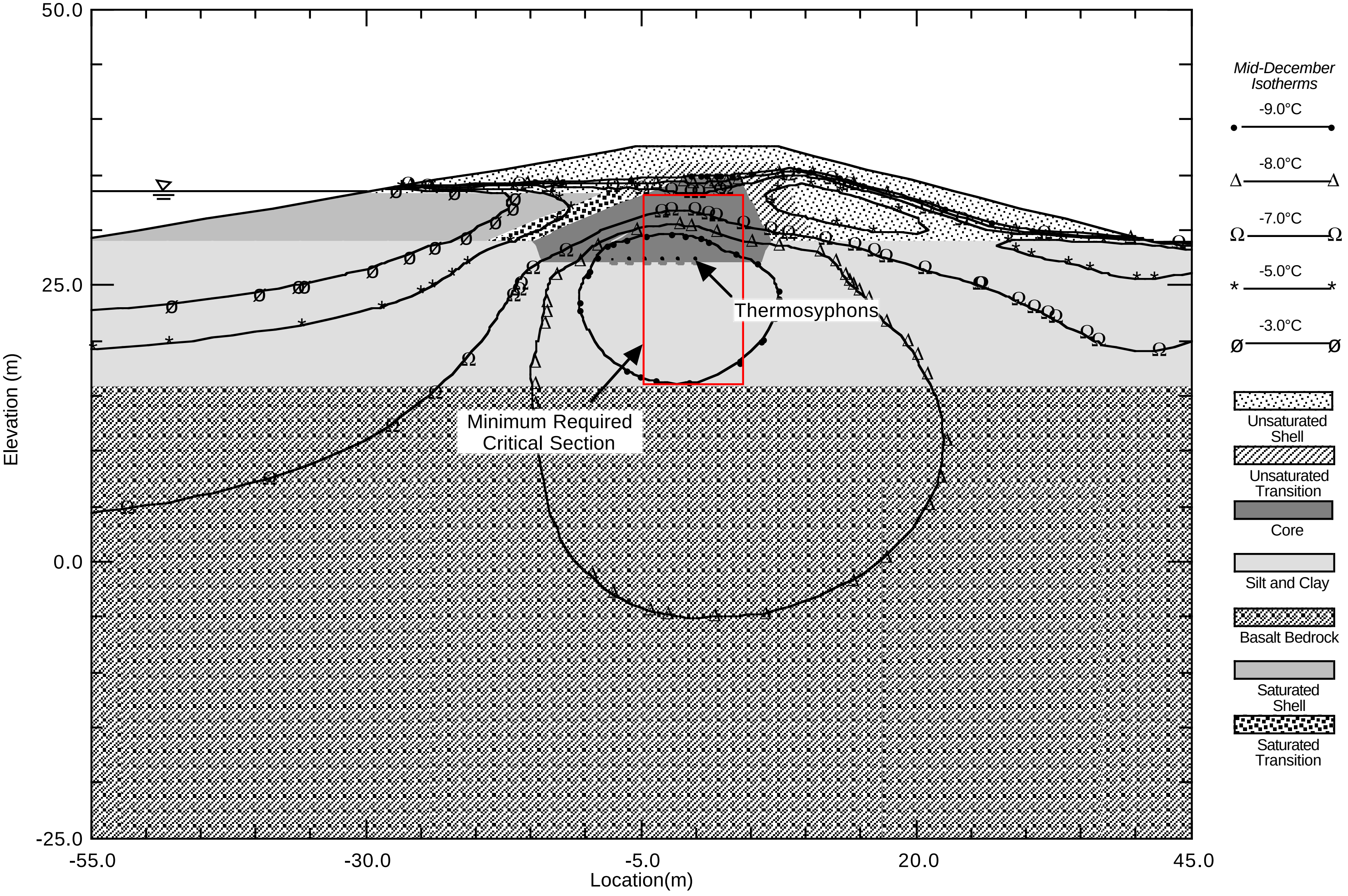
Predicted Temperature Distribution Two Years After Construction, North Dam

DATE: **September 2006**

Figure 11



EBA Engineering Consultants Ltd.	CLIENT SRK Consulting Engineers and Scientists		PROJECT **Thermal Design of Tailings Dams Doris North Project, NU**	
DWN.: **JTCS** CHKD.: **JTCS**			TITLE **Predicted Temperature Distribution Five Years After Construction, North Dam**	
EBA JOB NO.: **1100126**	FILE: **1100126 Fig 12**	REVISION NO.:	DATE: **September 2006**	**Figure 12**



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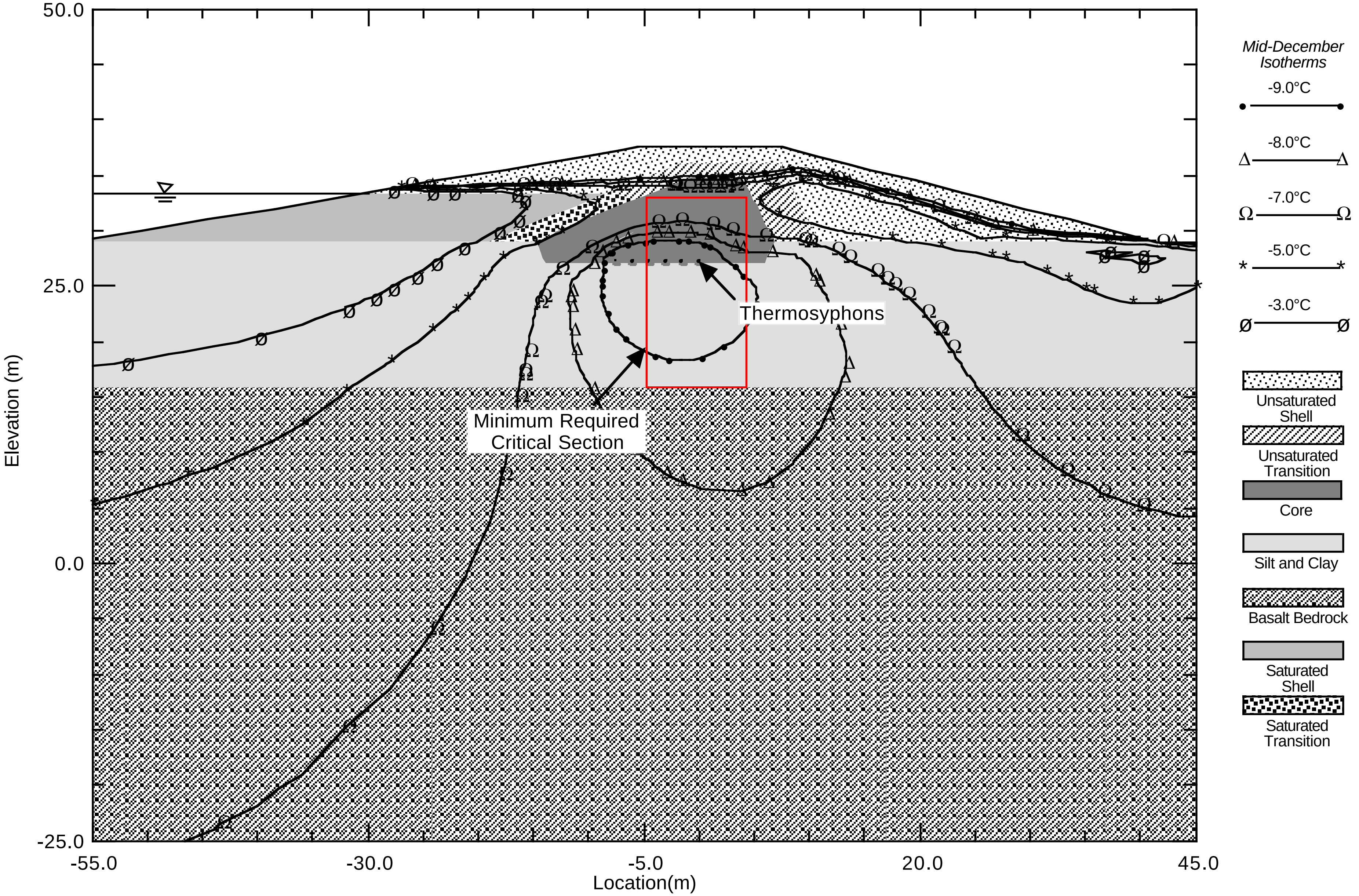
FILE: **1100126 Fig 13**
REVISION NO.:

PROJECT
Thermal Design of Tailings Dams
Doris North Project, NU

TITLE
Predicted Temperature Distribution Ten Years
After Construction, North Dam

DATE: **September 2006**

Figure 13



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CLIENT

SRK Consulting
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FILE: **1100126 Fig 14** REVISION NO.:

PROJECT

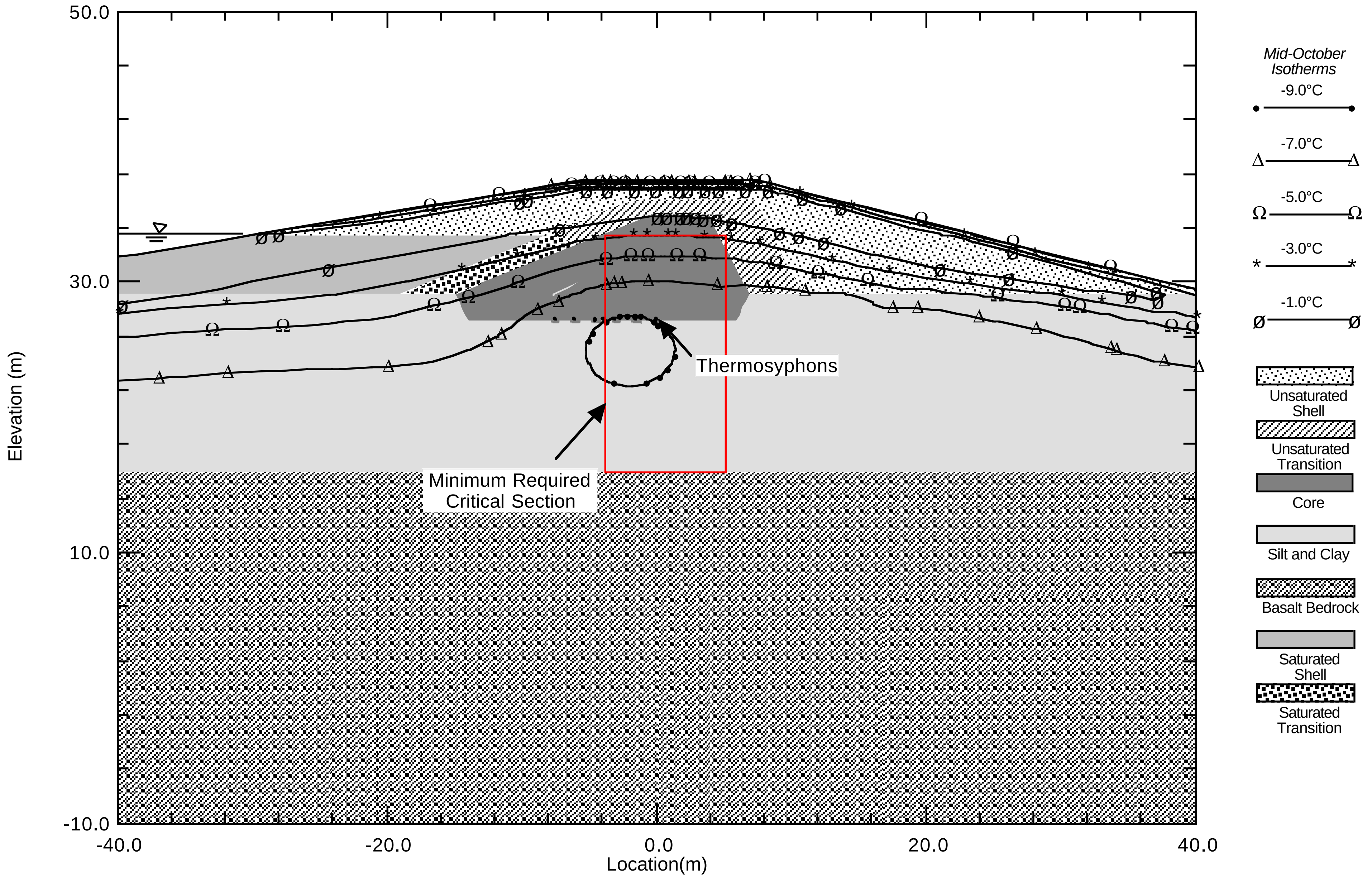
Thermal Design of Tailings Dams
Doris North Project, NU

TITLE

Predicted Temperature Distribution Twenty-Five Years After Construction, North Dam

DATE: **September 2006**

Figure 14



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EBA JOB NO.: **1100126**

CLIENT

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FILE: **1100126 Fig 15** REVISION NO.:

PROJECT

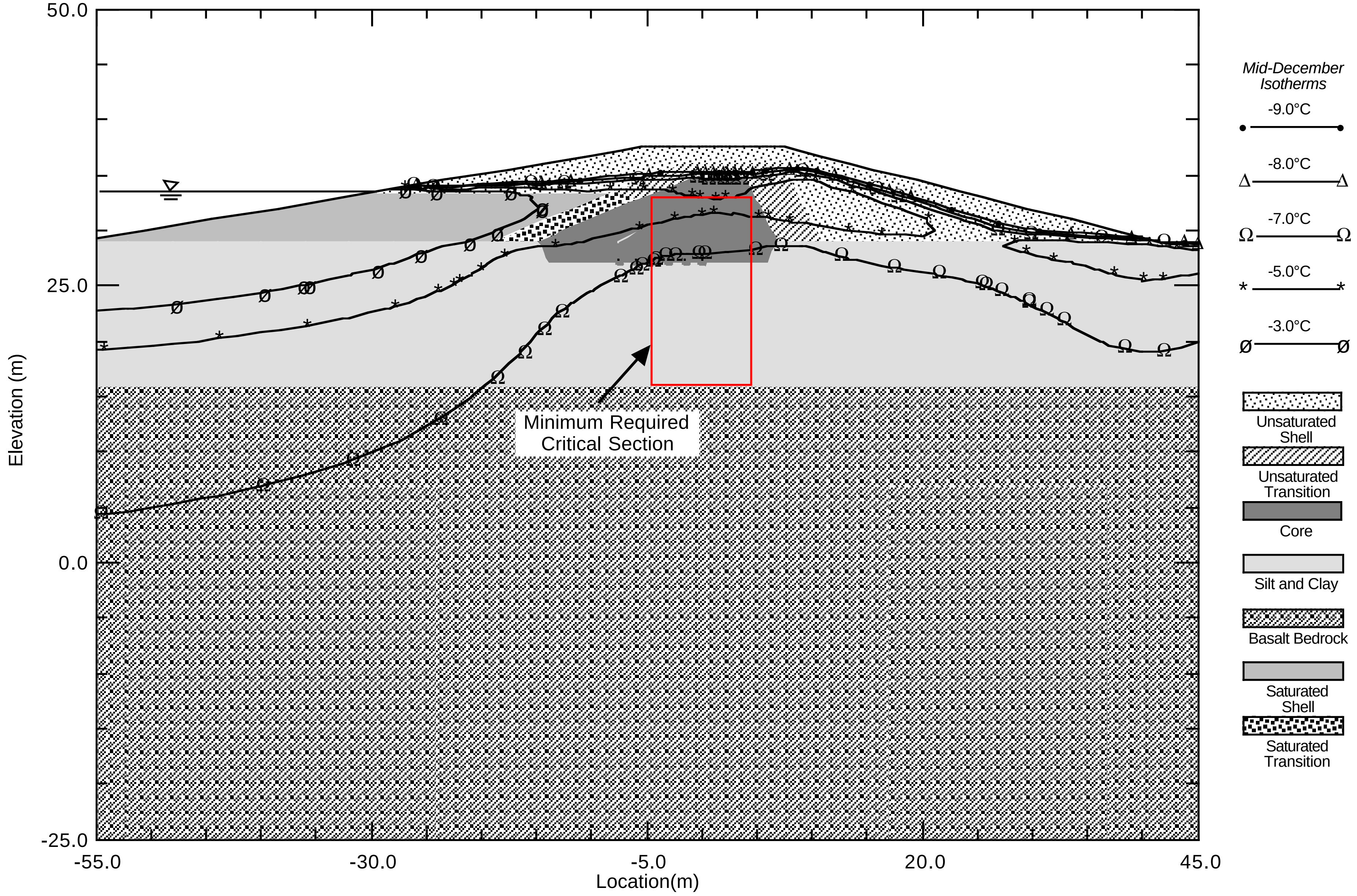
Thermal Design of Tailings Dams
Doris North Project, NU

TITLE

Predicted Temperature Distribution Two Extreme Warm Years After Construction, North Dam

DATE: **September 2006**

Figure 15



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FILE: **1100126 Fig 16** REVISION NO.:

PROJECT

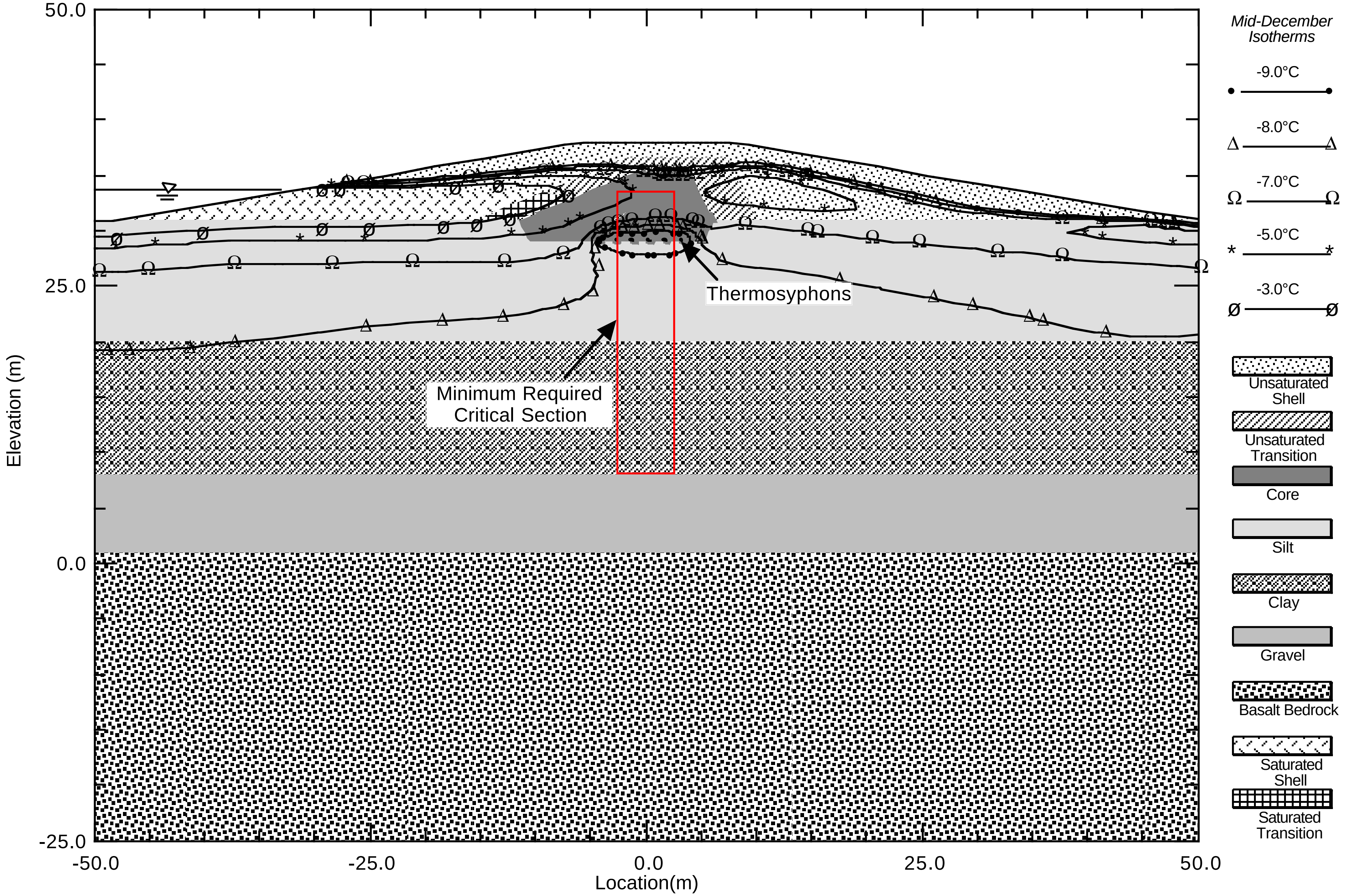
Thermal Design of Tailings Dams
Doris North Project, NU

TITLE

Predicted Temperature Distribution Ten Years After Construction, No Thermosyphons, North Dam

DATE: **September 2006**

Figure 16



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EBA JOB NO.: **1100126**

CLIENT

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FILE: **1100126 Fig 17**

REVISION NO.:

PROJECT

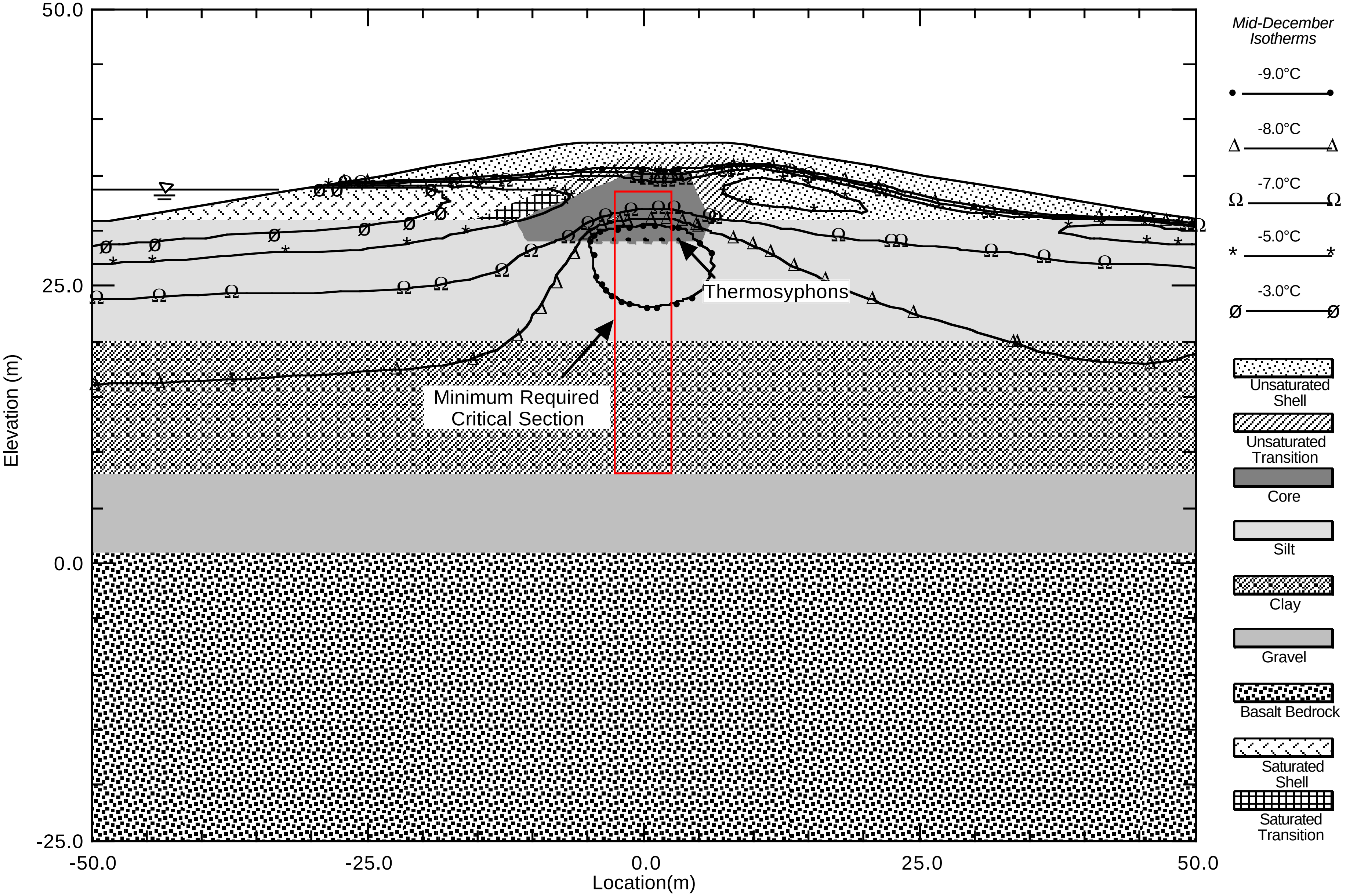
Thermal Design of Tailings Dams
Doris North Project, NU

TITLE

Predicted Temperature Distribution One Year After Construction, South Dam

DATE: **September 2006**

Figure 17



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CLIENT SRK Consulting Engineers and Scientists

PROJECT **Thermal Design of Tailings Dams**
Doris North Project, NU

TITLE **Predicted Temperature Distribution Two Years After Construction, South Dam**

DWN.: **JTCS** CHKD.: **JTCS**

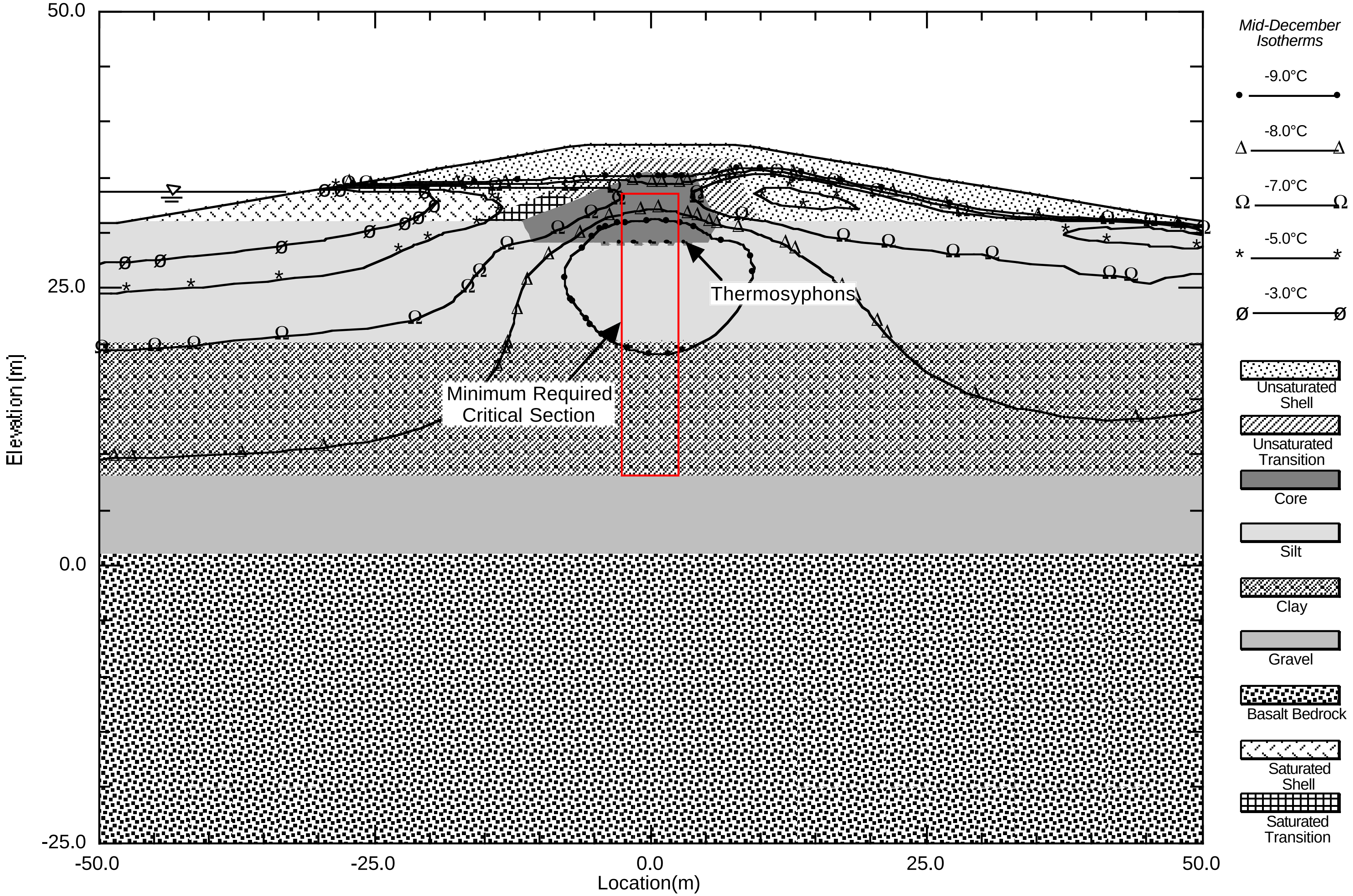
EBA JOB NO.: **1100126**

FILE: **1100126 Fig 18**

REVISION NO.:

DATE: **September 2006**

Figure 18



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EBA JOB NO.: **1100126**

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Engineers and Scientists

FILE: **1100126 Fig 19**

REVISION NO.:

PROJECT

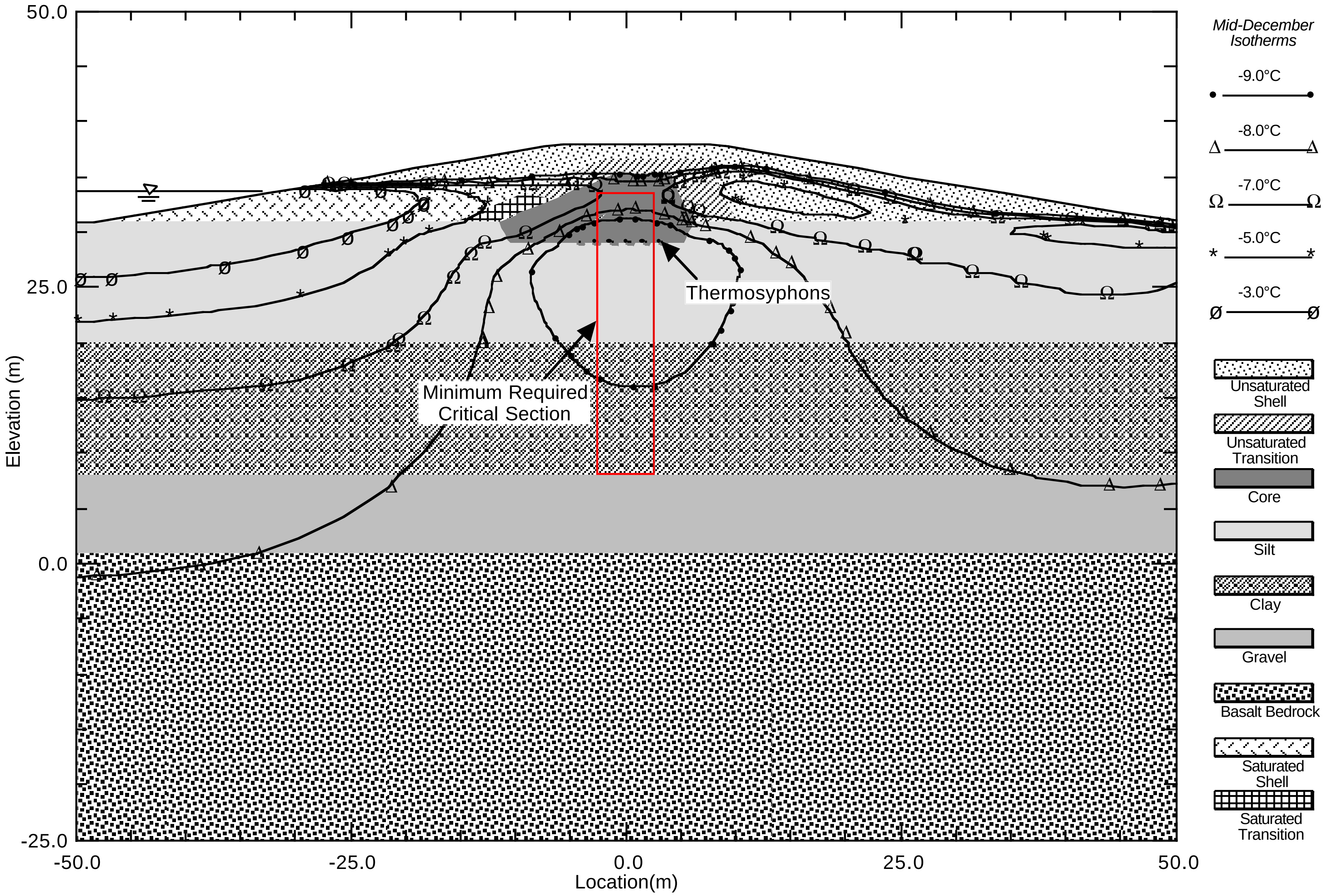
Thermal Design of Tailings Dams
Doris North Project, NU

TITLE

Predicted Temperature Distribution Five Years After Construction, South Dam

DATE: **September 2006**

Figure 19



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FILE: **1100126 Fig 20**

REVISION NO.:

PROJECT

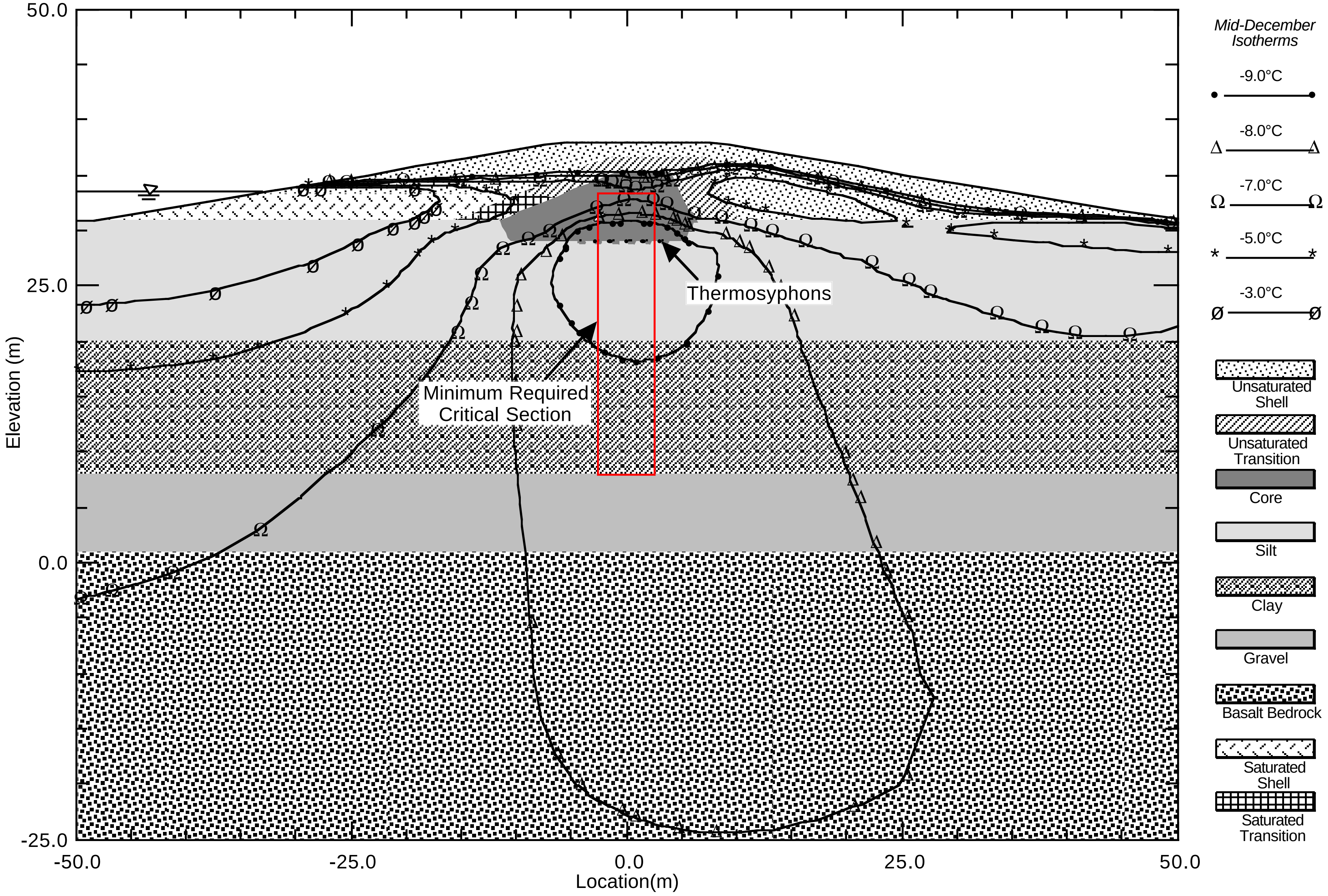
Thermal Design of Tailings Dams
Doris North Project, NU

TITLE

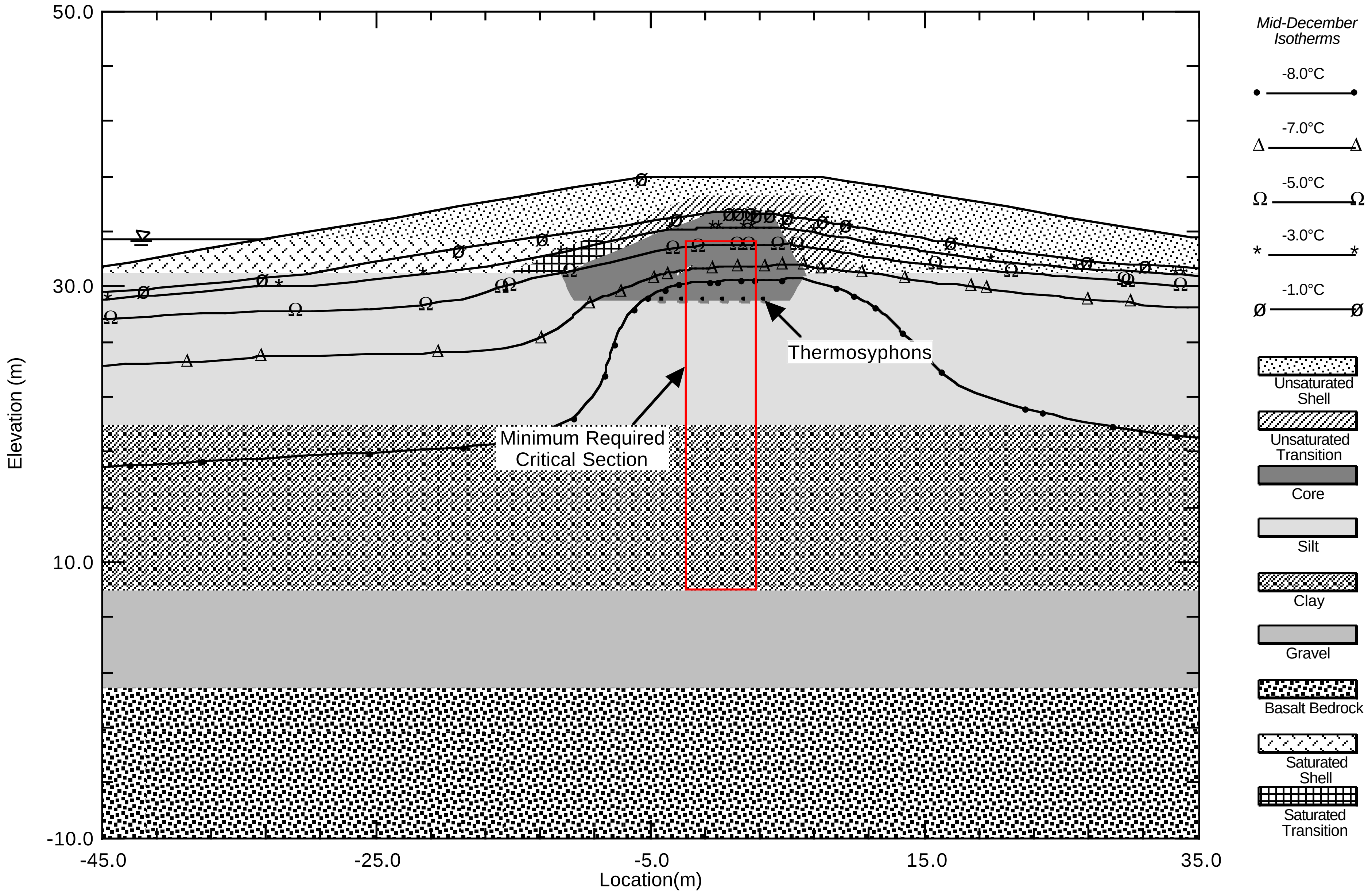
Predicted Temperature Distribution Ten Years After Construction, South Dam

DATE: **September 2006**

Figure 20



EBA Engineering Consultants Ltd.	CLIENT SRK Consulting Engineers and Scientists	PROJECT	**Thermal Design of Tailings Dams Doris North Project, NU**
DWN.: **JTCS** CHKD.: **JTCS**		TITLE	**Predicted Temperature Distribution Twenty-Five Years After Construction, South Dam**
EBA JOB NO.: **1100126**	FILE: **1100126 Fig 21** REVISION NO.:	DATE: **September 2006**	**Figure 21**



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FILE: **1100126 Fig 22** REVISION NO.:

PROJECT

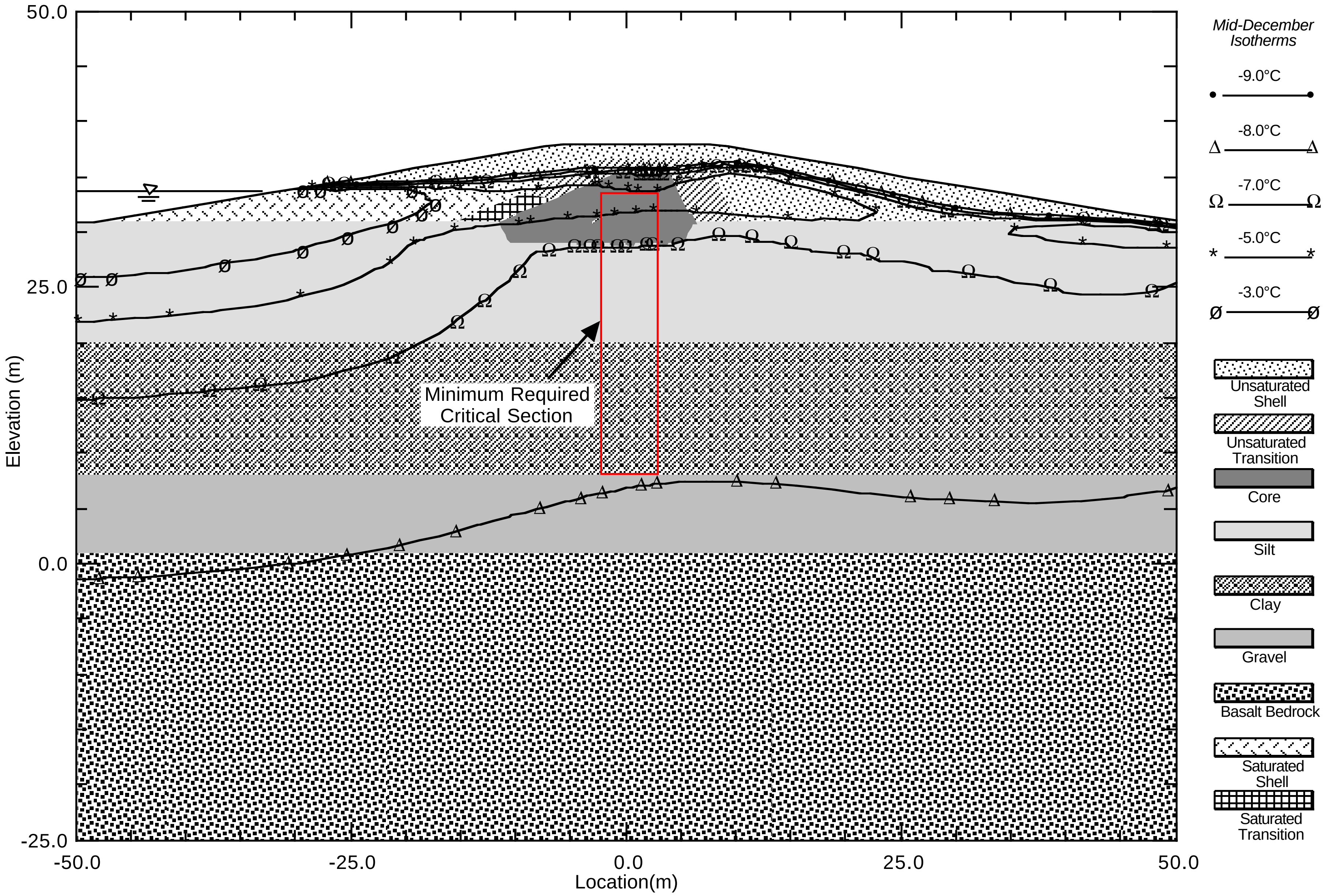
Thermal Design of Tailings Dams
Doris North Project, NU

TITLE

Predicted Temperature Distribution Two Extreme Warm Years After Construction, South Dam

DATE: **September 2006**

Figure 22



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EBA JOB NO.: **1100126**

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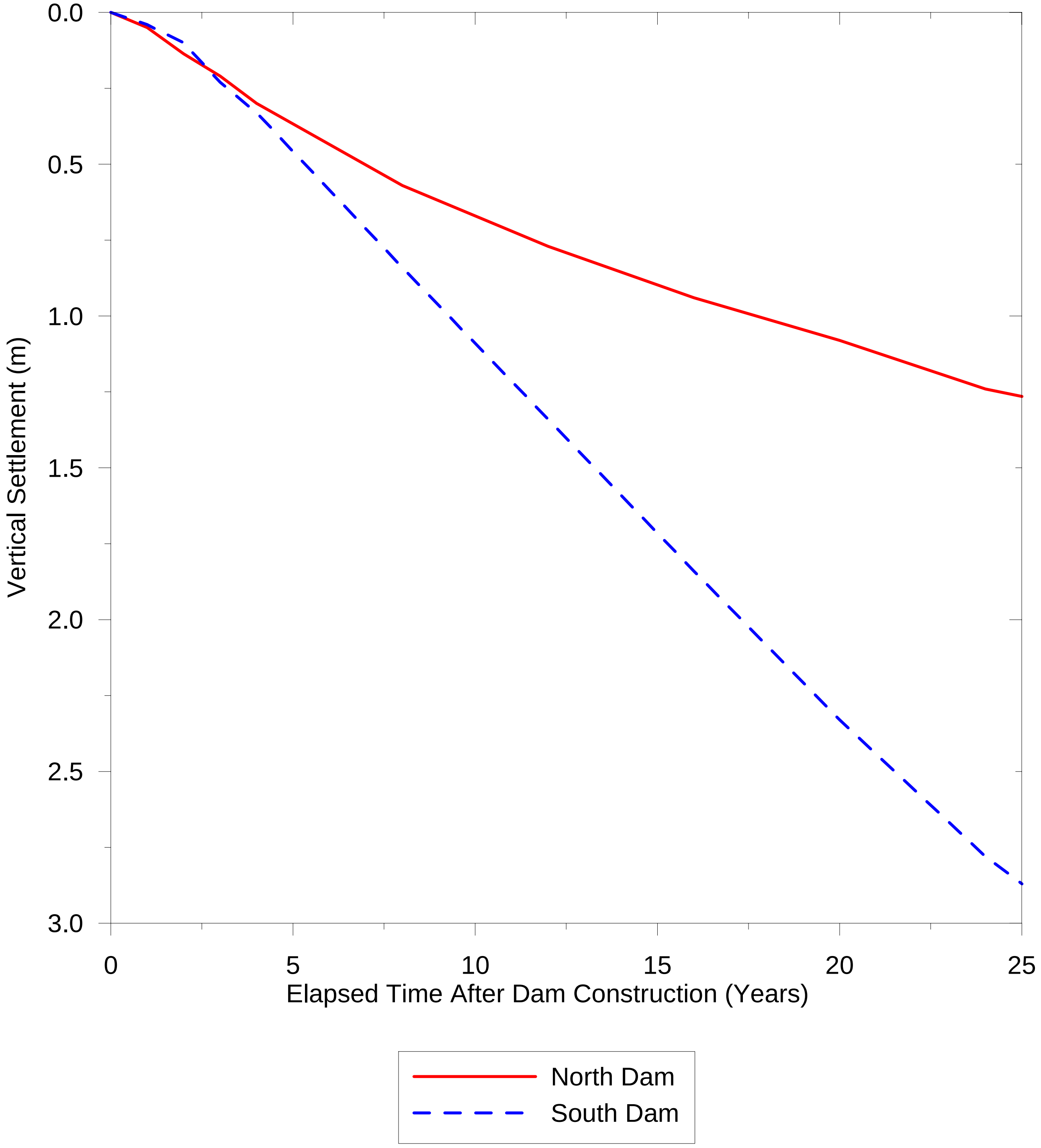
FILE: **1100126 Fig 23**
REVISION NO.:

PROJECT
Thermal Design of Tailings Dams
Doris North Project, NU

TITLE
Predicted Temperature Distribution Ten Years After Construction, No Thermosyphons, South Dam

DATE: **September 2006**

Figure 23



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FILE: 1100126_fig24.grf

REVISION NO.:

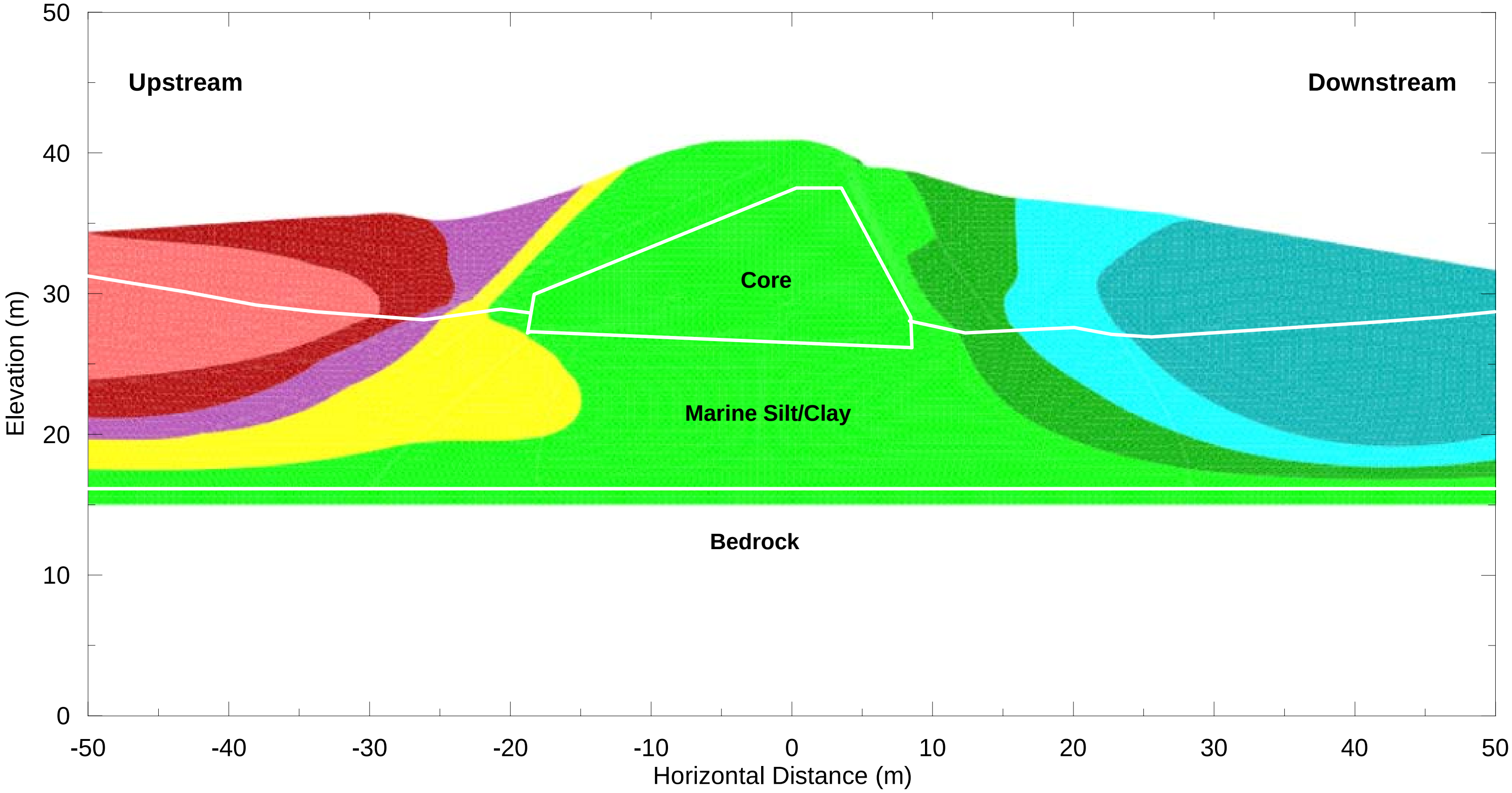
PROJECT
Thermal Design of Tailings Dams
Doris North Project, NU

TITLE
Predicted Settlement History of Core Crest
Doris North Dams

DATE: September 2006

Figure 24

16:25 09/22/06



Scale
Lateral Displacement
(m)

🟥 -4
🟥 -3
🟪 -2
🟨 -1
🟩 1
🟩 2
🟦 3
🟦
🟦

(zero contour omitted)

Note: Negative Displacement Upstream; Positive Displacement Downstream

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FILE: 1100126fig25.grf
REVISION NO.:

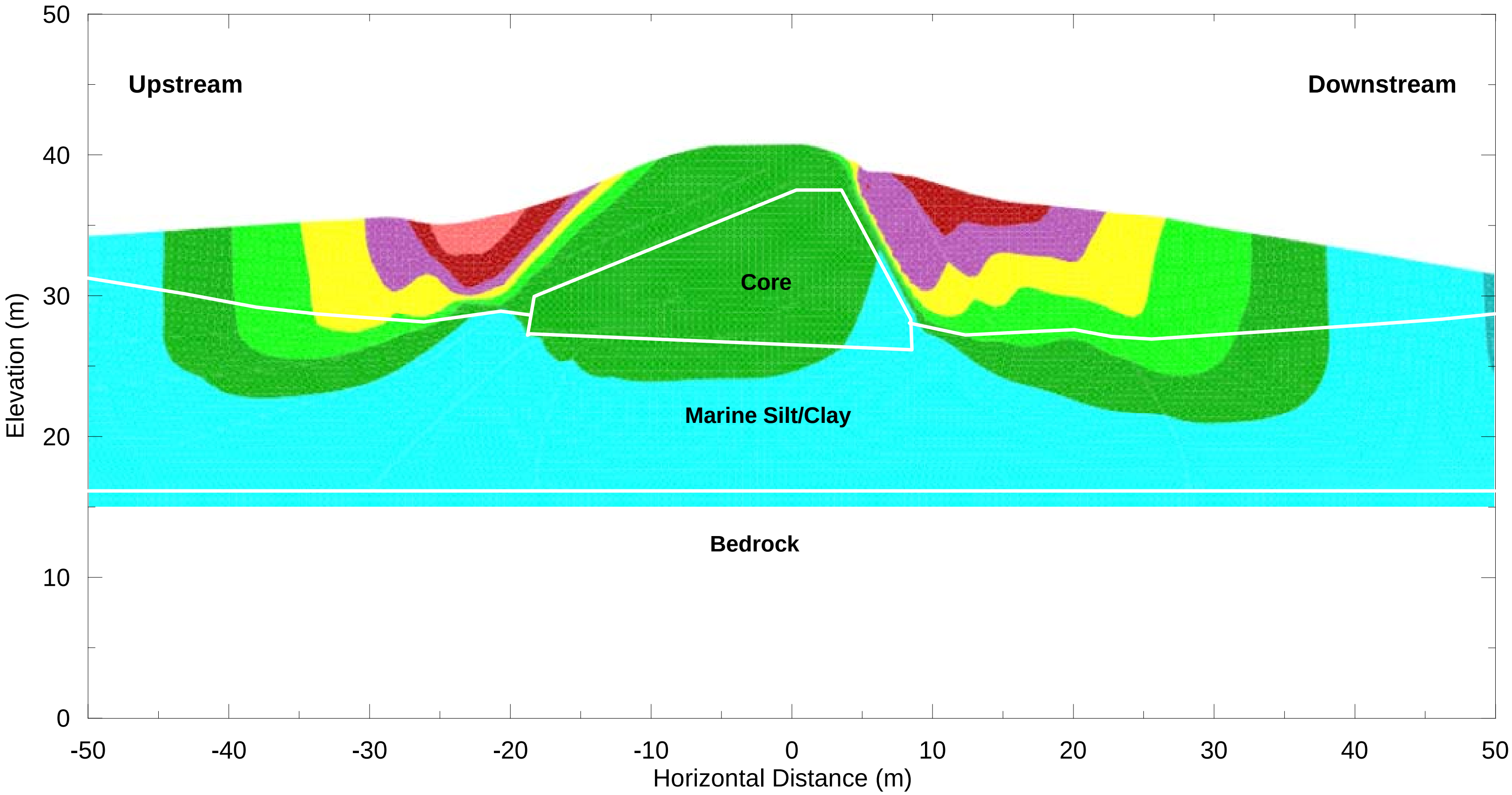
PROJECT
Thermal Design of Tailings Dams
Doris North Project, NU

TITLE
Predicted Lateral Creep Deformations
Ten Years After Dam Construction, North Dam

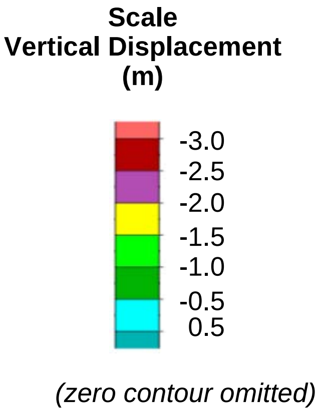
DATE: September 2006

Figure 25

15:42 09/22/06



Note: Negative Displacement Downward; Positive Displacement Upward



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EBA JOB NO. 1100126

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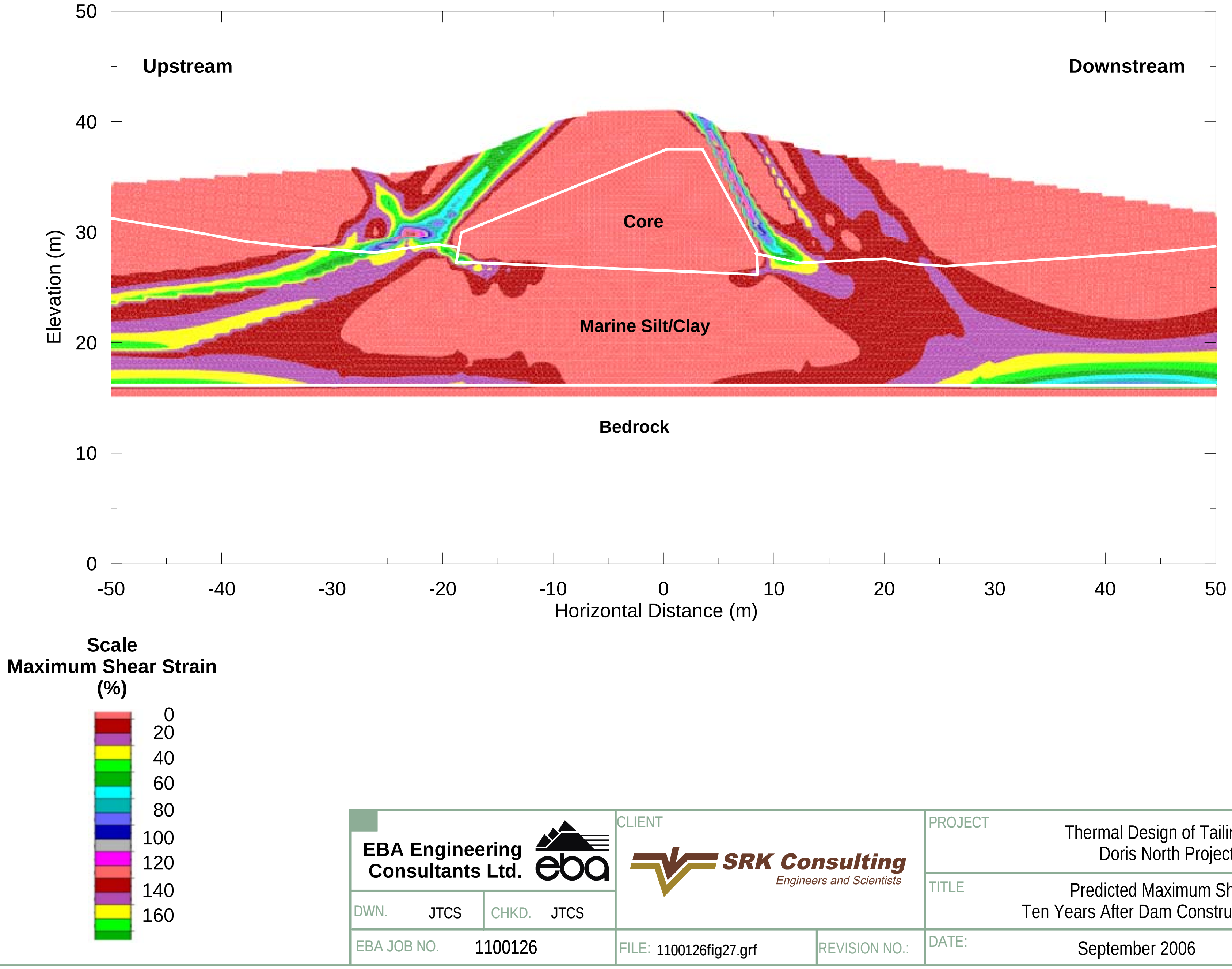
FILE: 1100126fig26.grf
REVISION NO.:

PROJECT
Thermal Design of Tailings Dams
Doris North Project, NU

TITLE
Predicted Vertical Creep Deformations
Ten Years After Dam Construction, North Dam

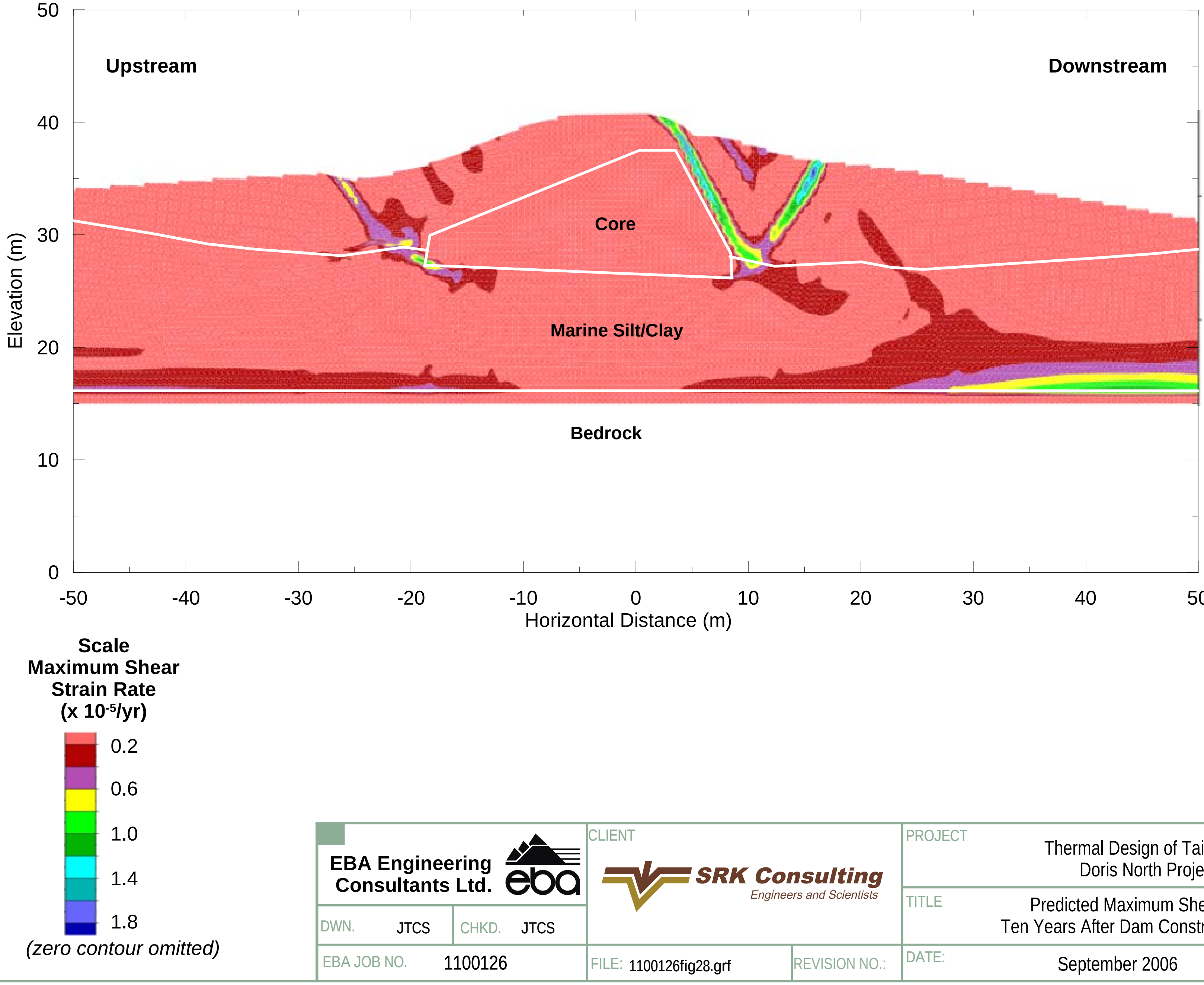
DATE: September 2006
Figure 26

15:45 09/22/06



EBA Engineering Consultants Ltd.	CLIENT SRK Consulting *Engineers and Scientists*	PROJECT	Thermal Design of Tailings Dams Doris North Project, NU
DWN. JTCS · CHKD. JTCS		TITLE	Predicted Maximum Shear Strain Ten Years After Dam Construction, North Dam
EBA JOB NO. 1100126	FILE: 1100126fig27.grf · REVISION NO.:	DATE: September 2006	Figure 27

15:43 09/22/06



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FILE: 1100126fig28.grf

REVISION NO.:

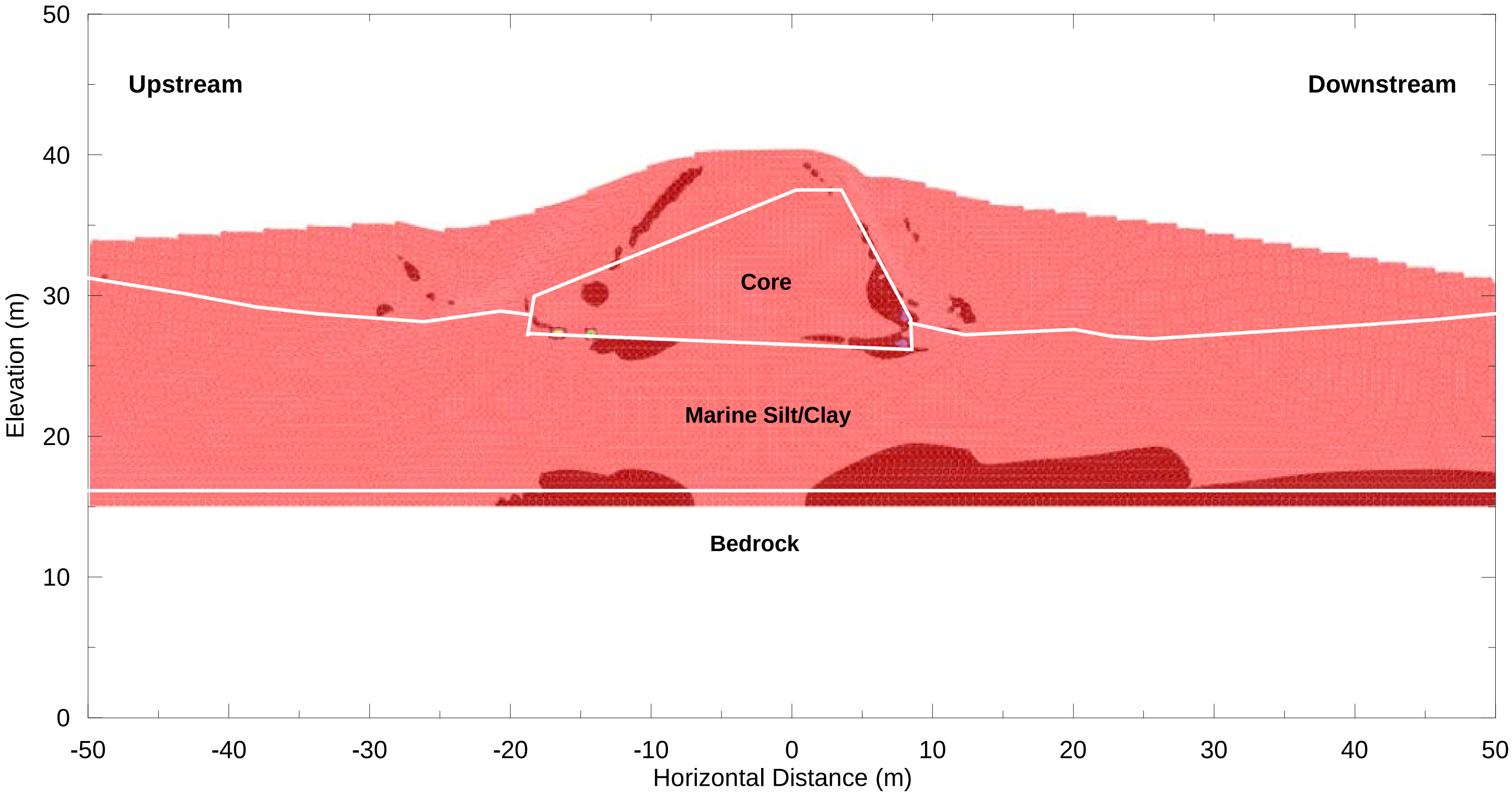
PROJECT
Thermal Design of Tailings Dams
Doris North Project, NU

TITLE
Predicted Maximum Shear Strain Rate
Ten Years After Dam Construction, North Dam

DATE: September 2006

Figure 28

15:48 09/22/06



Scale
Shear Stress
(kPa)

20
60
100
140
180

(zero contour omitted)

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FILE: 1100126fig29.grf

REVISION NO.:

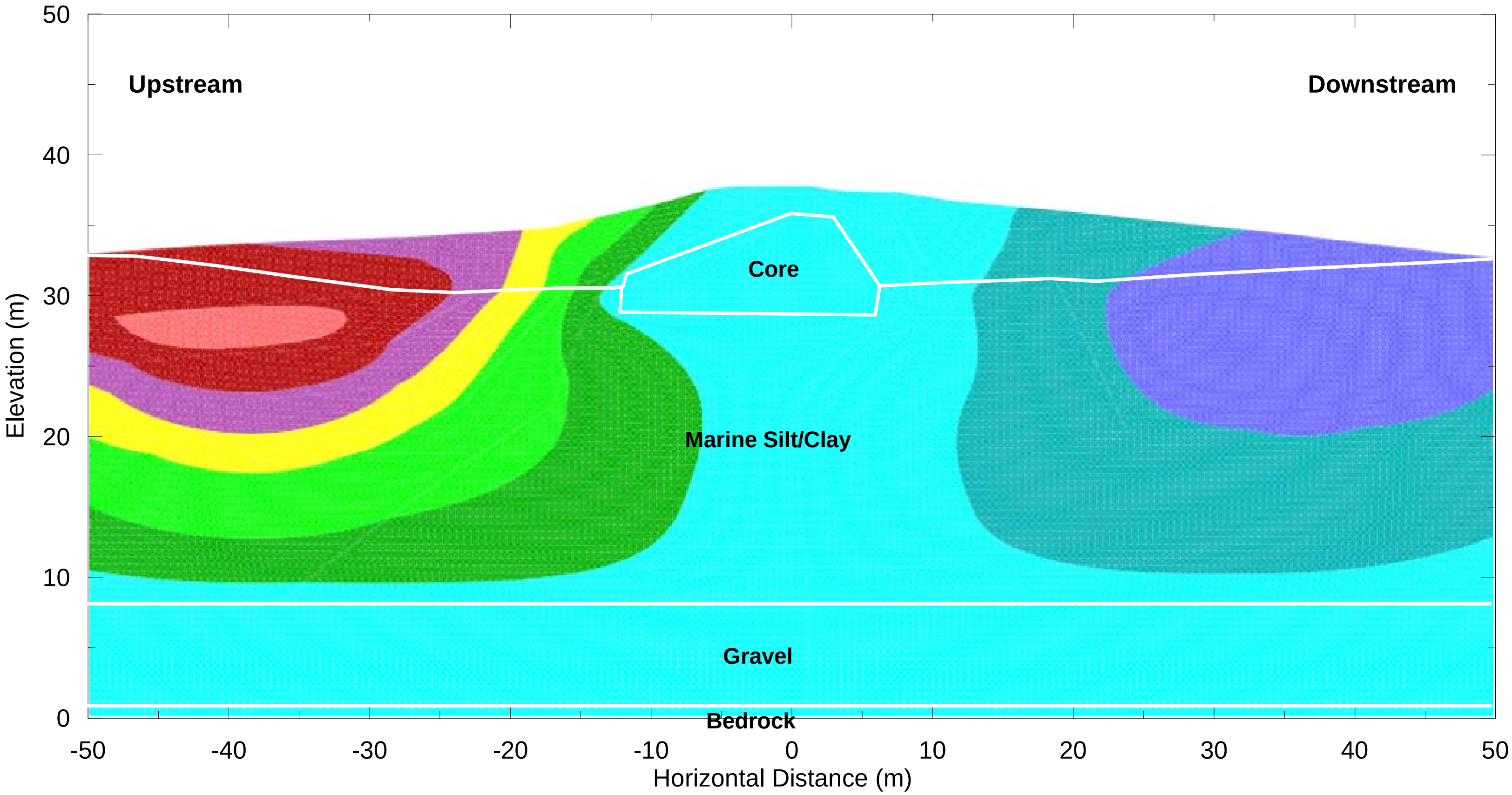
PROJECT
Thermal Design of Tailings Dams
Doris North Project, NU

TITLE
Predicted Shear Stress Distribution
Ten Years After Dam Construction, North Dam

DATE: September 2006

Figure 29

15:46 09/22/06



(zero contour omitted)

Note: Negative Displacement Upstream; Positive Displacement Downstream

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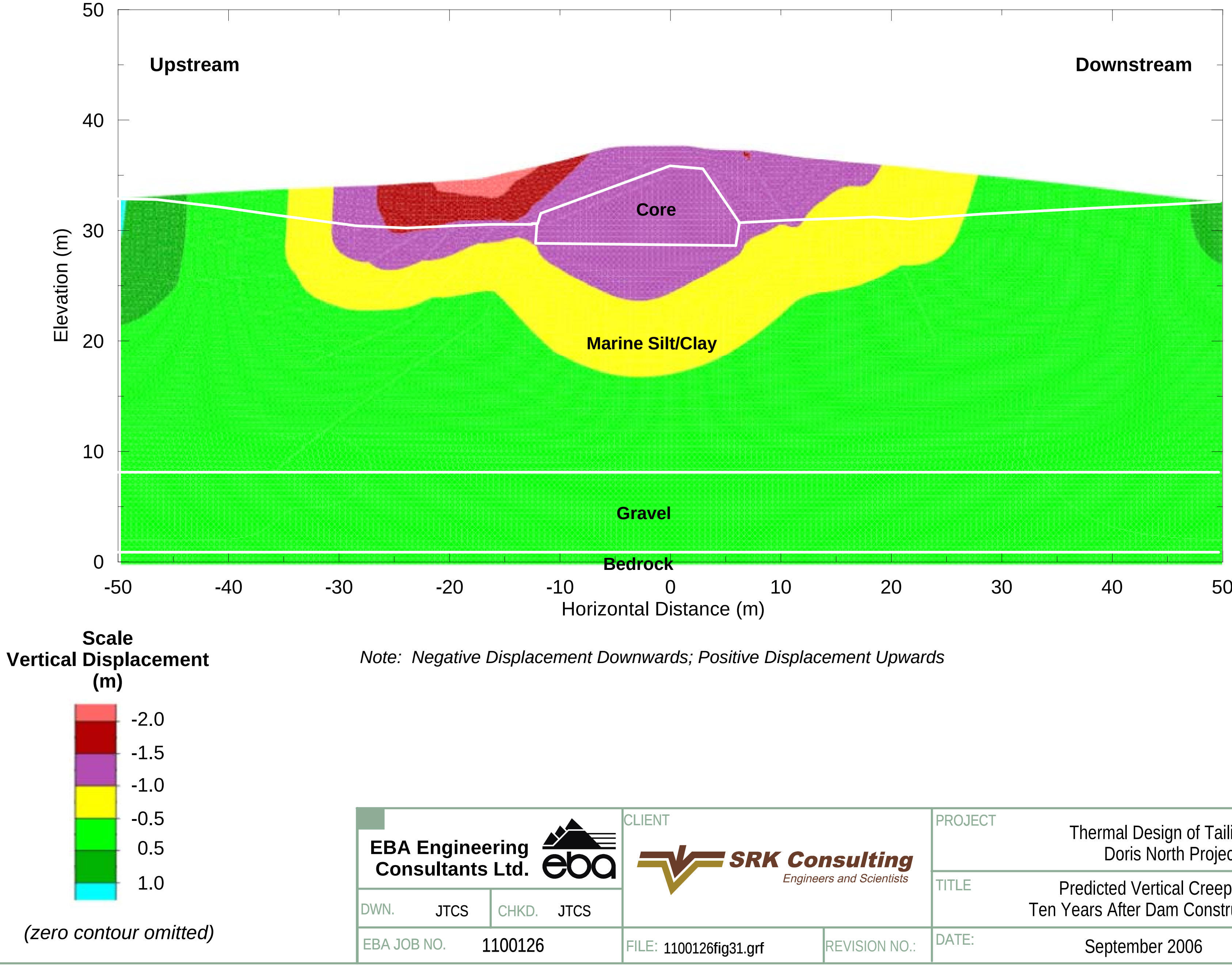
DWN. JTCS	CHKD. JTCS	TITLE	Predicted Lateral Creep Deformations Ten Years After Dam Construction, South Dam
EBA JOB NO. 1100126	FILE: 1100126fig30.grf	REVISION NO.:	DATE: September 2006
			Figure 30

15:54 09/22/06



Note: Negative Displacement Downwards; Positive Displacement Upwards

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FILE: 1100126fig31.grf

REVISION NO.:

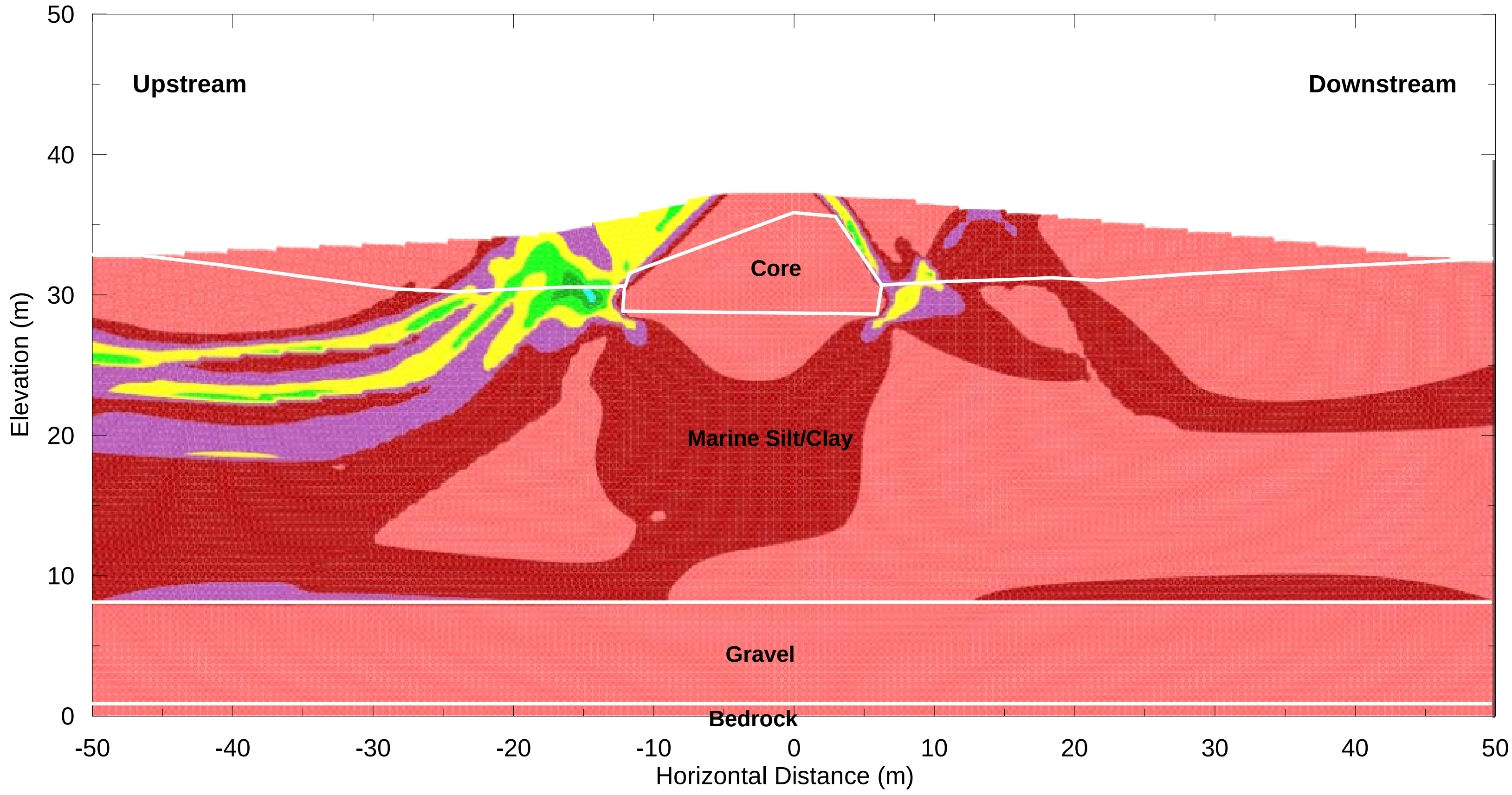
PROJECT
Thermal Design of Tailings Dams
Doris North Project, NU

TITLE
Predicted Vertical Creep Deformations
Ten Years After Dam Construction, South Dam

DATE: September 2006

Figure 31

15:49 09/22/06



Scale
Maximum Shear Strain
(%)

5
10
20
30
40
50

(zero contour omitted)

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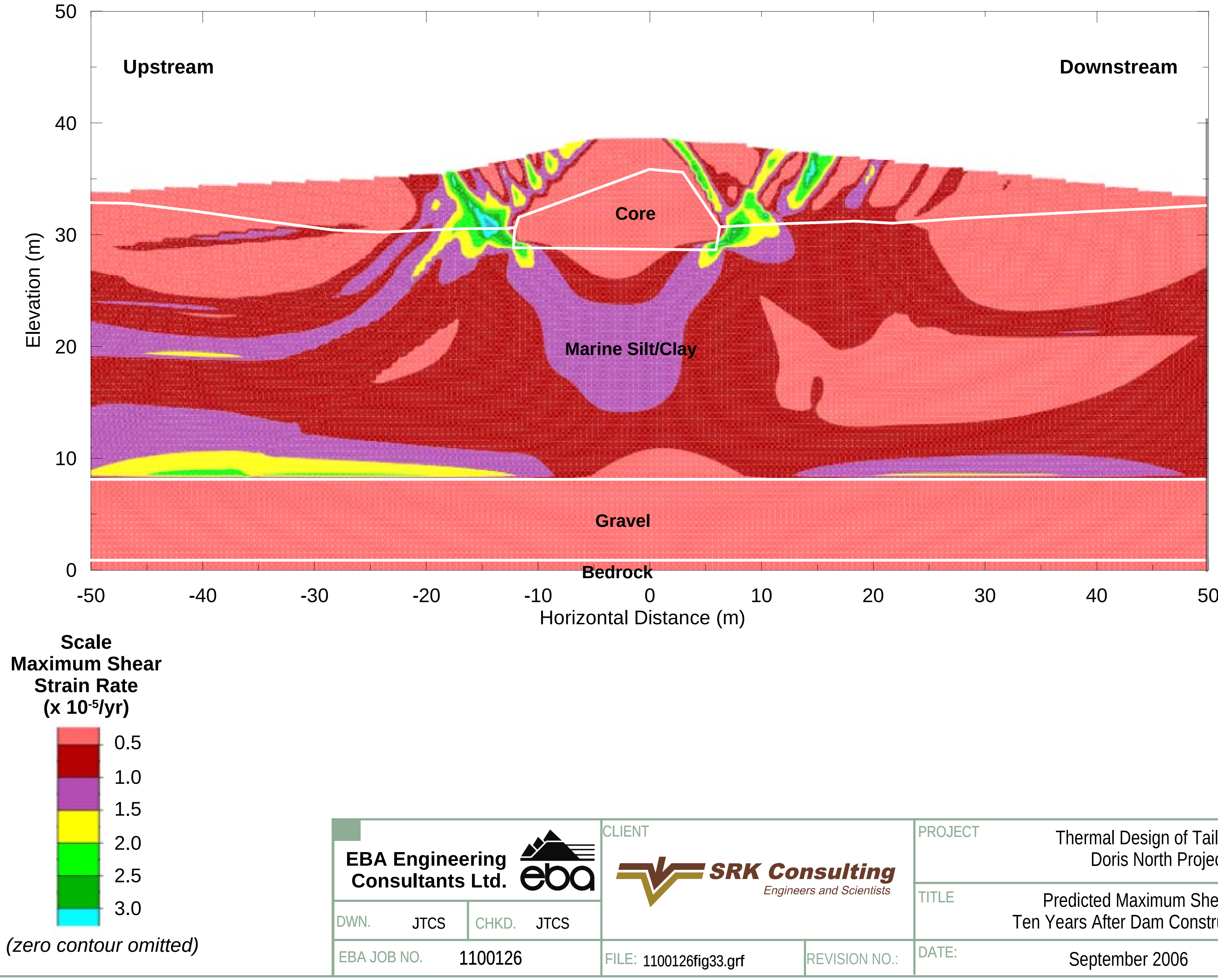
DWN. JTCS / CHKD. JTCS		TITLE	Predicted Maximum Shear Strain Ten Years After Dam Construction, South Dam
EBA JOB NO. 1100126	FILE: 1100126fig32.grf / REVISION NO.:	DATE: September 2006	Figure 32

15:50 09/22/06



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DWN. JTCS CHKD. JTCS

EBA JOB NO. 1100126

CLIENT

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Engineers and Scientists

FILE: 1100126fig33.grf

REVISION NO.:

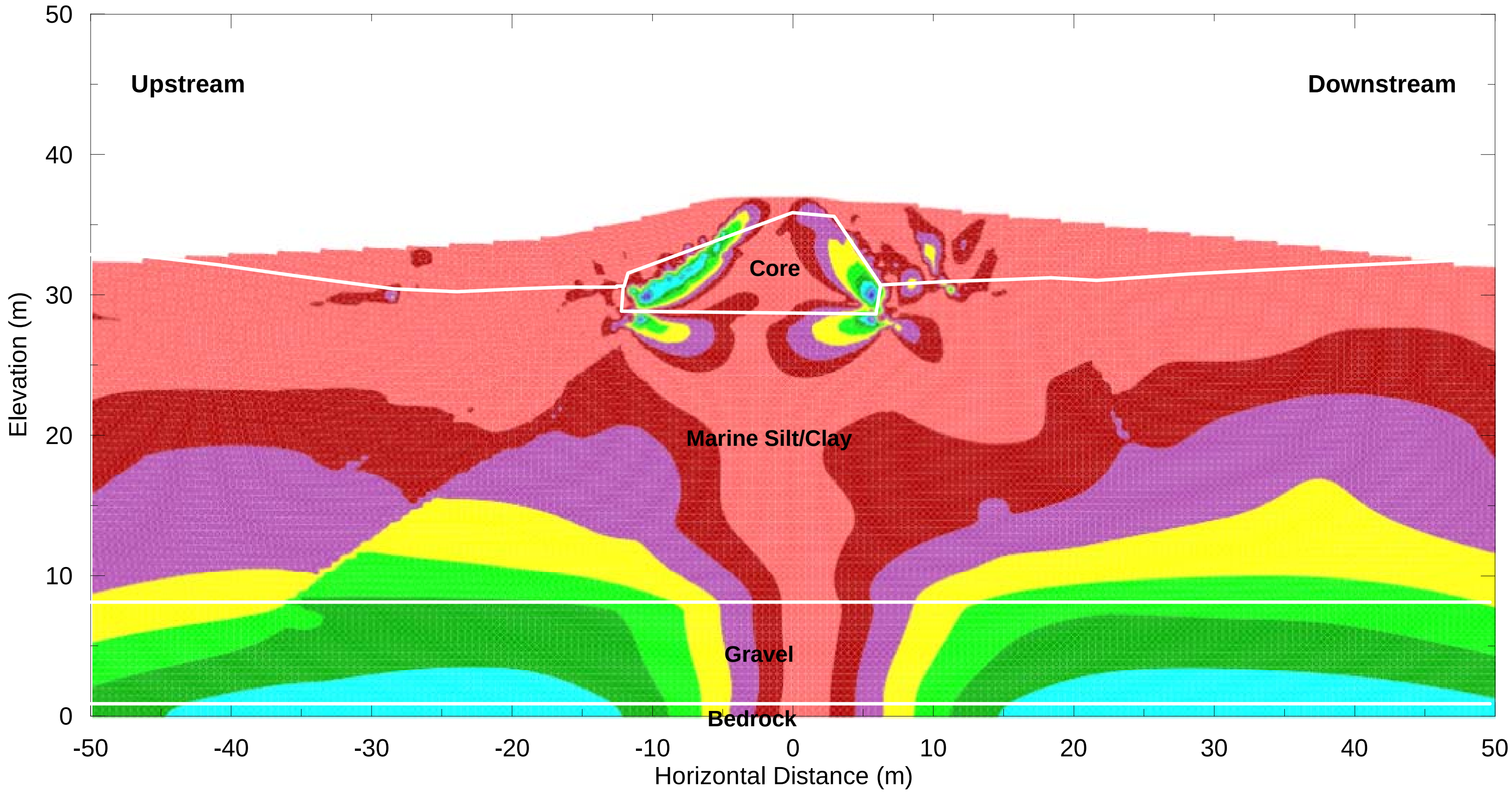
PROJECT
Thermal Design of Tailings Dams
Doris North Project, NU

TITLE
Predicted Maximum Shear Strain Rate
Ten Years After Dam Construction, South Dam

DATE: September 2006

Figure 33

15:51 09/22/06



(zero contour omitted)

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Engineers and Scientists

FILE: 1100126fig34.grf

REVISION NO.:

PROJECT
Thermal Design of Tailings Dams
Doris North Project, NU

TITLE
Predicted Shear Stress Distribution
Ten Years After Dam Construction, South Dam

DATE: September 2006

Figure 34

15:41 09/22/06

Appendix C

SRK Technical Memorandum re: Water Cover Design for Tail Lake

SRK Consulting
Engineers and Scientists

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Vancouver, B.C. V6E 3X2
Canada

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www.srk.com

Tel: 604.681.4196
Fax: 604.687.5532

Technical Memorandum

To:	Brian Labadie	**Date:**	September 16, 2005
cc:	Project File	**From:**	Maritz Rykaart, Ben Wickland
Subject:	Water Cover Design for Tail Lake	**Project #:**	1CM014.006

1 Introduction

This technical memorandum documents the design procedure, calculations and assumptions for the minimum water cover thickness of Tail Lake. Tail Lake will be used to sub-aqueously deposit tailings from the Doris North Project, and upon final closure there will be a permanent water cover over the tailings of 4.0 m. The calculations documented in this memorandum provide justification that this water cover is adequate.

The primary purpose of a water cover is to ensure that the covered mine waste, in this case tailings, is kept from oxidizing. Oxidizing will result in geochemical changes to the tailings, which in turn will result in poor quality water. It is generally understood that a stagnant water column of 0.3 m is sufficient to prevent oxidization of the underlying waste; however, in nature the water column cannot be stagnant, and as a result the tailings bed stability is affected through physical processes such as wave action, seiching, seasonal lake turnover, currents, and ice entrainment. The general rule of thumb is therefore to ensure a water cover of at least 1.0 m, to counter these processes. Such rules of thumb are however only a guideline, and cannot be used for an actual water cover design.

According to the MEND 1998 guidelines (MEND 1998), the objective of water cover design is: *"…to provide an adequate depth of water to ensure the consolidated bed of tailings is not entrained or remobilized during operation and after closure of the pond."* The water cover must be deep enough that the tailings do not become re-suspended due to wind generated waves and currents. Re-suspension occurs when the resistance of the bed of tailings is overcome by action of overlying water. The resistance of the bed is dependent on particle size, density, and cohesion. The action of the overlying water-wave action is dependent on:

- fetch length, the maximum distance of water over which waves may be generated,
- wind speed, for a maximum return period, and
- wind direction, duration.

This technical memorandum presents the design calculations for a minimum water cover thickness to prevent re-suspension from occurring.

2 Water Cover Design Approach

The current state-of-the art in water cover design is the procedure documented in MEND (1998). According to this guideline, there are five processes that affect bed stability; seiching, seasonal lake turnover, currents, wave action and ice entrainment. The guideline suggest that for small tailings impoundments (less than 5 km^2 water body area), and a water depth of 0 to 10 m, that only wave action and ice entrainment need to be accounted for in the design. Since the Tail Lake water body will vary in size between 81 and 130 ha (0.8 to 1.3 km^2), and its depth is between 4.0 and 9.2 m (this is based on the water level in Tail Lake ranging between 28.3 m and 33.5 m, with the tailings at an elevation of 24.3 m), it clearly falls within this category.

Note that the surface areas quoted for Tail Lake in this technical memorandum is based on the engineering stage curve for Tail Lake which includes the areas leading up to the North and South Dams. The actual body of water in Tail Lake at the normal water elevation of 28.3 m is 76.6 ha (as reported in the NNLP) in size; however, if the surface area leading up to the dams are included, the area increases to about 81 ha.

For re-suspension due to wave action, the MEND (1998) guideline uses the method proposed by Lawrence *et al*. (1991) to determine minimum water cover depth, but couples his approach with a critical bed velocity computation derived from the work of Komar and Miller (1975a,b). Since the modification adopted by MEND (1998) is less conservative than the original Lawrence *et al*. (1991) method, SRK have selected to use both methods in calculating a safe water cover thickness for Tail Lake. Both of these methods provide a way of calculating the minimum water cover depth at which no tailings re-suspension will occur, i.e. if the minimum water cover depth requirement is satisfied, then there will be no re-suspension of tailings.

Mian and Yanful (2001) and Bennet and Yanful (2001) has been documenting their research on water covers, and suggest that the procedures for water cover design, such as those proposed by Lawrence *et al*. (1991) and MEND (1998) are perhaps too conservative, and that water cover design should be based on an allowable re-suspension value, i.e. the water cover can be designed to allow some re-suspension provided that that amount of re-suspension would not result in exceedence of water quality criteria. This research has culminated in the development of a proposed new design methodology for selecting an optimum water cover depth (Samad and Yanful 2005). This method calculates the bed erosion for any specific water cover depth, using a similar wave theory approach as Lawrence *et al*. (1991), but refines it to account for shallow water waves and counter current flow. Furthermore, Samad and Yanful (2005) suggest that the tailings impoundment should be divided into a grid, and a minimum water cover depth requirement at each grid point should be calculated. This refinement accounts for changes in fetch distance and bathymetry at each grid point, and generally results in a reduced minimum water cover depth requirement. The grid method proposed by Samad and Yanful (2005) is less conservative than the methods described by MEND (1998) and Lawrence *et al*. (1991) and was therefore not applied to Tail Lake.

3 Minimum Water Cover Design

3.1 Primary Design Assumptions and Input Data

The primary design variables required for the water cover design using the MEND (1998) and Lawrence *et al*. (1991) methods are summarized in Table 1.

Table 1. Values of water cover design variables

Parameter	Baseline Design Value	Possible Range	Source
Fetch Distance [F]	1,350 m	500 to 3,500	Topographical maps of Tail Lake
Wind Speed [U, U_w]	11.1 m/s	6.1 to 22.2 m/s	Cambridge Bay hourly wind speed data
Threshold Velocity [U_t]	0.04 m/s	0.005 to 0.1 m/sec	Lawrence *et al*. (1991)
Wave Height Ratio [R]	1	Constant	Lawrence *et al*. (1991) & MEND (1998)
Median Particle Size [D_{50}]	0.06 mm	0.0001 to 0.08 mm	SRK 2005
Sediment Density	1,230 kg/m^3	Constant	SRK 2005

3.2 Results

Results of the water cover design calculations are presented in Figures 1, 2, 3 and 4. Each of these figures show the minimum water cover as calculated using both the conservative Lawrence *et al*. (1991) and the less conservative MEND (1998) methods. Figure 1 demonstrates the sensitivity of the calculation to fetch distance. As the fetch distance increases, the minimum water cover depth increases, with the MEND (1998) method suggesting that the water cover should be between 0.4 and 1.6 m over the likely range of fetch distances applicable at Tail Lake. Similarly, according the Lawrence *et al*. (1991) method, the range in water cover should be between 0.8 and 3.3 m.

The effect of wind speed on the water cover is illustrated in Figure 2. With increasing wind speed, the minimum water cover increases. According to the MEND (1998) method, the water cover should be between 0.5 and 1.7 m, whilst the equivalent water cover according to the Lawrence *et al*. (1991) method, should be between 1.0 and 3.0 m.

The MEND (1998) method uses the median particle size as a variable to account for bed shear stress, whilst the Lawrence *et al*. (1991) method uses the particle threshold velocity to account for bed shear stress. Figures 3 and 4 present the effect that different values of these properties have on the minimum water cover. As can be seen in Figure 3, as the median particle size increase, the required water cover decreases. For the range of likely particle sizes in the Doris North Project this will result in a range in water cover between 0.8 and 2.0 m. Similarly, as the threshold velocity increases, the water cover reduces for a likely range of 1.2 to 2.8 m of water cover, as illustrated in Figure 4.

For the chosen design parameters as listed in Table 1, the minimum water cover, depending on the calculation method used, ranges between 0.8 and 1.7 m. Using the values in the range of design parameters for each variable that would result in the most significant water cover, i.e. the maximum fetch distance, the maximum wind speed, the smallest median particle size and the lowest threshold velocity, results in a minimum water cover requirement of between 2.2 and 3.6 m, depending on which method is used. This is however a worst case scenario, included to demonstrate sensitivity of the calculation methods.

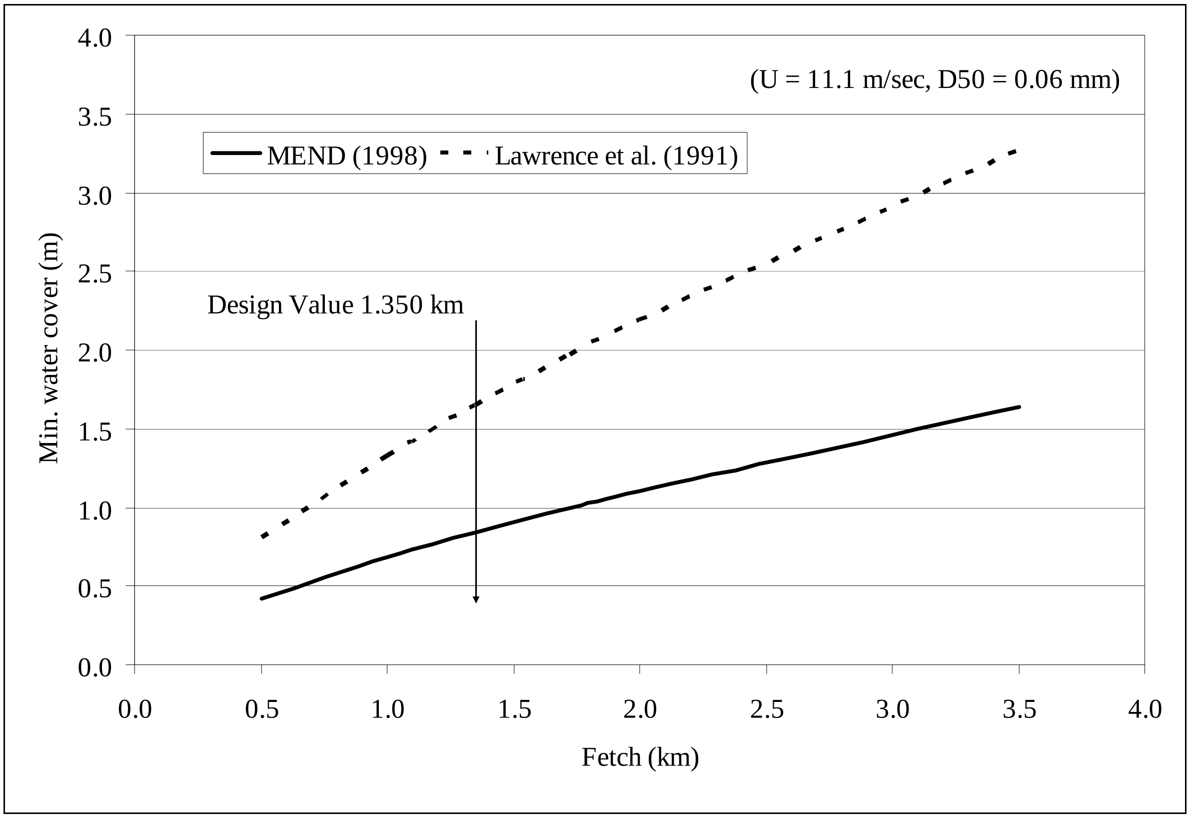

Figure 1. Water cover versus fetch distance

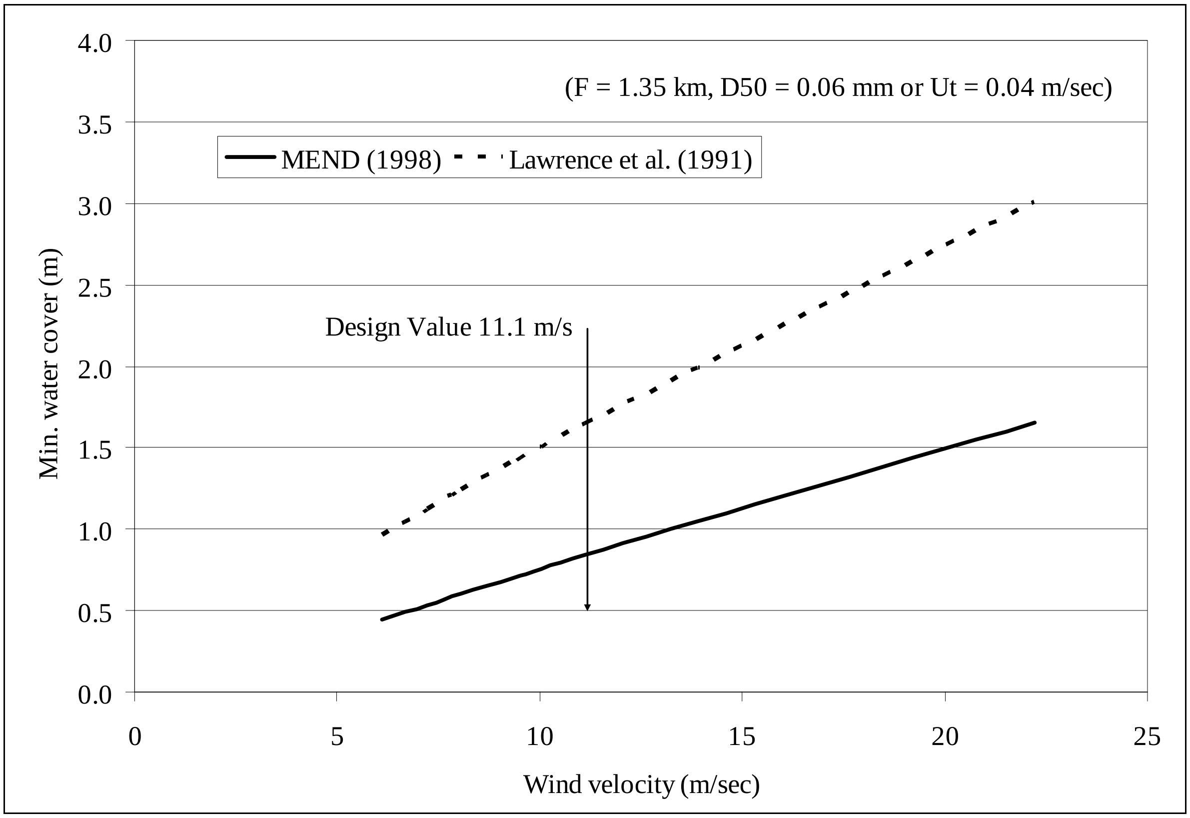

Figure 3. Water cover versus wind velocity

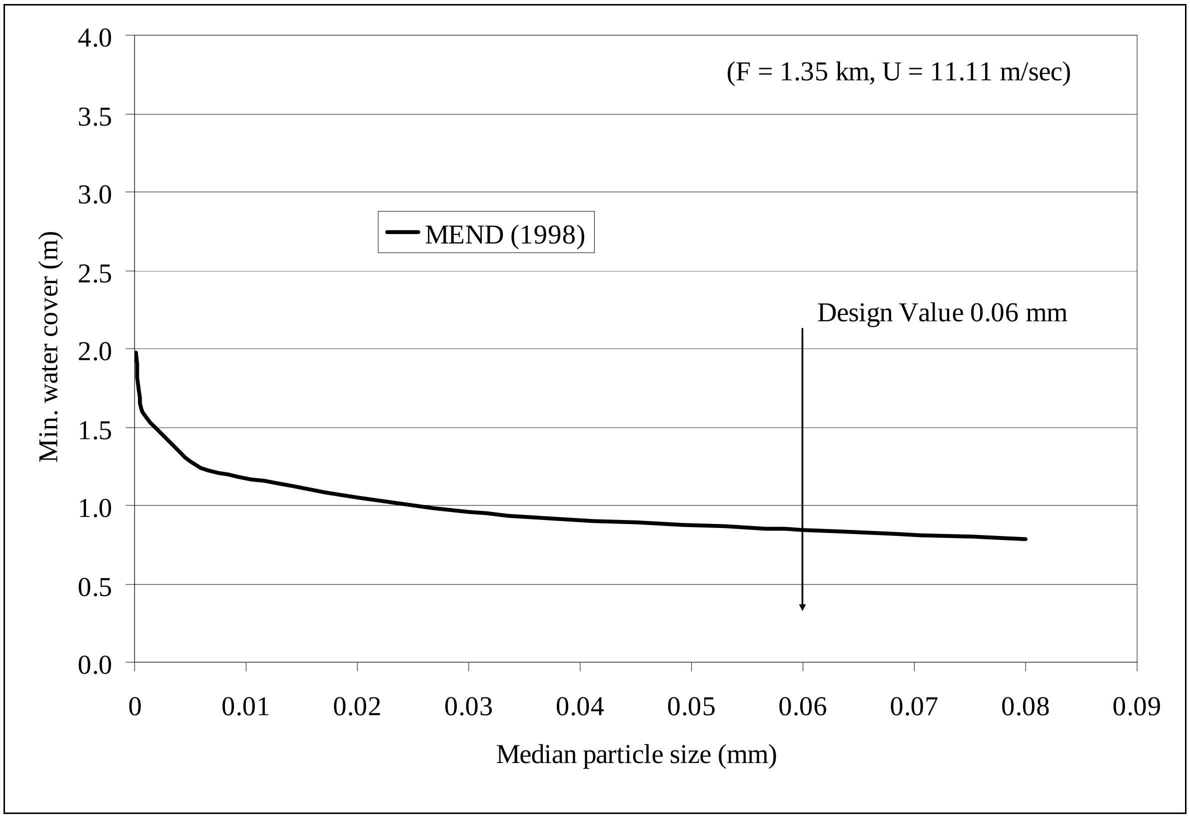

Figure 3. Water cover versus median particle size

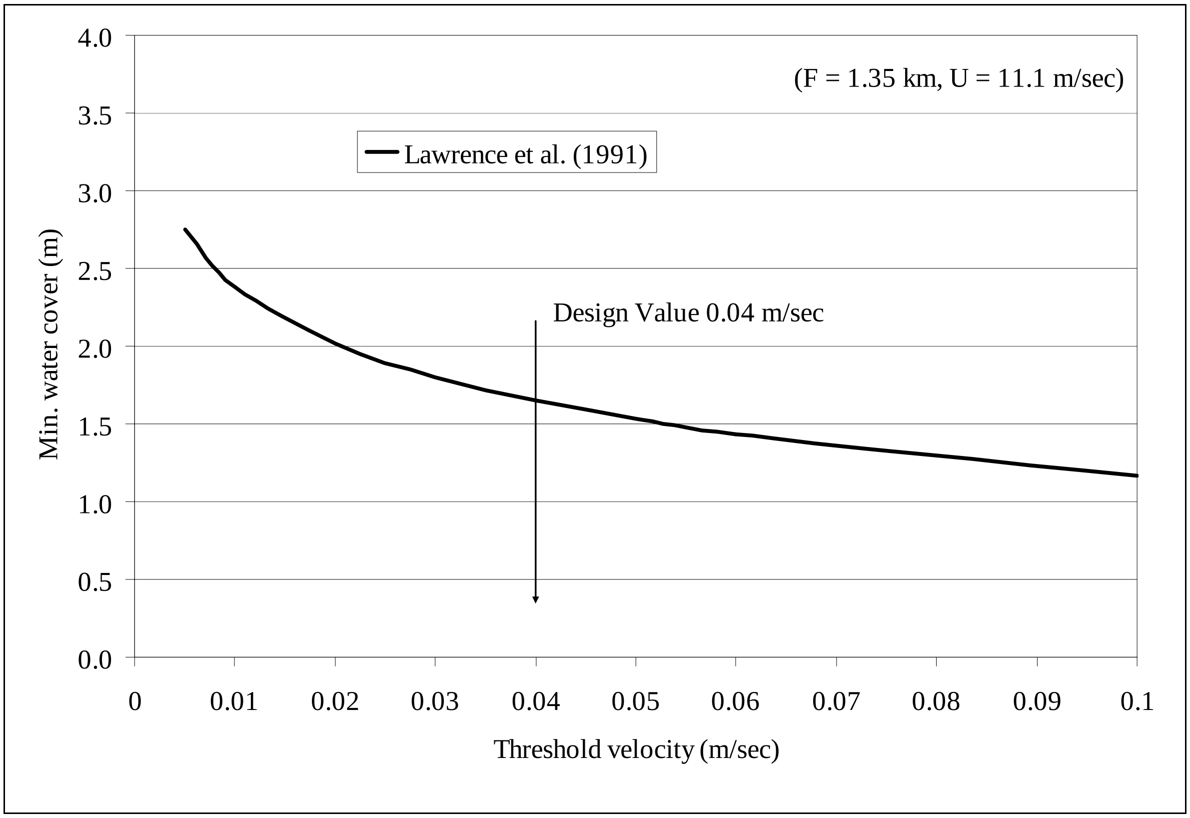

Figure 4. Water cover versus threshold velocity

3.3 Validity of Lawrence *et al*. (1991) and MEND (1998) Results

The water covers determined using the Lawrence *et al*. (1991) and MEND (1998) procedures assumes that wave development is consistent with deep water wave theory. Deep water wave theory applies when the ratio of water depth over wavelength is less than 0.5, which is a condition which is typically not met for shallow water covers (typically less than 5 m deep). Under such circumstances, shallow water wave theory must be applied, which results in calculating smaller significant wave heights and shorter significant wave periods.

Both the Lawrence *et al*. (1991) and MEND (1998) design procedures suggest that the water cover design does not apply if the deep water wave condition cannot be met; however, they do not propose a solution to overcome this problem. Samad and Yanful (2005) does provide a procedure to calculate the significant wave height and period using shallow wave theory, and SRK conducted a sensitivity analysis on the range of input parameters evaluated for Tail Lake to determine how much the significant wave height and significant wave period would vary if the appropriate wave theory was applied. The results of this sensitivity analysis are presented in Figures 5 and 6. SRK then substituted the appropriate shallow water wave theory significant wave height and wave period values into the Lawrence *et al*. (1991) and MEND (1998) design procedures and concluded that there was an overall variance in the design water cover of 4%, which only applied to shallow covers of less than 1 m thick. Therefore, SRK is satisfied that the design water covers are appropriate.

To summarize, the minimum water covers, based on wave action for the Doris North Project will be between 0.83 and 1.7 m, depending on which calculation method is used (this assumes a correction for the shallow wave theory). However, since the MEND (1998) method is considered the current state-of-the art method in calculating minimum water covers, the overall recommended minimum water cover due to wave action is 0.83 m.

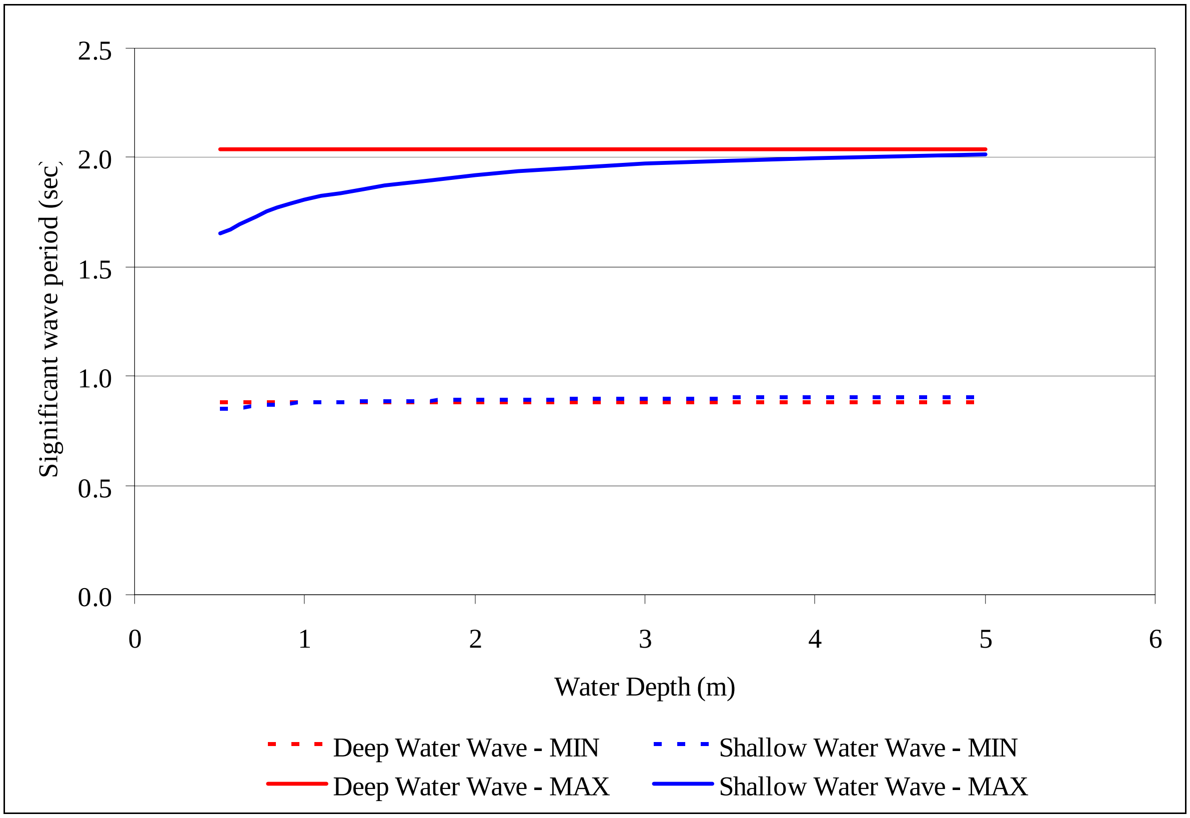

Figure 5. Variation of the significant wave period over different water covers using both shallow and deep water wave theory

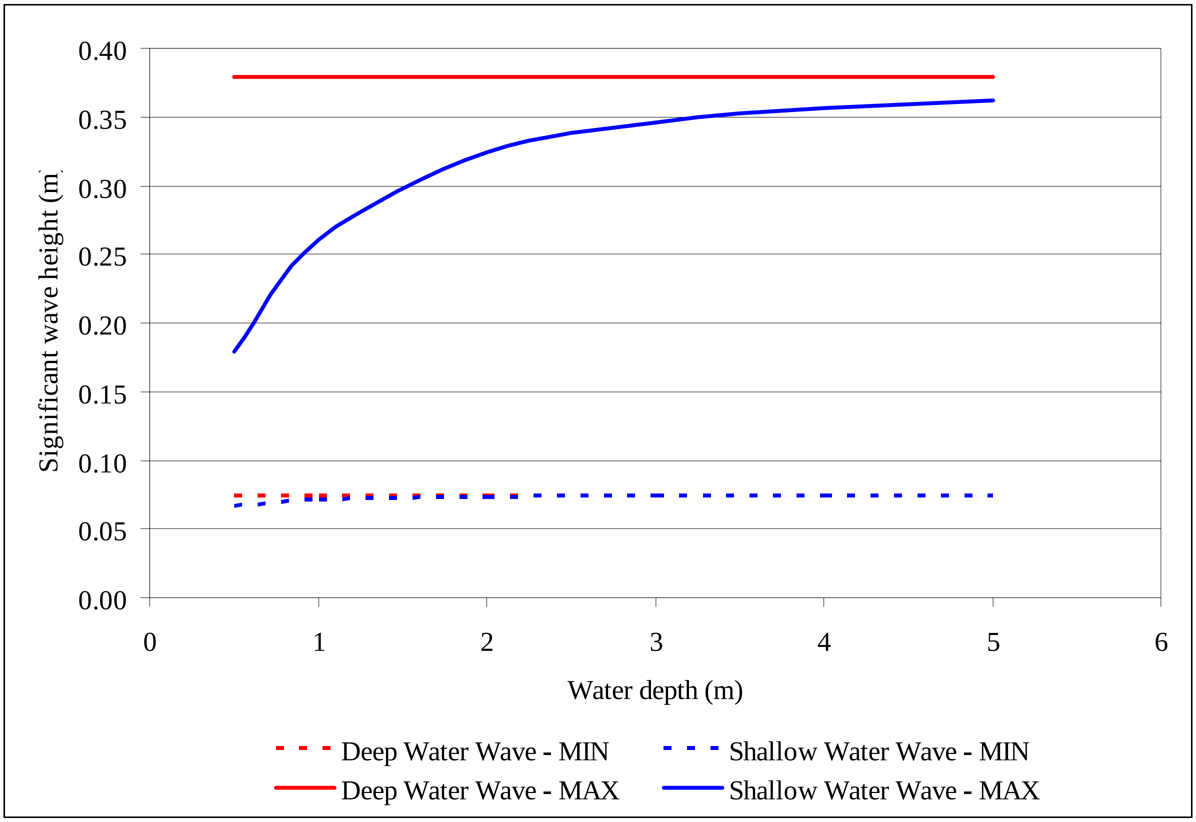

Figure 6. Variation of the significant wave height over different water covers using both shallow and deep water wave theory

3.4 Ice Entrainment

The MEND (1998) guideline recommends that the minimum water cover should be at least 10% greater than the maximum lake ice thickness that the pond might incur. A detailed study of lake ice thickness has not been conducted at Tail Lake; however, select ice thickness measurements during water sample and drilling programs suggest that the maximum lake ice thickness varies between 1.9 m and 2.2 m. Regional studies on lake ice thickness confirm that a reasonable maximum ice thickness at Tail Lake is probably around 2.2 m. Therefore, the minimum water cover to prevent tailings re-suspension through ice entrainment is 2.2 m + 10% = 2.42 m.

This value is greater than the selected design criteria for minimum water cover due to wave action, and therefore the specified minimum water cover for Tail Lake will be dominated by ice the ice entrainment value of 2.42 m. Furthermore, since the operating water cover would be 4.0 m, there is a significant factor of safety against ice entrainment. Conversely, the tailings surface could be up to 1.58 m above the design elevation of 24.3 m before ice entrainment would start to contribute towards tailings re-suspension.

Providing a 1.0 m allowance for an uneven final tailings deposition surface, would result in the minimum water cover depth being reduced to 3.0 m, which in turn implies that the factor of safety against ice entrainment reduces slightly, but still remain significant. Figure 7 presents a schematic of Tail Lake, including the deposition zones of tailings, confirming that at any given time, assuming level tailings surface, the minimum water cover depth of Tail Lake would be 4.0 m.

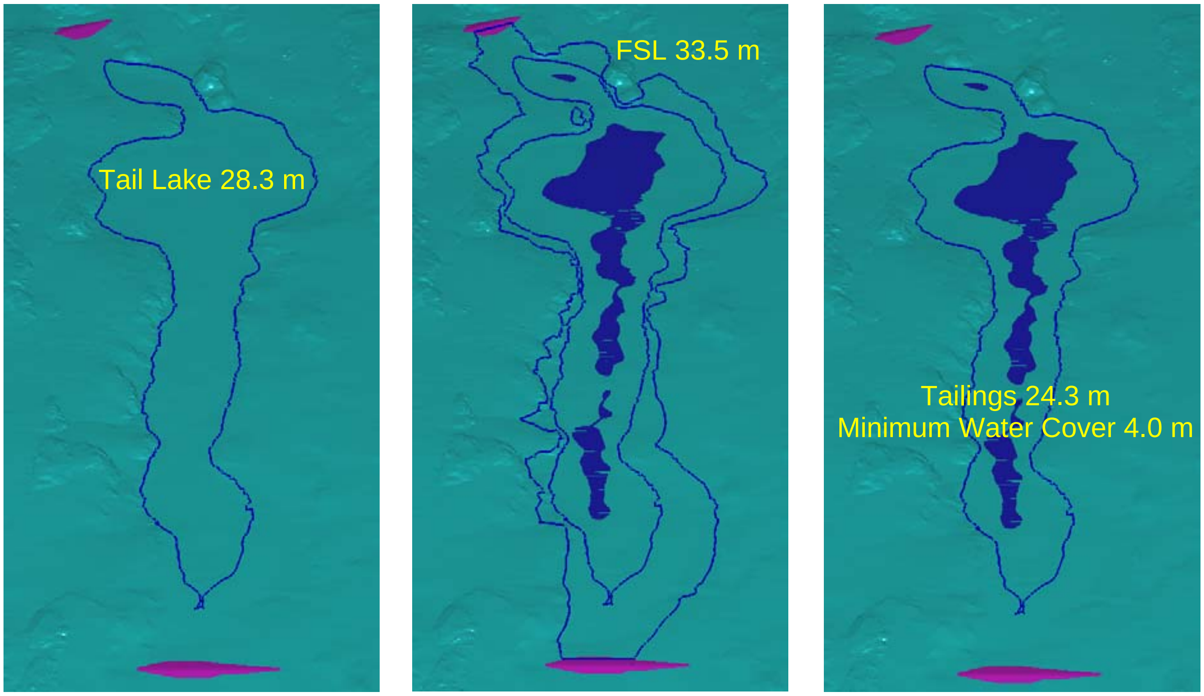

Figure 7. Schematic of tailings deposition location in Tail Lake

3.5 Extreme Drought Conditions

The design guideline by MEND (1998) states that the water cover should be designed taking into account standard water balance principles; however, it does not provide any procedure for taking into account drought conditions. Yanful (2005) documents a detailed procedure to account for drought conditions in the evaluation of a minimum water cover design.

Due to the substantial factor of safety available for the minimum water cover at Tail Lake, SRK opted to consider the effect of a severe drought on the water cover using a simplified procedure. The basic assumptions of this analysis can be summarized as follows:

- Tail Lake elevation at start of drought = 28.3 m (i.e. the post closure scenario)
- 5-year long drought
- Zero precipitation for entire duration (no rain or snow)
- 20% above average lake evaporation (i.e. 220 mm + 20% = 264 mm)

Applying these conditions, would result in a final lake water elevation in Tail Lake after the drought of 27.0 m. At this time, the minimum water cover depth over the tailings (at elevation 24.3 m) would be 2.7 m.

Furthermore, it should be noted that the total volume of water lost during this simulated drought is just under 1.0 million m^3, or 48% of the total volume of free water in Tail Lake (i.e. the volume excluding tailings). Under average climatic conditions it would take two years before the water level will reach the natural outflow elevation of 28.3 m, providing ample time for settlement, should any particles be re-suspended in any way.

This evaluation of a drought is extremely conservative, but still the minimum design water cover criteria are upheld, for wave action and ice entrainment. As before, providing a 1.0 m allowance for an uneven final tailings deposition surface would result in the minimum water cover depth during an extreme drought being reduced to 1.7 m. Under such a scenario, the minimum water cover requirement against wave action would still be upheld; however, ice entrainment can occur.

This is however not a significant concern, since as described above under such severe drought conditions, Tail Lake water cannot flow out of the basin, since the natural outflow elevation is at 28.3 m. Considering the fact that it would take two years before outflow would start again ice entrainment during drought conditions is not considered a concern.

3.6 Minimum Water Depth Requirement for Bed Stability

Table 2 summarizes the minimum water cover requirements to ensure bed stability as described in the preceding sections. It is clear that the dominating process in determining the water cover is ice entrainment. Therefore, the minimum water cover for Tail Lake should be 2.42 m. This implies that the tailings surface can have undulations up to 1.58 m high, allowing a substantial safety margin.

Furthermore, as demonstrated in the preceding sections, using the most conservative calculation method, and the worst case input variables, the maximum water cover would have to be 3.6 m, which still leaves a safety margin of 0.4 m. There is therefore no doubt that the 4.0 m water cover is sufficient to prevent tailings re-suspension in Tail Lake.

Table 2. Summary of minimum water cover requirements for Tail Lake

Condition	Design Value
Planned final tailings surface	24.3 m
Final water level in Tail Lake	28.3 m
Planned water cover thickness	4.0 m
Possible loss in water cover thickness due to uneven tailings deposition	1.0 m (remaining water cover = 3.0 m)
Possible loss in water cover thickness due to drought conditions	1.3 m (remaining water cover = 2.7 m)
Possible loss in water cover thickness due to uneven tailings deposition and drought conditions simultaneously	2.3 m (remaining water cover = 1.7 m)
Minimum water cover due to wave action (deep water wave theory)	0.80 m
Minimum water cover due to wave action (shallow water wave theory)	0.83 m
Minimum water cover due to ice plucking	2.42 m

4 References

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Appendix D
Winter 2006 Geotechnical Field Investigation (SRK 2006b)

Winter 2006 Geotechnical Field Investigation and Geophysical Survey at Tail Lake, Doris North Project, Nunavut, Canada

Prepared for

Miramar Hope Bay Limited

Prepared by

SRK Consulting
Engineers and Scientists

August 2006

Winter 2006 Geotechnical Field Investigation and Geophysical Survey at Tail Lake, Doris North Project, Nunavut, Canada

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Table of Contents

1 Introduction 1
- 1.1 Background 1
- 1.2 Summary of Field Program 1

2 Methodology 2
- 2.1 Drilling 2
- 2.2 Laboratory Testing 2
- 2.3 Thermistor Maintenance 3
- 2.4 Geophysics 4

3 Results of Drilling Program 5
- 3.1 Summary of Drill Hole Profiles 5
  - 3.1.1 SRK06-01 5
  - 3.1.2 SRK06-02 5
- 3.2 Laboratory Testing Results 5

4 Results of Geophysical Survey 6

5 Discussion 7

6 References 9

List of Tables

Table 1: As-Built Drill Hole Coordinates 2
Table 2: Laboratory Testing Program 3
Table 3: Doris North Project Site Thermistors 4
Table4: Results of Foundation Indicator Testing 5

List of Figures

Figure 1: Winter 2006 Drill Hole Locations
Figure 2: Thermistor Locations

List of Appendices

Appendix A: Drill Logs
Appendix B: Laboratory Test Results
Appendix C-1: Thermistor String Data – Figures
Appendix C-2: Thermistor String Data – Tables
Appendix D: AMCL Geophysical Study Report

1 Introduction

1.1 Background

Miramar Hope Bay Limited (MHBL) is in the process of preparing final detailed engineering designs for the Doris North Project located in the Hope Bay Belt, Nunavut. MHBL contracted SRK Consulting (Canada) Inc. (SRK) to carry out additional geotechnical field investigations to support the final design of the tailings impoundment. This field program, which was carried out in April and May 2006 was specifically designed to achieve the following objectives:

- Confirmation and infill foundation characterization drilling upstream and downstream of the two tailings dams, i.e. the North and South Dams.
- Carry out annual thermistor maintenance on the 32 thermistors that has been installed since 2003 in the belt, and to update the thermal database.
- Attempt to conduct a geophysical survey of the ice-rich permafrost overburden around Tail Lake and along the two dam footprints, to possibly allow for a more comprehensive characterization of potential shoreline erosion zones.

This report presents the results of the study as described.

1.2 Summary of Field Program

The confirmation and infill drill program was to include ten drill holes, three at each dam, one along the North Dam spillway, and the remaining three at selected locations around Tail Lake. Unseasonably warm conditions however resulted in the program being shut down after completing only two drill holes at the South Dam (Figure 2).

All the thermistors were physically inspected by an SRK engineer and regular maintenance was carried out on thermistors to ensure that they remain operational as long as possible. Data was manually collected from all thermistors during the summer season, and this report includes all updated data received to the end of September 2006.

A geophysical survey was attempted to try and determine the ice content in the permafrost overburden. A specialist subcontractor tried four different geophysical techniques with varying degrees of success. Geophysical techniques are not well suited to ice-rich permafrost in a saline environment, and ultimately the only useful information that could be gleaned was depth to bedrock in shallow overburden zones using Ground Penetrating Radar.

Throughout the field program the weather conditions was relatively unseasonable, and highly variable. Winds were generally from the north and northeast at 5-20 km/h, with daytime temperatures reaching 8°C and overnight lows down to of -25°C. During the program, conditions ranged from sunny clear skies to overcast with blizzards and freezing rain.

2 Methodology

2.1 Drilling

The Tail Lake boreholes were drilled using the HQ3 (61mm core, 96mm hole diameter) triple tube diamond coring system with a Boyle 17 hydraulic drill rig. To ensure recovery of the ice-rich overburden the drilling fluid had to be chilled to below 8^{o}C. A chiller was not available, so the chilled brine was made by mixing water from Tail Lake with sodium chloride and adding snow and ice to cool the liquid down. Due to the unseasonably warm weather the brine could seldom be chilled to lower than about -2^{o}C.

From past experience the maximum run length of the drill was set at about 1.5 m. As recovery dropped, run lengths were reduced even further. Drilling was done using 2 crews, working 12-hour shifts.

SRK engineers Messrs. Alvin Tong, E.I.T. and Lowell Wade, E.I.T. supervised the drill, logged the core, and collected representative soil samples for geotechnical testing. Samples were shipped to EBA Engineering’s soil testing laboratories in Yellowknife and Edmonton. All remaining core is stored in core boxes, outside, under ambient conditions at Windy Camp.

The drill hole locations were set out by MHBL according to co-ordinates provide by SRK. Survey details of the completed drill holes are listed in Table 1.

Table 1: As-Built Drill Hole Coordinates

Hole ID	Northing[1]	Easting[1]	Inclination
SRK06-01	7555903.0	435623.0	-90°
SRK06-12	7555939.0	435590.0	-90°

1. UTM Projection NAD 83 Zone 13.

2.2 Laboratory Testing

A total of 24 bulk (i.e. disturbed) soil samples were collected and shipped to the laboratory. Seven of these samples were ultimately subjected to basic foundation indicator and salinity testing. Table 2 summarizes the samples collected and the testing carried out.

Table 2: Laboratory Testing Program

Sample [ID:Depth (Type)][1]	**Natural Moisture Content**	**Particle Size Distribution**		**Atterberg Limits**	**Salinity**
		Sieve	**Hydrometer**		
SRK06-01-01: 2.0m (CL)[2]					
SRK06-01-02: 4.4m (CL)					
SRK06-01-03: 5.4m (CL)	✓	✓	✓	✓	✓
SRK06-01-04: 8.0m (CL)					
SRK06-01-05: 9.5m (CL)					
SRK06-01-06: 12.0m (CL)	✓	✓	✓	✓	✓
SRK06-01-07: 16.0m (CL)					
SRK06-01-08: 18.0m (CL)					
SRK06-01-09: 19.3m (CL)					
SRK06-01-10: 21.3m (CL)	✓	✓	✓	✓	✓
SRK06-02-01: 0.6m (CL)	✓	✓	✓	✓	✓
SRK06-02-02: 3.7m (CL)					
SRK06-02-03: 5.5m (CL)					
SRK06-02-04: 7.3m (CL)					
SRK06-02-05: 10.3m (CL)					
SRK06-02-06: 11.8m (CL)	✓	✓	✓	✓	✓
SRK06-02-07: 13.2m (CL)					
SRK06-02-08: 14.7m (CL)					
SRK06-02-09: 16.7m (CL)					
SRK06-02-10: 19.0m (CL)	✓	✓	✓	✓	✓
SRK06-02-11: 24.5m (ML)[3]					
SRK06-02-12: 25.7m (ML)					
SRK06-02-13: 28.6m (ML)	✓	✓	✓	✓	✓
SRK06-02-14: 31.0m (ML)					

1. Soil type is designated soil symbol according to the Unified Soil Classification System (USCS).
2. CL = Clay.
3. ML = Silt.

2.3 Thermistor Maintenance

A total of 33 thermistor strings has been installed at the Doris North project site since 2003. Table 3 summarize the details of these strings, and their locations are depicted on Figure 3. An SRK engineer visited each of the 31 strings that are still operational, to collect data and to conduct maintenance and repair as necessary. Miramar staff continued to take regular readings of all thermistor strings during the summer months and Appendix C-1 and C-2 contain the complete updated data for all strings up to September 30, 2006.

Table 3: Doris North Project Site Thermistors

Drill Hole Number	String Serial Number	Number of Beads	String Status	Reference Document
SRK-11	00577-2	5	Working	SRK 2003a
SRK-13	00577-1	5	Broken – unrepairable	SRK 2003a
SRK-14	690007	6	Working	SRK 2003a
SRK-15	690012	10	Working	SRK 2003a
SRK-16	00577-3	5	Working	SRK 2003a
SRK-19	690006	6	Working	SRK 2003b
SRK-20	690009	6	Working	SRK 2003b
SRK-22	690003	6	Working	SRK 2003b
SRK-23	690008	6	Working	SRK 2003b
SRK-24	690001	6	Working	SRK 2003b
SRK-26	690002	6	Working	SRK 2003b
SRK-28	690011	6	Working	SRK 2003b
SRK-32	690010	6	Working	SRK 2003a
SRK-33	690005	6	Working	SRK 2003a
SRK-34A	690004	6	Working	SRK 2003a
SRK-35	690000	6	Working	SRK 2003b
SRK-37	690013	10	Working	SRK 2003a
SRK-38	TS0015	8	Working	SRK 2003a
SRK-39	TS0011	8	Working	SRK 2003a
SRK-40	TS0014	8	Working	SRK 2003a
SRK-41	TS0012	9	Working, 9th bead broken	SRK 2003a
SRK-42	TS0013	8	Working	SRK 2003a
SRK-43	TS0010	8	Working	SRK 2003a
SRK-50	TS1618	13	Working	SRK 2005a
SRK-51	TS2048	12	Working	SRK 2005b
SRK-52	TS2047	12	Working	SRK 2005b
SRK-53	TS1625	6	Working	SRK 2005b
SRK-54	TS1626	6	Working	SRK 2005a
SRK-55	TS1624	6	Broken – unrepairable	SRK 2005a
SRK-56	TS1621	6	Working, last 3 beads broken	SRK 2005a
SRK-57	TS1623	6	Working, two internal beads broken	SRK 2005b
SRK-58	TS1622	6	Working	SRK 2005b
SRK-62	TS2046	12	Working	SRK 2005b

2.4 Geophysics

SRK contracted Associated Mining Consultants Ltd. (AMCL) to carry out a two-phase geophysical survey on the ice-rich saline permafrost soils along the two dam alignments and along the entire Tail Lake perimeter. The first phase entailed carrying out a test survey using four different geophysical techniques, to determine which technique would be best suited to achieve the study objectives. The four techniques used included electrical imaging, time domain electromagnetics, seismic refraction and ground penetrating radar. The test sections was at the two dam locations, which could readily be correlated with good quality drill hole data.

Based on the results of the phase one test surveys, it was concluded that the second phase coverage would only be conducted using the ground-penetrating radar (GPR). The entire lake perimeter along the proposed full supply level (FSL) of 33.5 m was subsequently surveyed using the GPR.

3 Results of Drilling Program

3.1 Summary of Drill Hole Profiles

3.1.1 SRK06-01

SRK06-01 is a vertical hole that extends to a depth of 24.5 m. Sample recovery was about 90% over the entire length of the hole.

The stratigraphy consists of 0.2 m of surface organics and peat overlying 6.2 m of frozen sandy silt. Below this layer there was a 16.1 m thick layer of ice-rich silt and clay. The next soil unit is a thin veneer of silty sand with gravel, immediately above Gabbro bedrock, which is at 24.3 m.

3.1.2 SRK06-02

SRK06-02 is a vertical hole that extends to a depth of 35.6 m. During the drilling of this hole the brine could not be sufficiently chilled, with the resultant effect that only about a 65% core recovery was achieved.

This hole consists of 0.5 m of surface organics and peat overlying 5.0 m of sandy silt, overlying a 13.0 m thick layer of ice-rich silt and clay. This unit was followed by a thin layer of sand and gravel after which Gabbro bedrock was encountered at a depth of 34.7 m.

3.2 Laboratory Testing Results

Seven samples were subjected to basic foundation indicator testing, with the primary results summarized in Table 4. Complete laboratory data sheets are included as Appendix C.

Table4: Results of Foundation Indicator Testing

Sample [ID:Depth (Type)]	Salinity (ppt)	Water Content (%)	Liquid Limit (%)	Plastic Limit (%)	Plasticity Index (%)
SRK06-01-03: 5.4m (CL)	67	27.0	29	17	12
SRK06-01-06: 12.0m (CL)	44	57.3	44	22	22
SRK06-01-10: 21.3m (CL)	67	27.1	38	20	18
SRK06-02-01: 0.6m (CL)	6	63.3	33	22	11
SRK06-02-06: 11.8m (CL)	6	46.3	43	22	21
SRK06-02-10: 19.0m (CL)	80	68.0	37	19	18
SRK06-02-13: 28.6m (ML)	86	20.8	N/A	N/A	N/A

4 Results of Geophysical Survey

Since geophysical mapping of ice-rich saline permafrost poses significant technical challenges, four different techniques was first tried along the North and South Dam center lines where drill hole data was readily available with which to calibrate the geophysics. Electrical imaging (OhmMapper), time domain electromagnetic (TEM), and ground penetrating radar (GPR) was tested at the South Dam and seismic refraction and TEM was tested at the North Dam.

The OhmMapper results showed poor penetration into the conductive clays close to surface. The signal was limited to the first 2 or 3 m of the profile after which it becomes noisy and the data showed poor correlation if any with the drill hole data.

TEM data was also very noisy, and although a number of different field settings were tested, the inverted model could not converge to an acceptable solution.

The seismic refraction data did yield some interesting results, and it was possible, at least for some tested sections, to accurately map the boundary between frozen overburden and bedrock. Unfortunately, the interpreted profile was not supported by the drill holes in all locations, and AMCL concluded that the small contrast in velocity between frozen soil and the underlying basalt is probably the primary reason.

Subsequently, the OhmMapper, TEM and seismic refraction technique was not used in the second phase of the study.

The GPR did yield reasonably useful data; however, the penetration was limited to between 5 and 10 m. Within the limit of penetration, the GPR could also be used to indicate the silt and clay to sand and gravel interfaces. Appendix D contains the complete AMCL report, which includes complete profiles of the Tail Lake perimeter along which the GPR survey was done.

5 Discussion

The drilling program could not be completed as planned due to unseasonably warm conditions, which prevented the drilling fluid from being chilled sufficiently. The two holes that could be completed at the South Dam confirm that the foundation conditions along the dam centerline is consistent with that at some distance immediately upstream and downstream of the dam centerline. This provides sufficient confidence that the current foundation information available for the two tailings dams is sufficient.

Of the four geophysical techniques tested, only the GPR yielded useful data. The maximum penetration achieved with the GPR was limited to between 5 and 10 m, and did manage to identify the frozen ground to bedrock interface if it fell within the penetration depth. The GPR could also in some instances identify the silt and clay interface with sand and gravel.

The entire Tail Lake perimeter along the proposed full supply level (35.5 m) was profiled using the GPR, and provides some infill information useful in characterizing the potential for shoreline erosion around the lake.

This report, **"Winter 2006 Field Investigation and Geophysical Survey at Tail Lake, Doris North Project, Nunavut, Canada"**, has been prepared by SRK Consulting (Canada) Inc.

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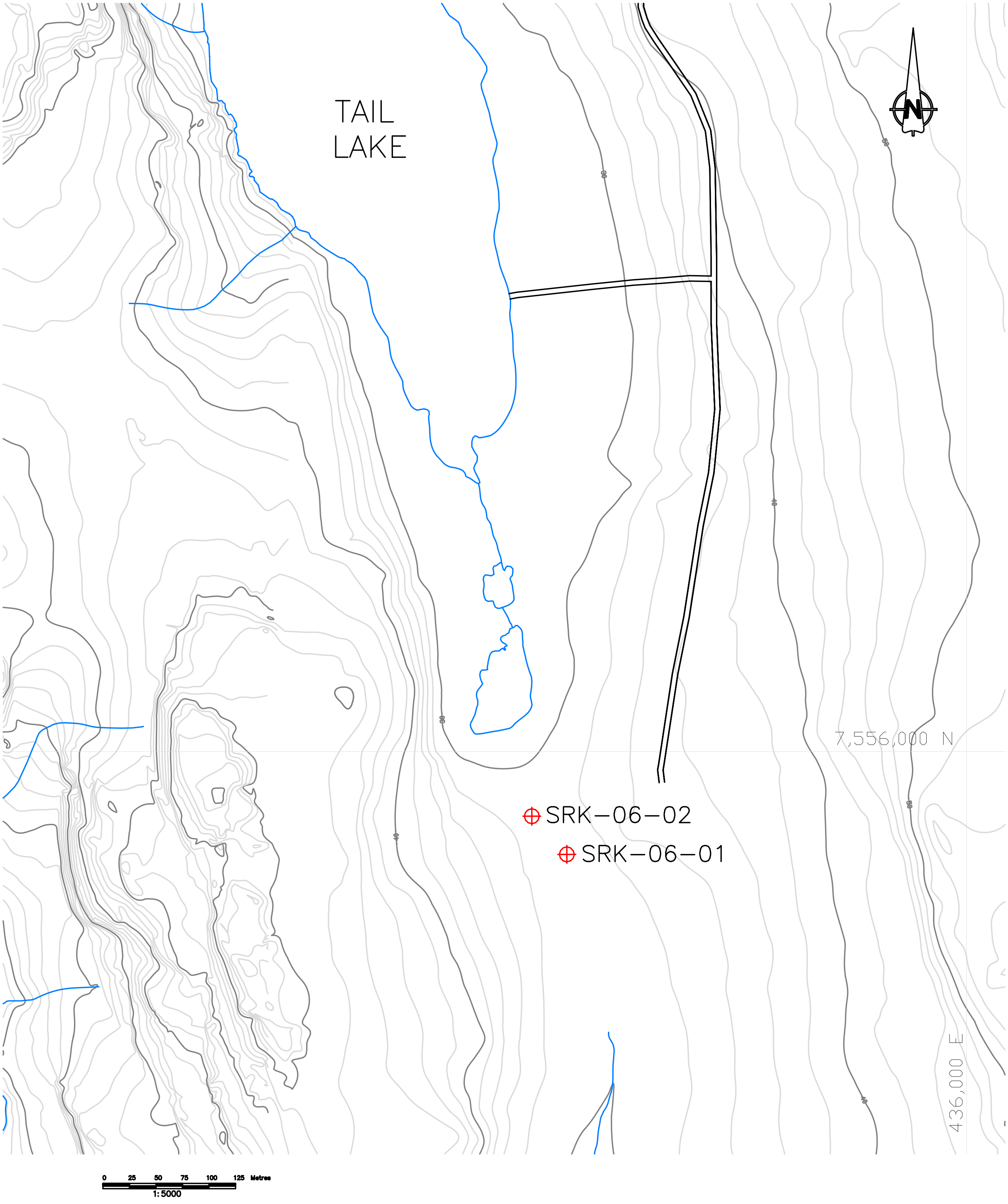
Figures



SRK Consulting
Engineers and Scientists
Vancouver

MIRAMAR HOPE BAY LIMITED

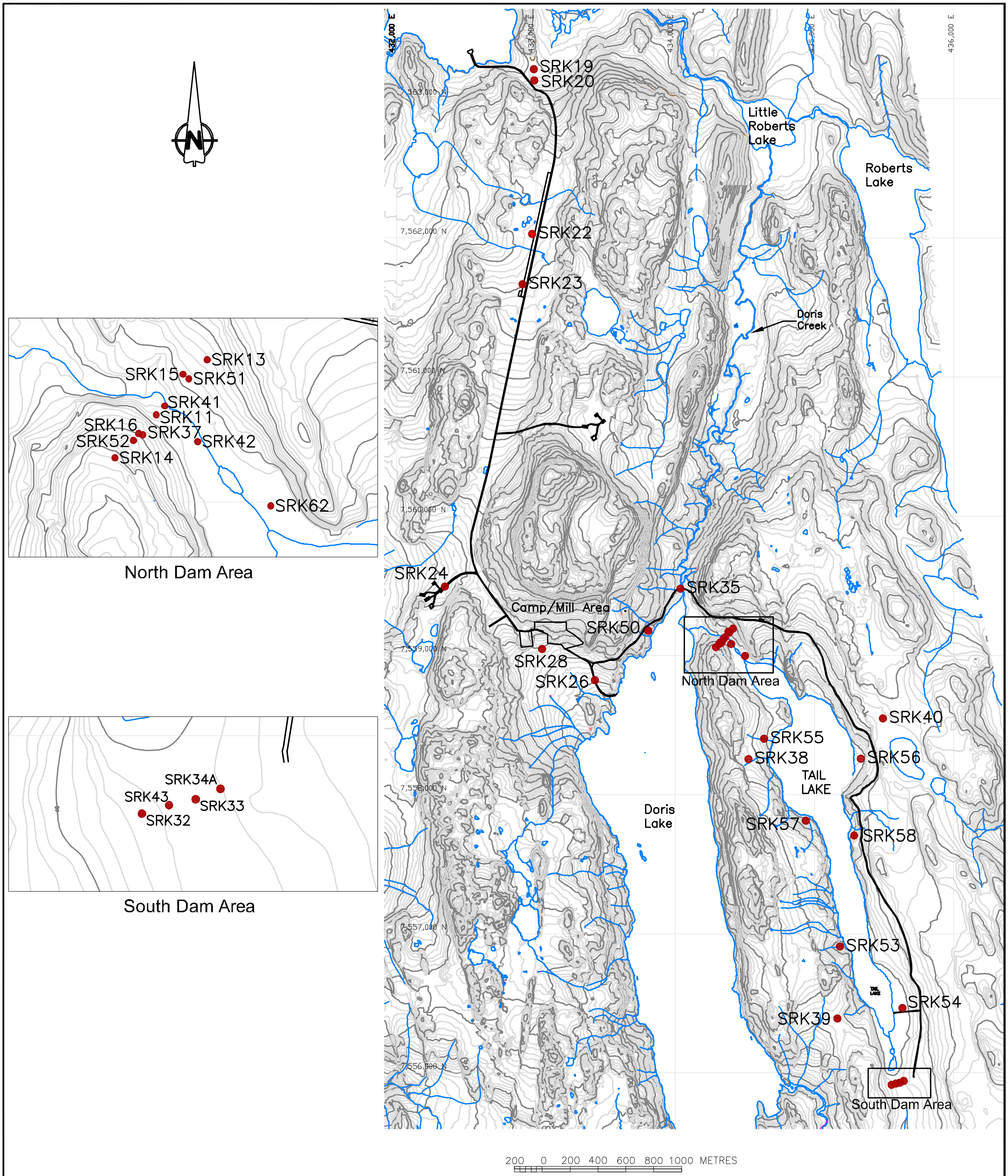
Doris North Project
Winter 2006 Geotechnical Field Investigations

Winter 2006 Drillhole Locations

SRK JOB NO.: 1CM014.008
FILE NAME: Autocad–2006\Phase 111 Jetty Design\ SITE–MAP.dwg

Doris North Project

DATE:	APPROVED:	FIGURE:
Aug. 2006	EMR	1



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Engineers and Scientists
Vancouver

MIRAMAR HOPE BAY LIMITED

Doris North Project
Winter 2006 Geotechnical Field Investigations

Thermistor Locations

SRK JOB NO.: 1CM014.008
FILE NAME: Site–Plan–2006_Thermistors.dwg

Doris North Project

DATE: Aug. 2006
APPROVED: EMR
FIGURE: 2

Appendix A

Drill Logs

SRK Consulting
Engineers and Scientists

BOREHOLE LOG

PROJECT: Doris North - Detailed Infrastructure Design
LOCATION: Tail Lake South Dam
FILE No: HOPE BAY (1CM014.008)
BORING DATE: 2006-05-03 **TO** 2006-05-04
DIP: 90.00 **AZIMUTH:**
COORDINATES: 7555903.00 N 435623.00 E **DATUM:**

BOREHOLE: SRK06-01
PAGE: 1 **OF** 2
DRILL TYPE: Triple tube (HQ)
DRILL: Boyles 17
CASING: None

SAMPLE CONDITION

☒	Remoulded
▨	Undisturbed
■	Lost
◧	Core

TYPE OF SAMPLER

DC	Diamond core barrel
GS	Grab sample
SS	Split spoon

LABORATORY AND IN SITU TESTS

C	Consolidation	Ku	Thermal conductivity Unfrozen (W / m°C)
D	Bulk density (kg/m3)	Kf	Thermal conductivity Frozen (W / m°C)
Dr	Specific gravity	PS	Particle size analysis
Ksat	Saturated hydraulic cond. (cm/s)		

STRATIGRAPHY

ELEVATION - m / DEPTH - m	DESCRIPTION
0.00	
0.00 -0.20	Organic Peat, Nbn. 10% ice.
0.20 -0.26	Ice + Clayey SILT, Tan, 80%
0.26 -1.55	Sandy SILT, Tan, with traces of organic (Peaty), Nbe, 20% ice
1.55 -1.60	Ice + Sandy SILT, very small amount of organic, 80% ice
1.60 -2.80	Sandy SILT with traces of clay, Vr, 6cm thick ice lenses at 2m, 2.4m, 2.6m. 50% ice. Tested brine salinity at 9% and -9c
2.80 -4.40	Sandy SILT with traces of clay, Vr, grey, 30% ice
4.40 -5.40	Snady SILT with trace of clay, trace of organic at 4.4-4.6m, Vs, 30% ice
5.40 -5.70	Sandy SILT, grey, medium toughness, poorly graded, Nbn, 5% ice
5.70 -6.40	Sandy SILT as above
6.40 -8.00	Silty CLAY, dark grey, dense, Nbn, 5% ice. From 7.5-8m no ice content observed due to thawing. Water pump failed during the drilling unawarely, core came out hot and thawed. Tried to repair pump after notification
8.00 -9.50	Silty CLAY, dark grey, very dense, No ice contect observed due to another pump failure
9.50 -10.70	Silty CLAY, grey, dense, Vr, 10% ice
10.70 -12.50	Silty CLAY, grey with black layers, Vr, 15-20% ice, 2.8m thick ice lenses, clear ice
12.50 -13.80	Silty CLAY, dark grey, dense, Vs, 10% ice. Tested brine at 12% salinity and -8c
13.80 -15.50	LOSS
15.50 -17.00	Silty CLAY, dark grey, dense, Vr, 10% ice. 1.8m thick ice lenses
17.00 -18.50	Silty CLAY, dark grey, dense, Vr, 10% ice, 2.5m thick ice lenses
18.50 -19.25	Silty CLAY, dark grey, dense, Vr, 5% ice, 1.5cm thick ice lenses
19.25 -19.27	ICE
	Silty CLAY, grey Silty CLAY, not frozened, firm

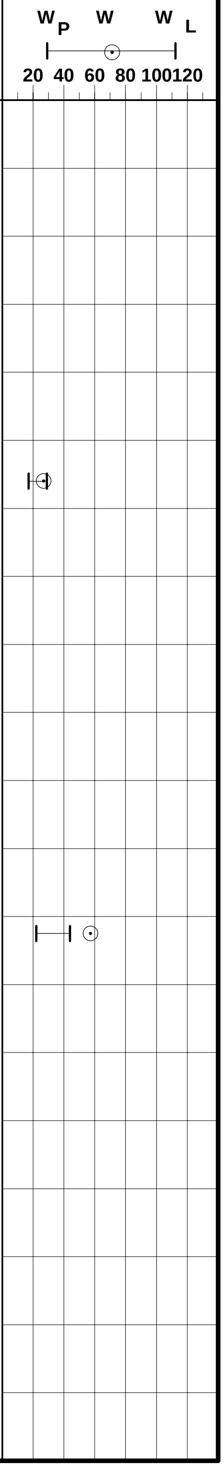
SAMPLES

TYPE AND NUMBER	CONDITION	RECOVERY %	N or RQD
SRK06-01-01	☒	100	0
SRK06-01-02	☒	100	0
SRK06-01-03	☒	100	0
SRK06-01-04	☒	100	0
SRK06-01-05	☒	100	0
SRK06-01-06	☒	100	0
SRK06-01-07	☒	100	0
SRK06-01-08	☒	100	0
SRK06-01-09	☒	100	0

WELL DETAILS & WATER LEVEL - m

LABORATORY and IN SITU TESTS

WATER CONTENT and LIMITS (%)



DEPTH - ft: 5 10 15 20 25 30 35 40 45 50 55 60 65

DEPTH - m: 1 2 3 4 5 6 7 8 9 10 11 12 13 14 15 16 17 18 19

Z:\06 REFERENCE MATERIAL\S\geotec.log\templates\loglog SRK m23 HopeBay 20m.sty PLOTTED: 2006-08-30 16:19hrs

SRK Consulting
Engineers and Scientists

BOREHOLE LOG

PROJECT: Doris North - Detailed Infrastructure Design
LOCATION: Tail Lake South Dam
FILE No: HOPE BAY (1CM014.008)
BORING DATE: 2006-05-03 **TO** 2006-05-04
DIP: 90.00 **AZIMUTH:**
COORDINATES: 7555903.00 N 435623.00 E **DATUM:**

BOREHOLE: SRK06-01
PAGE: 2 **OF** 2
DRILL TYPE: Triple tube (HQ)
DRILL: Boyles 17
CASING: None

SAMPLE CONDITION		TYPE OF SAMPLER		LABORATORY AND IN SITU TESTS			
☒	Remoulded	DC	Diamond core barrel	C	Consolidation	Ku	Thermal conductivity Unfrozen (W / m°C)
▨	Undisturbed	GS	Grab sample	D	Bulk density (kg/m3)	Kf	Thermal conductivity Frozen (W / m°C)
■	Lost	SS	Split spoon	Dr	Specific gravity	PS	Particle size analysis
◧	Core			Ksat	Saturated hydraulic cond. (cm/s)		

DEPTH - ft	DEPTH - m	WELL DETAILS & WATER LEVEL - m	STRATIGRAPHY: ELEVATION - m / DEPTH - m	STRATIGRAPHY: DESCRIPTION	STRATIGRAPHY: SYMBOL	SAMPLES: TYPE AND NUMBER	SAMPLES: CONDITION	SAMPLES: RECOVERY %	SAMPLES: N or RQD	LABORATORY and IN SITU TESTS	WATER CONTENT and LIMITS (%) W_P W W_L 20 40 60 80 100 120
			-19.29	Ice, layered, cloudy							
70	21 22		19.29 -19.32 19.32 -19.60 19.60 -19.65	Silty CLAY, not frozened, firm consistency, medium toughness, medium plasticity, no dilatency, moist, lensed structure. Soil has mottle appearance		SRK06-01-10	☒	100	0		
			19.65 -20.30	Ice, as above							
75	23		20.30	Silty CLAY, as above							
			-21.80	LOSS							
			21.80 -22.03	Silty CLAY, as above							
	24		22.03	Silty CLAY, as above, Vr, 5% ice, clear ice							
80	25		-22.30 22.30 -22.60 22.60	Medium SAND, well graded, Nbe to occasional 5mm clear ice lenses, reddish brown, silt/clay, grey							
85	26		-23.30 23.30 -24.30 24.30 -25.00	Silty SAND, well graded, soft consistency, laminated structure, moist, low toughness, low plasticity, Nbe. Lots of excess water when thaw in hand.							
	27		25.00	Medium SAND, as above, washed from hole as water pressure had to be increased to drill bedrock							
90	28			BEDROCK, grabbo.							
				EOH							
95	29										
	30										
100	31										
105	32										
	33										
110	34										
115	35										
	36										
120	37										
125	38										
	39										
130											

SRK Consulting
Engineers and Scientists

BOREHOLE LOG

PROJECT: Doris North - Detailed Infrastructure Design
LOCATION: Tail Lake South Dam
FILE No: HOPE BAY (1CM014.008)
BORING DATE: 2006-05-06 **TO** 2006-05-07
DIP: 90.00 **AZIMUTH:**
COORDINATES: 7555939.00 N 435590.00 E **DATUM:**

BOREHOLE: SRK06-02
PAGE: 1 **OF** 2
DRILL TYPE: Triple tube (HQ)
DRILL:
CASING: None

SAMPLE CONDITION
- ☒ Remoulded
- ▨ Undisturbed
- ■ Lost
- ◧ Core

TYPE OF SAMPLER
- DC Diamond core barrel
- GS Grab sample
- SS Split spoon

LABORATORY AND IN SITU TESTS

C	Consolidation	Ku	Thermal conductivity Unfrozen (W / m°C)
D	Bulk density (kg/m3)	Kf	Thermal conductivity Frozen (W / m°C)
Dr	Specific gravity	PS	Particle size analysis
Ksat	Saturated hydraulic cond. (cm/s)		

STRATIGRAPHY

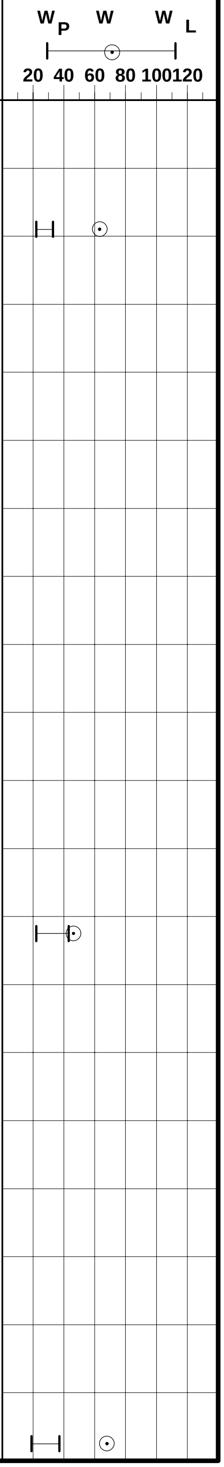
ELEVATION - m / DEPTH - m	DESCRIPTION
0.00	
0.00 -0.10	Sandy CLAY, high organic content, topsoil/peat, Nbn, 10% ice, dark brown
0.10 -0.50	Sandy SAND, with a trace of clay. Nbn, less than 5% ice, with some organic, dense
0.50 -0.60	Ice + SAND, poorly graded, 90% ice
0.60 -2.00	Sandy SILT, poorly graded, light grey, Vs, 40% ice, ice lenses approx. 1cm thick.
2.00 -2.20	Sandy SILT, as above
2.20 -2.50	Ice + sandy SILT, poorly graded, light grey, 80% ice
2.50 -3.50	LOSS LOSS
3.50 -3.70	Ice + sandy SILT, poorly graded, light grey, 70% ice
3.70 -4.40	Silty SAND, brown, poorly graded, Nbn, 5% ice
4.40 -5.00	Sandy SILT, light grey, poorly graded, low plasticity, no ice observed due to thawign of drilling. Thaw due to plugging of bit from muddy return
5.00 -5.25	LOSS
5.25 -5.50	Silty SAND, brown, poorly graded, Nbn, 5% ice
5.50 -5.75	Silty CLAY, dark grey very dence Nbn, 5% ice
5.75 -6.00	LOSS
6.00 -7.25	Silty CLAY, dark grey, very dense, Vr, 10% ice, 1cm thick ice lenses
7.25 -8.75	Silty CLAY, dark grey, very dense, Vr, 20% ice, 2cm thick ice lenses
8.75 -10.25	Silty CLAY, dark grey, very dense, Vr, 20% ice, 3cm thick ice lenses
10.25 -10.55	LOSS
10.55 -11.75	Silty CLAY, dark grey, very dense, Vr, 20% ice, 2cm thick ice lenses
11.75 -12.10	Silty CLAY, Vr, 5% ice, laminated. Tested brine at 8% and -2c
12.10 -12.30	Silty CLAY, Vr, 70% ice, brown ice (possibly due to drilling
12.30 -12.50	Silty CLAY, grey, with black laminated interbeddings of 2-4cm thick, Vr, 15% ice, clear ice without inclusions, ice lenses less 2.5cm thick. Soil is between 2-4cm thick.
12.50 -13.20	LOSS
13.20 -14.00	Silty CLAY, as above
14.00 -14.65	LOSS
14.65 -15.50	Slity CLAY, as above, Nbe with occasional Vx, 5% ice
15.50 -16.88	LOSS
16.88 -17.00	Silty CLAY, as above
17.00 -18.35	LOSS. Tested brine at -6c and 7% salinity
18.35 -18.50	Silty CLAY, as above
18.50 -19.00	LOSS
19.00	Silty fine SAND with clay, soft

SAMPLES

TYPE AND NUMBER	CONDITION	RECOVERY %	N or RQD
SRK06-02-01	☒	100	0
SRK06-02-02	☒	0	0
SRK06-02-03	☒	0	0
SRK06-02-04	☒	0	0
SRK06-02-05	☒	0	0
SRK06-02-06	☒	100	0
SRK06-02-07	☒	0	0
SRK06-02-08	☒	0	0
SRK06-02-09	☒	0	0
SRK06-02-10	☒	100	0

LABORATORY and IN SITU TESTS

WATER CONTENT and LIMITS (%)



SRK Consulting
Engineers and Scientists

BOREHOLE LOG

PROJECT: Doris North - Detailed Infrastructure Design
LOCATION: Tail Lake South Dam
FILE No: HOPE BAY (1CM014.008)
BORING DATE: 2006-05-06 **TO** 2006-05-07
DIP: 90.00 **AZIMUTH:**
COORDINATES: 7555939.00 N 435590.00 E **DATUM:**

BOREHOLE: SRK06-02
PAGE: 2 **OF** 2
DRILL TYPE: Triple tube (HQ)
DRILL:
CASING: None

SAMPLE CONDITION	**TYPE OF SAMPLER**	**LABORATORY AND IN SITU TESTS**	
☒ Remoulded	DC Diamond core barrel	C Consolidation	Ku Thermal conductivity Unfrozen (W / m°C)
▨ Undisturbed	GS Grab sample	D Bulk density (kg/m3)	Kf Thermal conductivity Frozen (W / m°C)
■ Lost	SS Split spoon	Dr Specific gravity	PS Particle size analysis
◨ Core		Ksat Saturated hydraulic cond. (cm/s)	

DEPTH - ft: 70, 75, 80, 85, 90, 95, 100, 105, 110, 115, 120, 125, 130

DEPTH - m: 21, 22, 23, 24, 25, 26, 27, 28, 29, 30, 31, 32, 33, 34, 35, 36, 37, 38, 39

ELEVATION - m / DEPTH - m	DESCRIPTION	TYPE AND NUMBER	CONDITION	RECOVERY %	N or RQD	WATER CONTENT and LIMITS (%) W
20.00	consistency, moist, laminiated fine sand less then 2.5cm, silty clay less then 2.5cm, low toughness, low plasticity, ice lenses approx. 2.5cm thick, with minor occasional Vx, total of 3% ice					
-21.05 21.05	LOSS					
-21.50 21.50	Clayey SILT, Vr, 30% ice					
	LOSS					
-24.50 24.50	Silty SAND, mottled. Vs, 7% ice, ice lenses less than 1cm thick. Tested brine at -1.8c and 8% salinity	SRK06-02-11	☒	0	0	
-25.70 25.70	Medium SAND, well graded to coarse sand, Vs, 7% ice, ice lenses less than 1cm thick, large cobble at bottom of the hole	SRK06-02-12	☒	0	0	
-26.00 26.00						
-26.55 26.55	Coarse SAND, well graded, Nbe, 7% ice					
-27.02 27.02	Fine to medium SAND with silty clay interbeds. Layers at 5-10cm thick Granite Boulders with coarse sand interbed. Well graded. Nbe, 7% ice	SRK06-02-13	☒	0	0	⊙
-29.00 29.00	LOSS	SRK06-02-14	☒	0	0	
-30.97 30.97	Coarse SAND, Nbe, with occasional 2mm thick ice layers, 7% ice					
-31.13 31.13	LOSS					
-34.73 34.73	BEDROCK, Qtz veined follow fractures at 30 degrees, slickensided surfaces, black discoloration on surface, undulations faces.					
-35.58 35.58	EOH					

WATER CONTENT and LIMITS (%): W_P W W_L — 20 40 60 80 100120

Z:\06 REFERENCE MATERIALS\geotec.log\templates\loglog SRK m23 HopeBay 20m.sty PLOTTED: 2006-08-30 16:19hrs

Appendix B

Laboratory Test Results

EBA Engineering Consultants Ltd.

MOISTURE CONTENT TEST RESULTS

Project: SRK 2006 Testing Services
Hope Bay Gold Project
BH No:
Project No.: 1780176
Date Tested: 1-Jun-06
Location: Hope Bay, NT
By: DKKS
Client: SRK Consulting

Test No.	SampleNo.	Depth(m)	Wet+Tare	Dry+Tare	Tare	% Moisture Content
SRK06-01-03	4150-3	N/A	660.9	522.8	12.2	27.0
SRK06-01-06	4150-6	N/A	441.9	285.4	12.5	57.3
SRK06-01-10	4150-10	N/A	707.9	559.7	12.3	27.1
SRK06-02-01	4150-11	N/A	518.4	322.1	12.2	63.3
SRK06-02-06	4150-16	N/A	586.1	405	14.2	46.3
SRK06-02-10	4150-20	N/A	826.9	497.8	13.9	68.0
SRK06-02-13	4150-23	N/A	903.8	750.3	13.9	20.8
SRK06-11-01	4150-25	N/A	705.0	593.4	12.4	19.2
SRK06-12-02	4150-27	N/A	270.2	189.5	12.1	45.5
SRK06-12-03	4150-28	N/A	640.9	441.3	12.4	46.5
SRK06-13-01	4150-29	N/A	292.4	207.0	12.2	43.8
SRK06-13-02	4150-30	N/A	701.6	474.3	12.5	49.2
SRK06-14-02	4150-32	N/A	242.8	173.1	12.2	43.3
SRK06-15-01	4150-34	N/A	288.9	229.8	12.3	27.2
SRK06-15-02	4150-35	N/A	603.9	425.1	12.3	43.3
SRK06-16-01	4150-37	N/A	654.0	479.5	12.2	37.3
SRK06-16-02	4150-38	N/A	176.8	108.6	12.1	70.7
SRK06-17-01	4150-40	N/A	793.9	662.5	13.9	20.3
SRK06-17-02	4150-41	N/A	693.0	514.5	13.9	35.7
SRK06-17-03	4150-42	N/A	262.5	186.8	12.4	43.4

EBA Engineering Consultants Ltd.

POREWATER SALINITY

Project:	Hope Bay Gold	Sample No.:	SRK06
Project No.:	0701-1780176	Date Tested:	06-06-28
Client:	Miramar Hope Bay Limited	Tested By:	KP

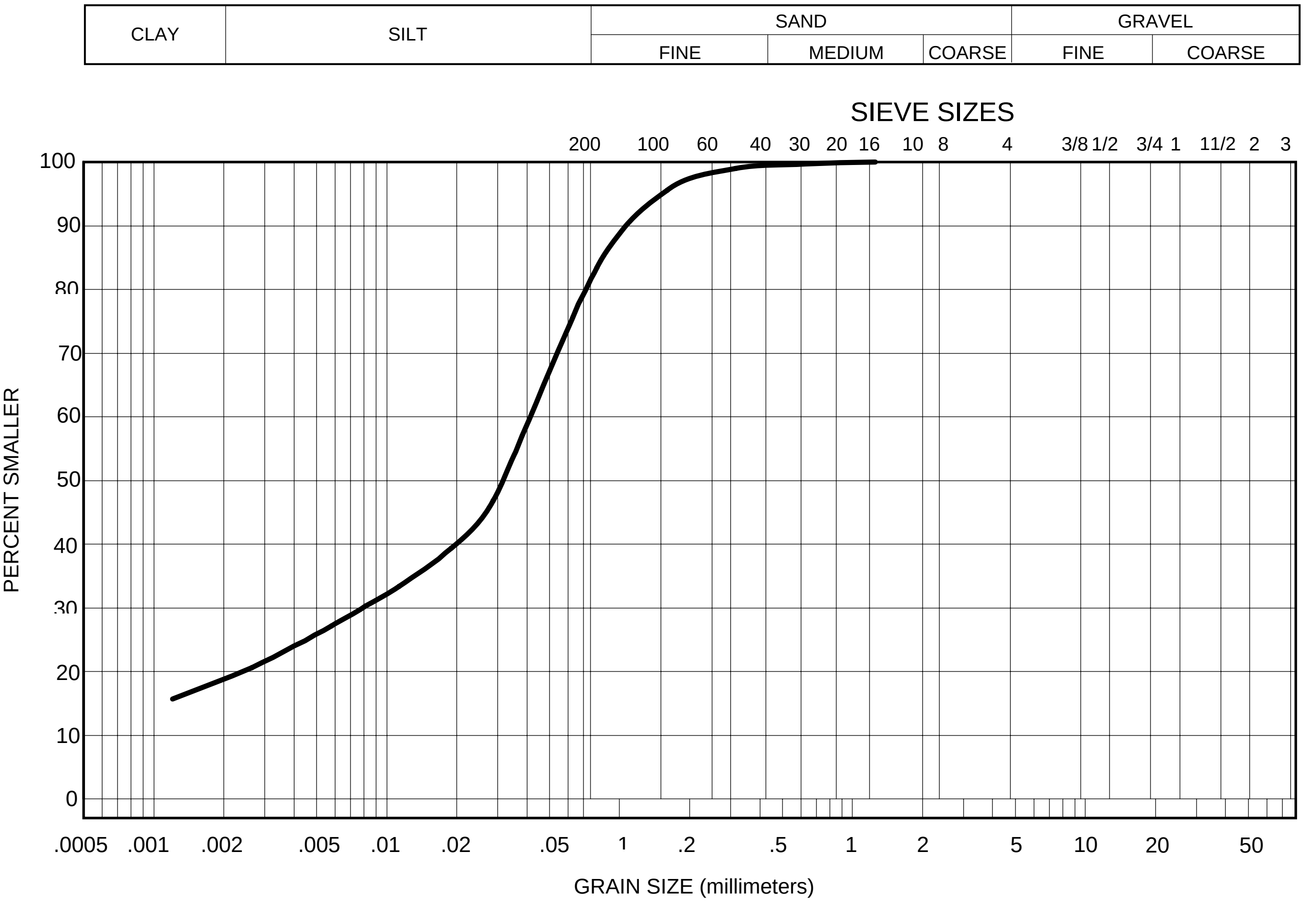
Sample Number	Depth (m)	Salinity (ppt)
01-03		67
01-06		44
01-10		67
02-01		6
02-06		60
02-10		80
02-13		86
11-01		89

EBA Engineering Consultants Ltd.

GRAIN SIZE DISTRIBUTION

Project: SRK 2006 Testing Services.Hope Bay Gold Project
Project Number: 1780176
Client: SRK Consulting Inc.
Attention: Mr. Alvin Tong
Date Tested: June 14-16, 2006
Sample ID: SRK06-01-03
Depth: n/a
Sample Number: n/a
Lab Number: 4150-3
Soil Description: SILT, some clay, some sand
Natural Moisture Content: 27.0%
Remarks: LL=29%, PL=17%, PI=12%

SIEVE	PERCENTAGE PASSING
2.5	
1.25	
0.630	100
0.315	99
0.160	96
0.08	83
0.029	47
0.019	39
0.0113	33
0.0081	30
0.0058	27
0.0026	21
0.0012	16



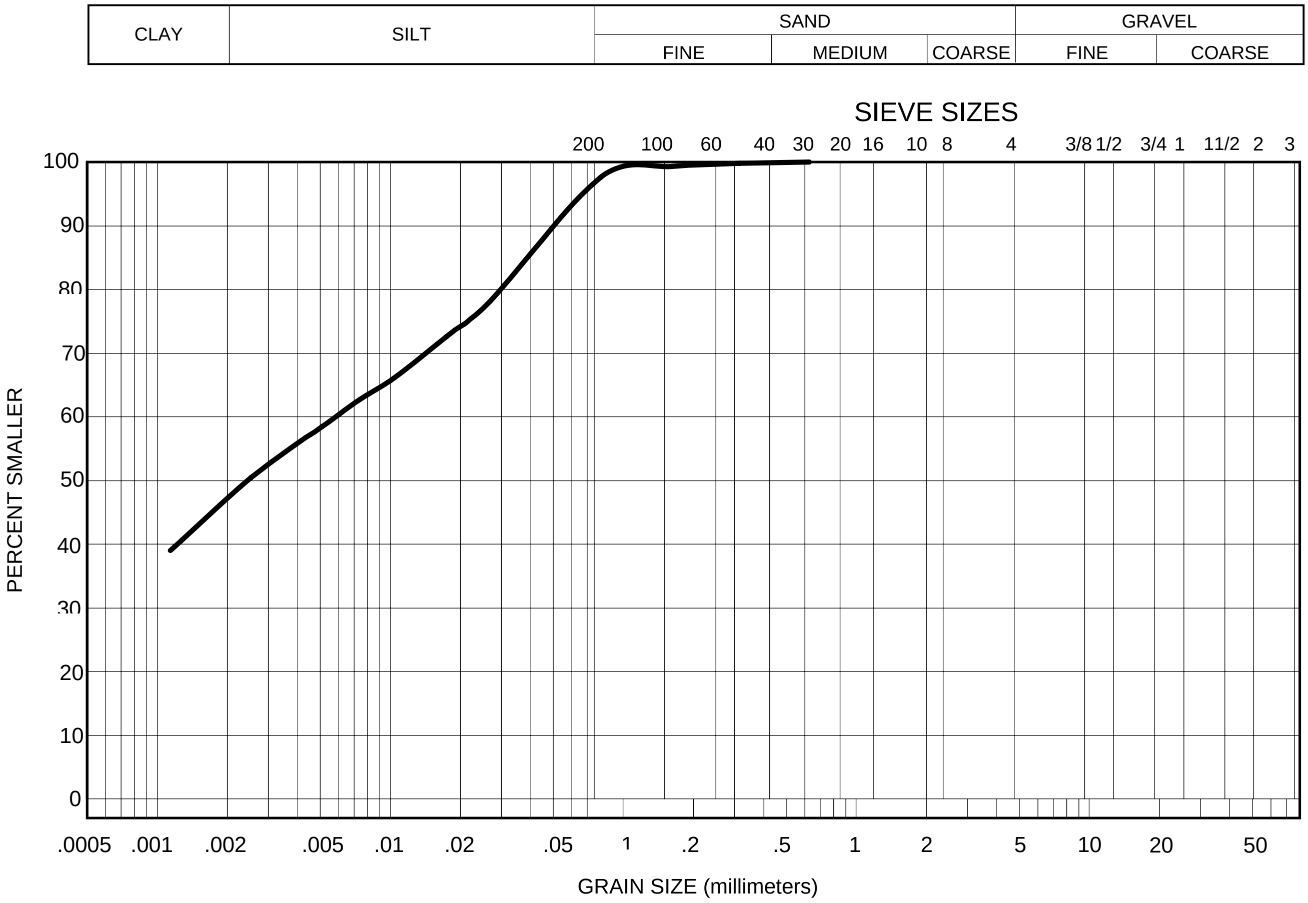
Reviewed By:____________________ P.Eng.

EBA Engineering Consultants Ltd.

GRAIN SIZE DISTRIBUTION

Project: SRK 2006 Testing Services.Hope Bay Gold Project
Project Number: 1780176
Client: SRK Consulting Inc.
Attention: Mr. Alvin Tong
Date Tested: June 14-16, 2006
Sample ID: SRK06-01-06
Depth: n/a
Sample Number: n/a
Lab Number: 4150-6
Soil Description: SILT and CLAY, trace sand
Natural Moisture Content: 57.3%
Remarks: LL=44%, PL=22%, PI=22%

SIEVE	PERCENTAGE PASSING
2.5	
1.25	
0.630	100
0.315	100
0.160	99
0.08	98
0.027	78
0.017	73
0.0104	66
0.0074	63
0.0051	58
0.0025	50
0.0011	39



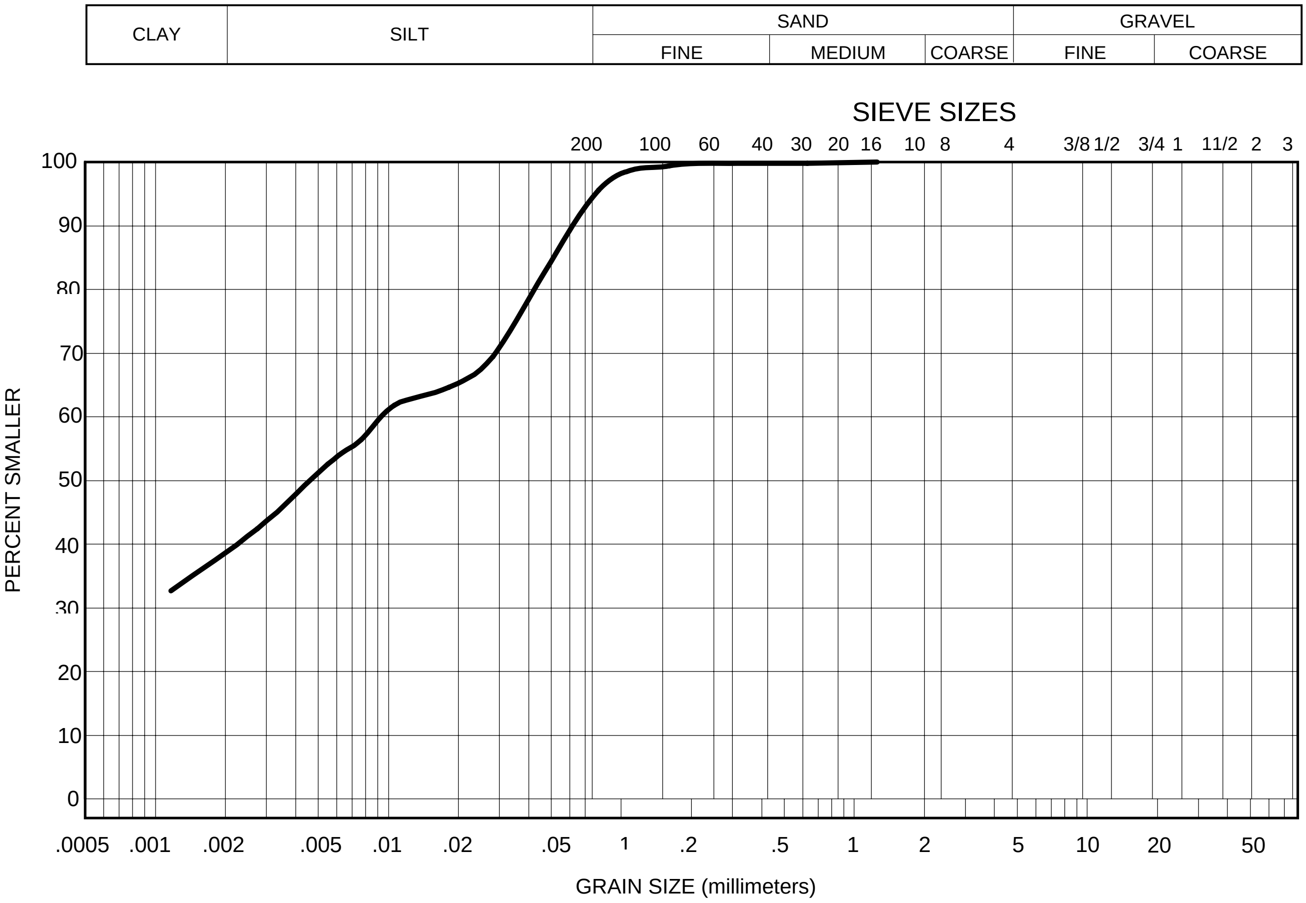
Reviewed By:__________________ P.Eng.

EBA Engineering Consultants Ltd.

GRAIN SIZE DISTRIBUTION

Project: SRK 2006 Testing Services.Hope Bay Gold Project
Project Number: 1780176
Client: SRK Consulting Inc.
Attention: Mr. Alvin Tong
Date Tested: June 14-16, 2006
Sample ID: SRK06-01-10
Depth: n/a
Sample Number: n/a
Lab Number: 4150-10
Soil Description: SILT and CLAY, trace sand
Natural Moisture Content: 27.1%
Remarks: LL=38%, PL=20%, PI=18%

SIEVE	PERCENTAGE PASSING
2.5	
1.25	
0.630	
0.315	100
0.160	99
0.08	96
0.028	69
0.018	65
0.0106	62
0.0077	56
0.0055	53
0.0027	42
0.0012	33

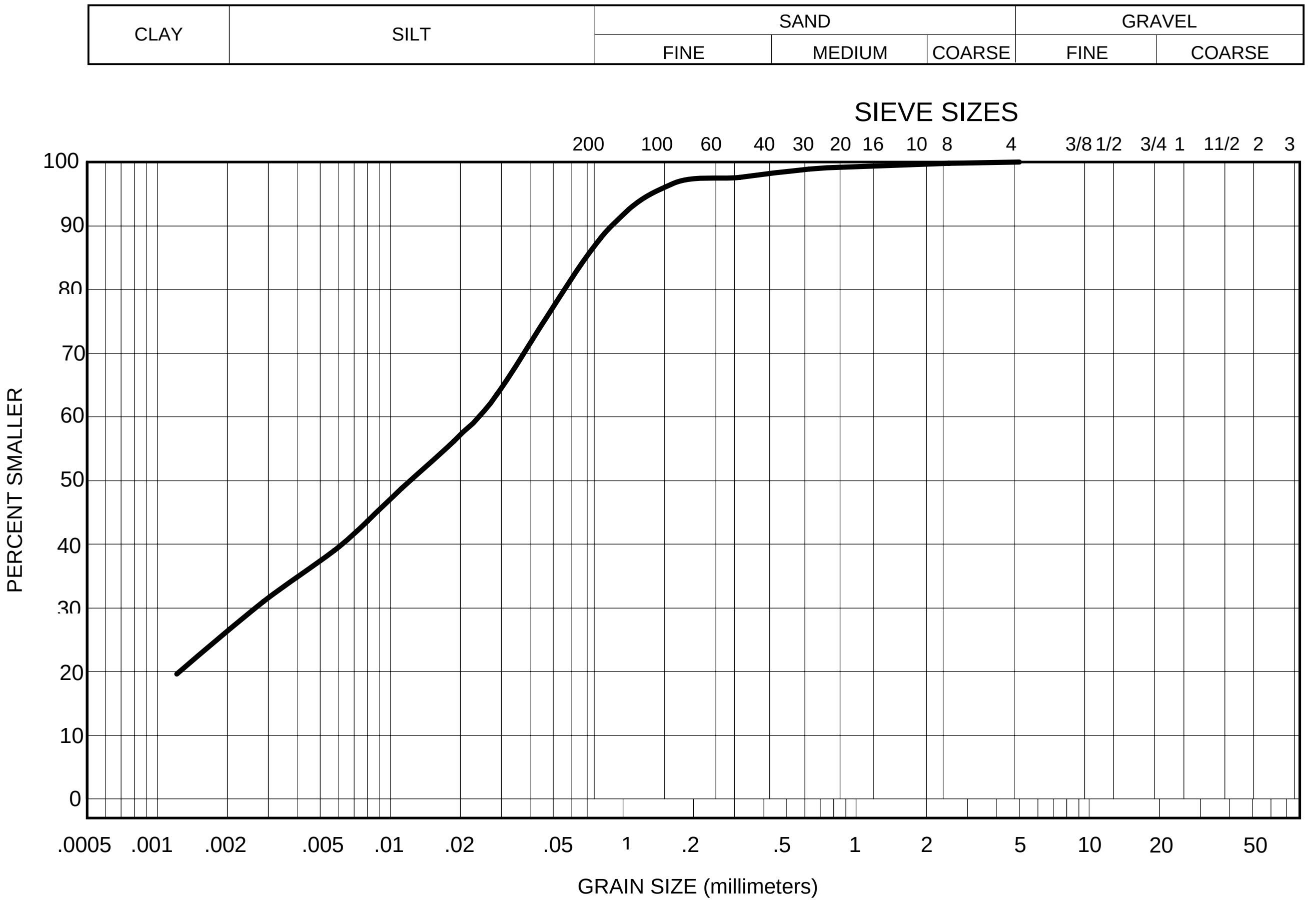


Reviewed By:___________________ P.Eng.

GRAIN SIZE DISTRIBUTION

Project: SRK 2006 Testing Services.Hope Bay Gold Project
Project Number: 1780176
Client: SRK Consulting Inc.
Attention: Mr. Alvin Tong
Date Tested: June 14-16, 2006
Sample ID: SRK06-02-01
Depth: n/a
Sample Number: n/a
Lab Number: 4150-11
Soil Description: SILT, clayey, some sand
Natural Moisture Content: 63.3%
Remarks: LL=33%, PL=22%, PI=11%

SIEVE, mm	PERCENTAGE PASSING
2.5	100
1.25	99
0.630	99
0.315	98
0.160	96
0.08	88
0.029	64
0.019	56
0.0111	49
0.0080	44
0.0058	39
0.0028	31
0.0012	20



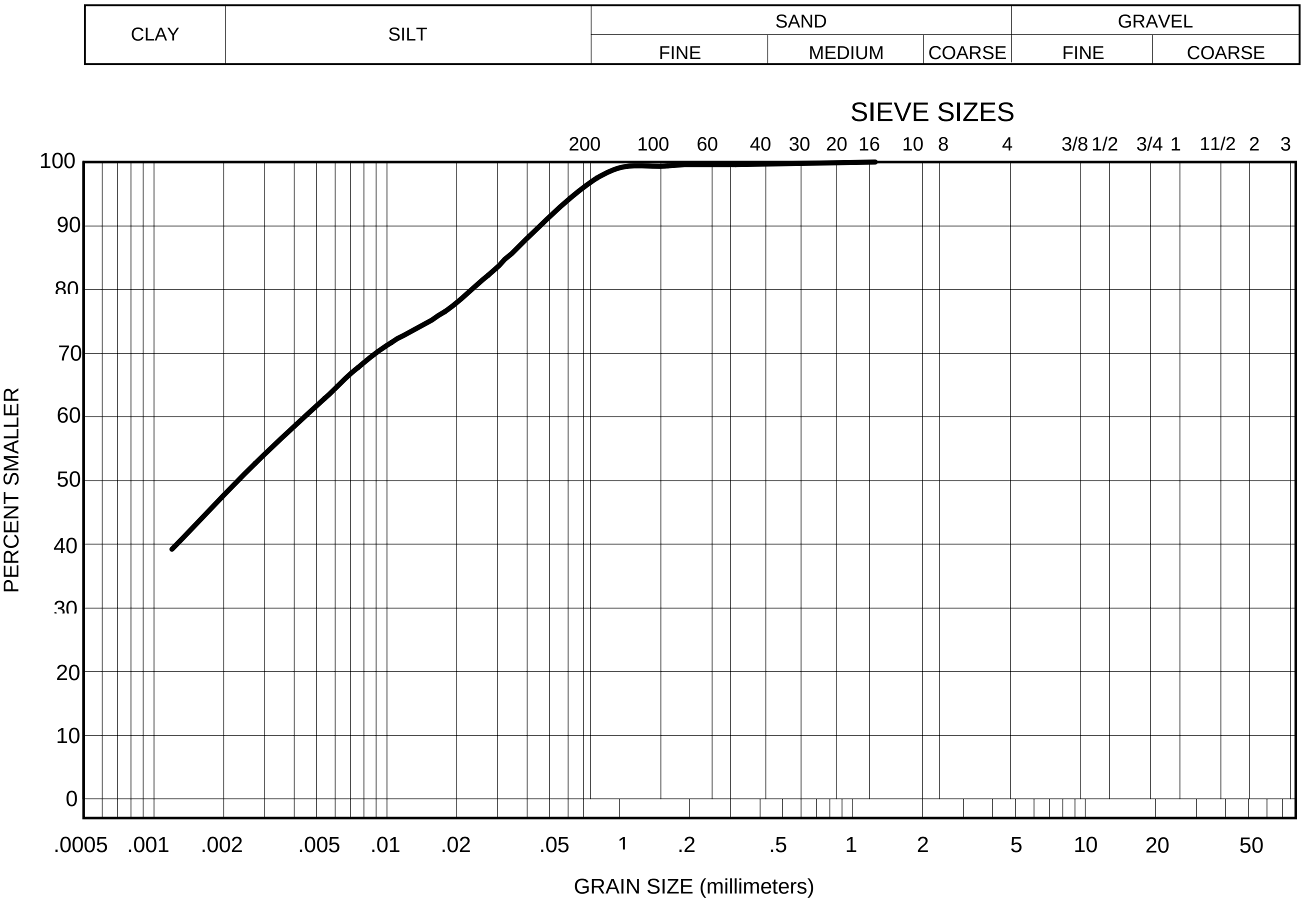
Reviewed By:__________________ P.Eng.

EBA Engineering Consultants Ltd.

GRAIN SIZE DISTRIBUTION

Project: SRK 2006 Testing Services.Hope Bay Gold Project
Project Number: 1780176
Client: SRK Consulting Inc.
Attention: Mr. Alvin Tong
Date Tested: June 13-14,15-16, 2006
Sample ID: SRK06-02-06
Depth: n/a
Sample Number: n/a
Lab Number: 4150-16
Soil Description: SILT and CLAY, trace sand
Natural Moisture Content: 46.3%
Remarks: LL=43%, PL=22%, PI=21%

SIEVE, mm	PERCENTAGE PASSING
2.5	
1.25	
0.630	
0.315	100
0.160	99
0.08	97
0.027	82
0.018	76
0.0105	72
0.0075	68
0.0054	63
0.0027	53
0.0012	39

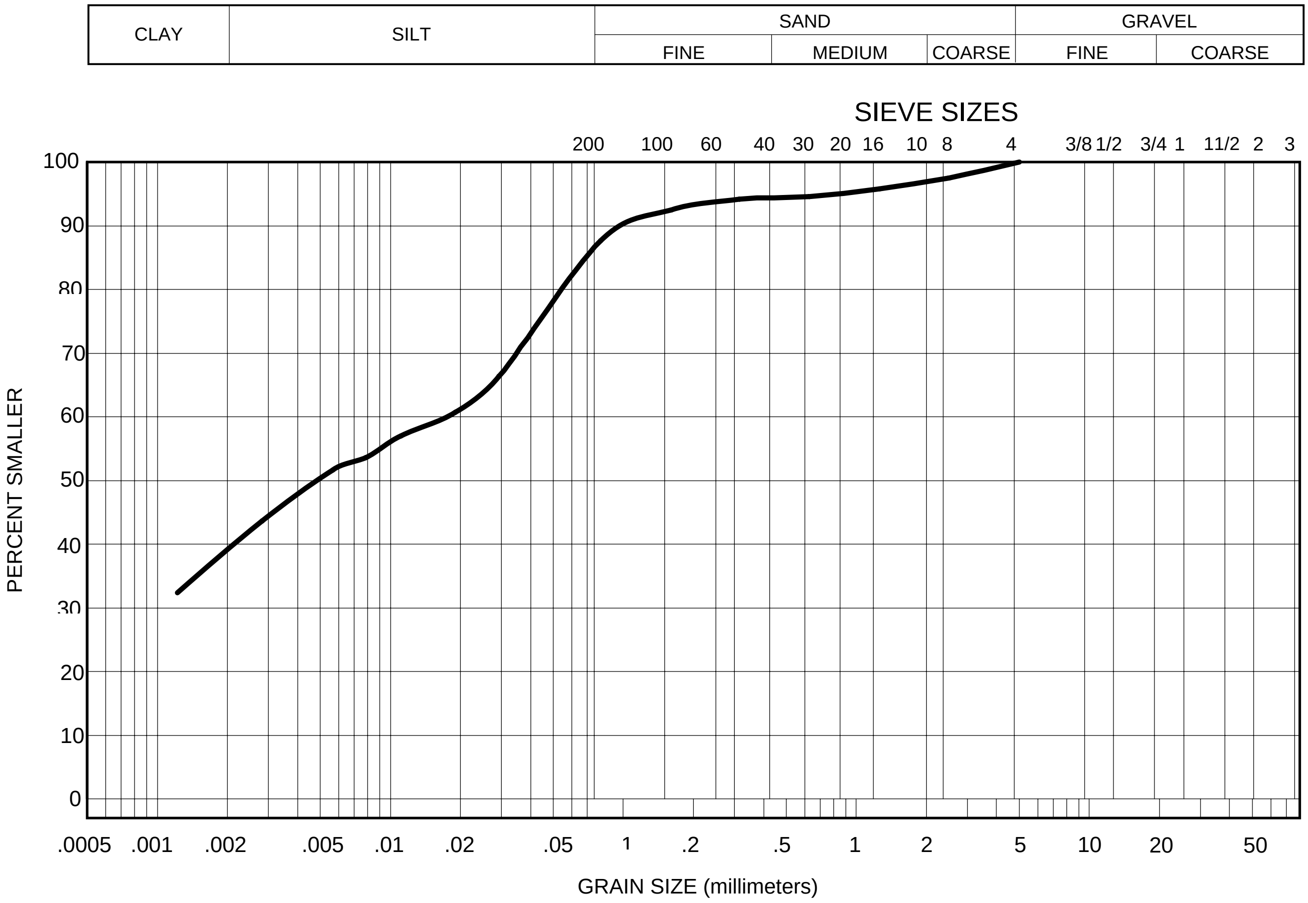


Reviewed By:__________________ P.Eng.

GRAIN SIZE DISTRIBUTION

Project: SRK 2006 Testing Services.Hope Bay Gold Project
Project Number: 1780176
Client: SRK Consulting Inc.
Attention: Mr. Alvin Tong
Date Tested: June 13-14,15-16, 2006
Sample ID: SRK06-02-10
Depth: n/a
Sample Number: n/a
Lab Number: 4150-20
Soil Description: SILT and CLAY, some sand
Natural Moisture Content: 68.0%
Remarks: LL=37%, PL=19%, PI=18%

SIEVE, mm	PERCENTAGE PASSING
5.0	100
2.5	97
1.25	96
0.630	95
0.315	94
0.160	92
0.08	88
0.029	66
0.019	61
0.0111	57
0.0078	54
0.0057	52
0.0028	44
0.0012	32



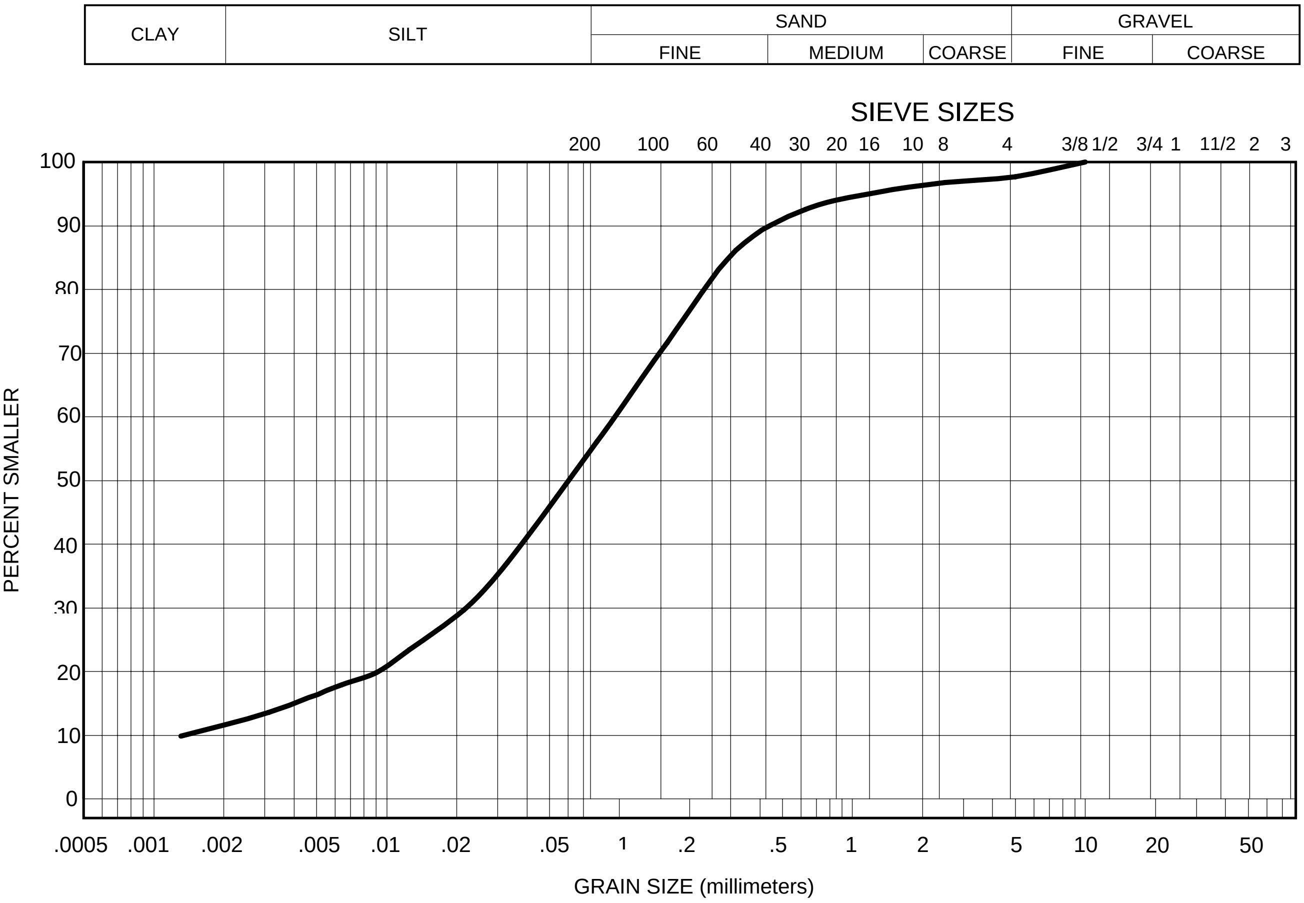
Reviewed By:__________________ P.Eng.

EBA Engineering Consultants Ltd.

GRAIN SIZE DISTRIBUTION

Project: SRK 2006 Testing Services.Hope Bay Gold Project
Project Number: 1780176
Client: SRK Consulting Inc.
Attention: Mr. Alvin Tong
Date Tested: June 13-14, 2006
Sample ID: SRK06-02-13
Depth: n/a
Sample Number: n/a
Lab Number: 4150-23
Soil Description: SILT and SAND, some clay, trace gravel
Natural Moisture Content: 20.8%
Remarks: N.P.

SIEVE, mm	PERCENTAGE PASSING
10.0	100
5.0	98
2.5	97
1.25	95
0.630	93
0.315	86
0.160	72
0.08	56
0.033	37
0.021	30
0.0125	23
0.0090	20
0.0064	18
0.0031	14
0.0013	10



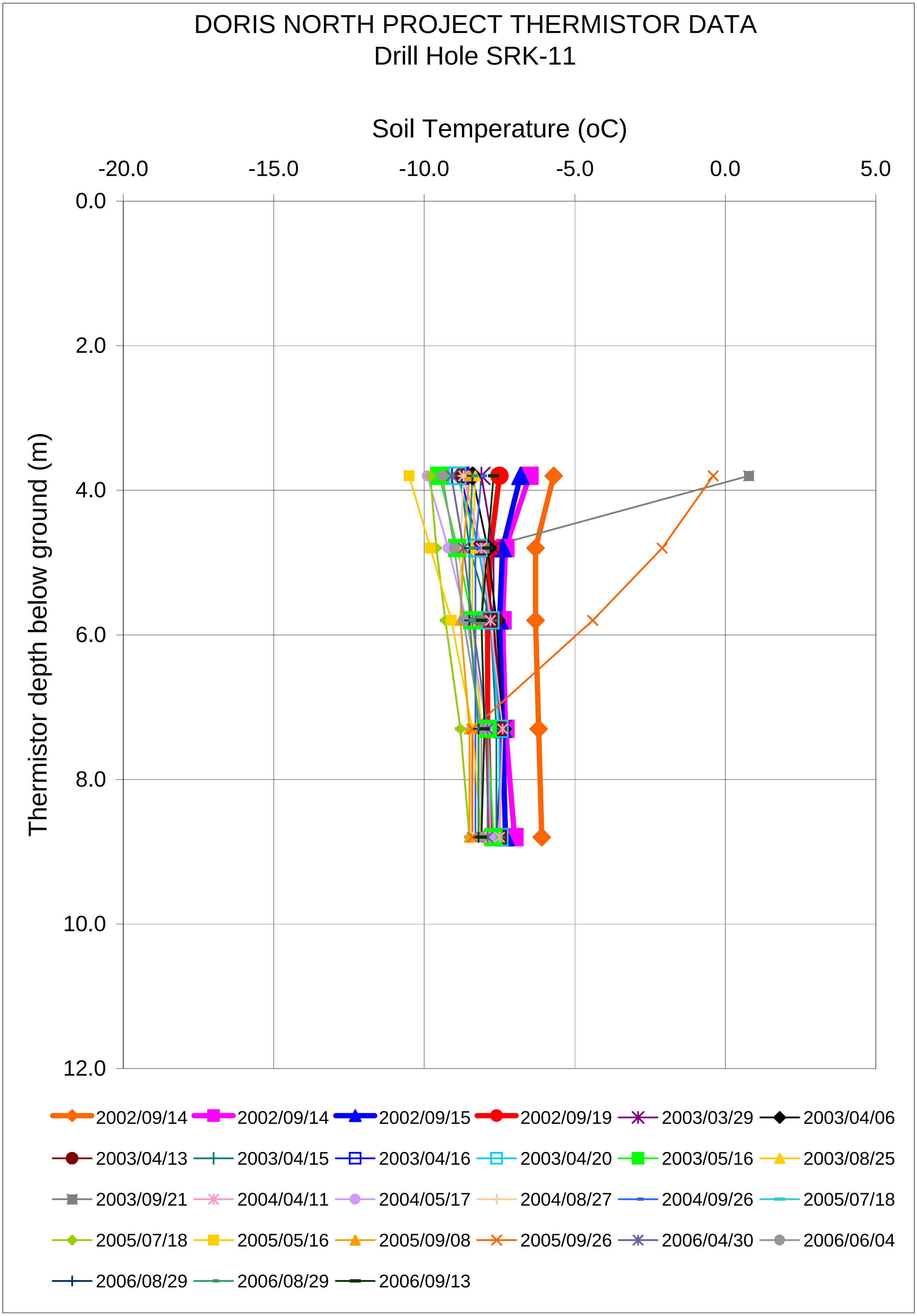
Reviewed By:___________________ P.Eng.

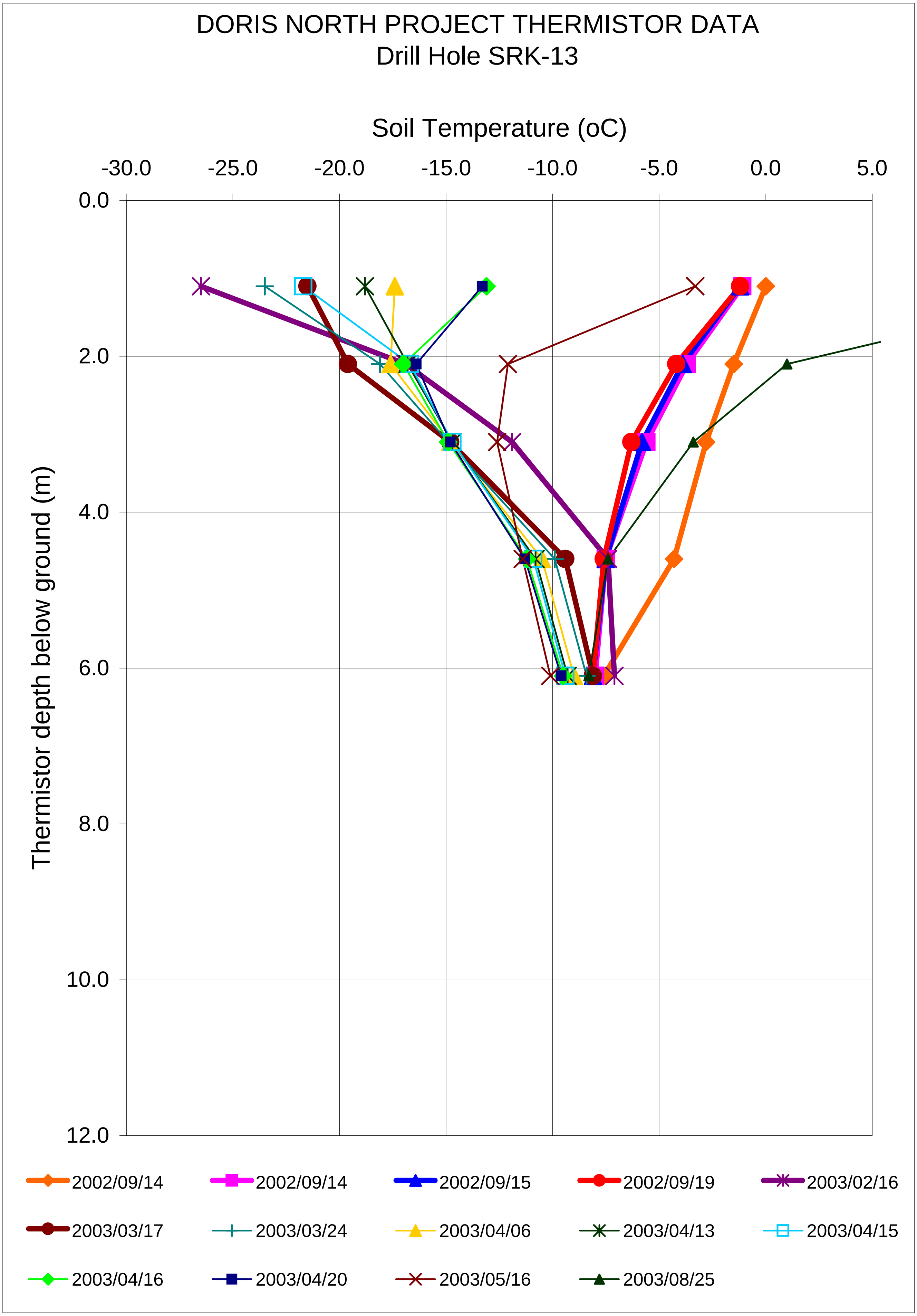
Appendix C

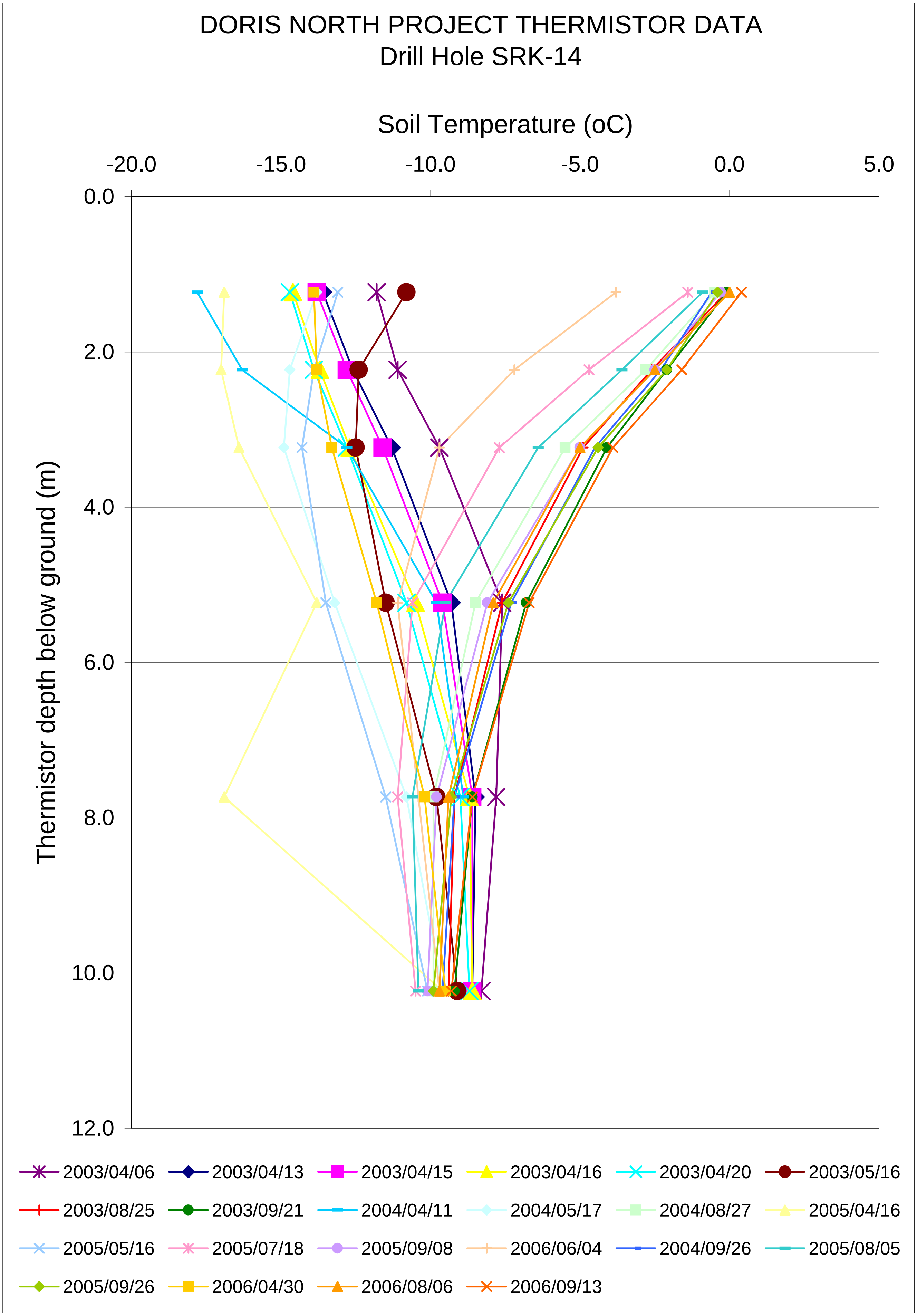
Thermistor String Data

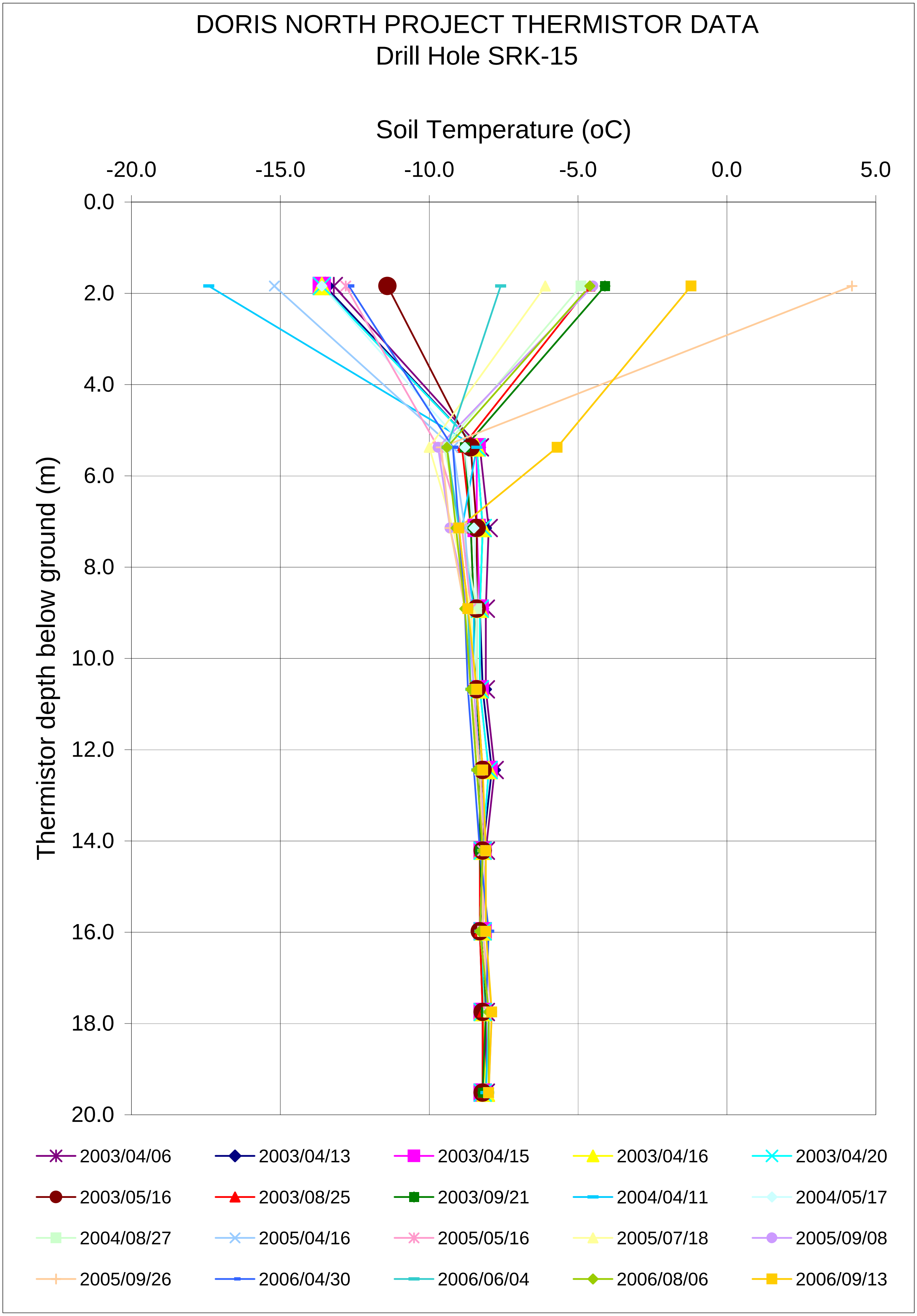
Appendix C-1

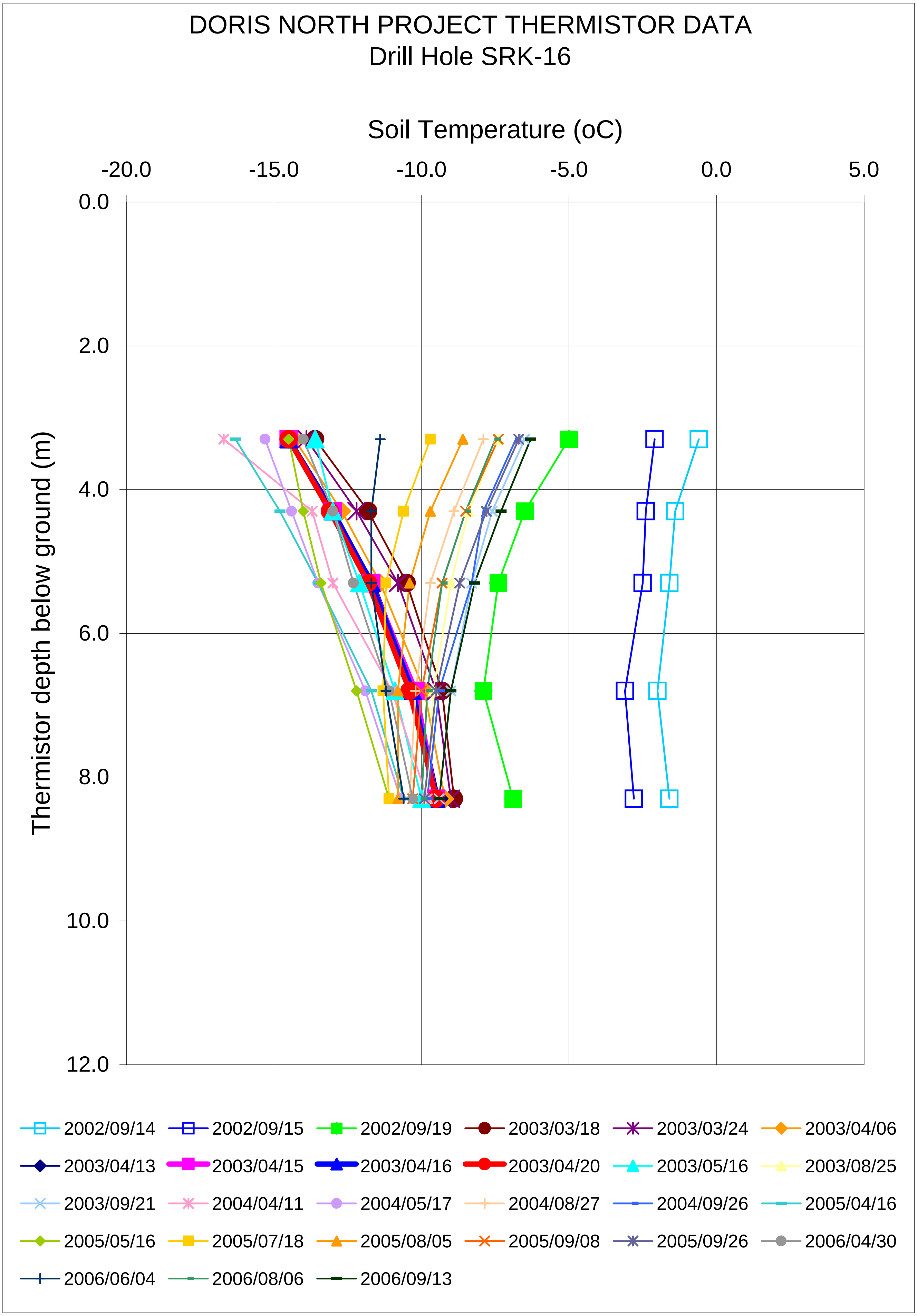
Thermistor String Data - Figures

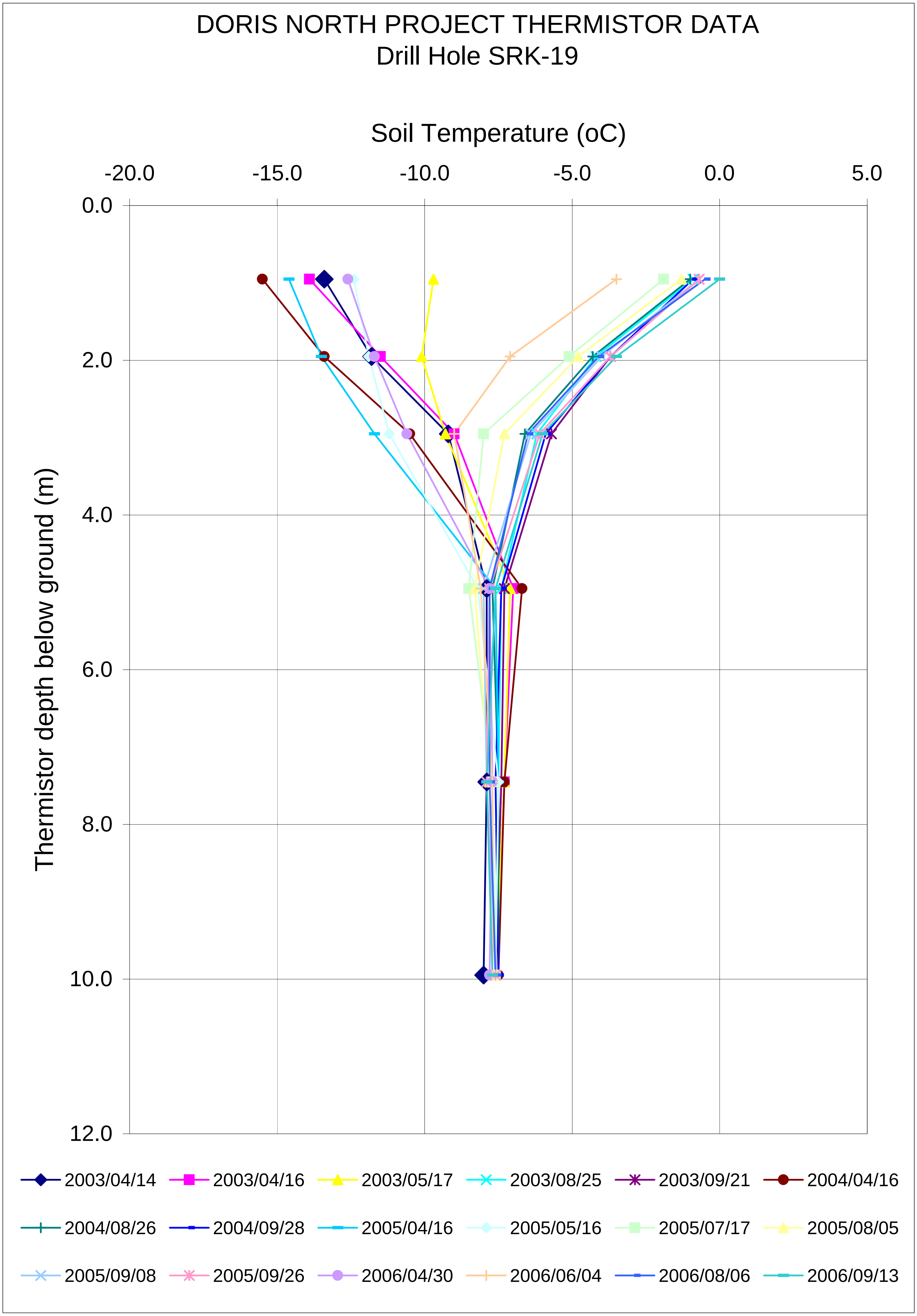

DORIS NORTH PROJECT THERMISTOR DATA
Drill Hole SRK-11
Soil Temperature (oC)
-20.0
-15.0
-10.0
-5.0
0.0
5.0
Thermistor depth below ground (m)
0.0
2.0
4.0
6.0
8.0
10.0
12.0
2002/09/14
2002/09/14
2002/09/15
2002/09/19
2003/03/29
2003/04/06
2003/04/13
2003/04/15
2003/04/16
2003/04/20
2003/05/16
2003/08/25
2003/09/21
2004/04/11
2004/05/17
2004/08/27
2004/09/26
2005/07/18
2005/07/18
2005/05/16
2005/09/08
2005/09/26
2006/04/30
2006/06/04
2006/08/29
2006/08/29
2006/09/13

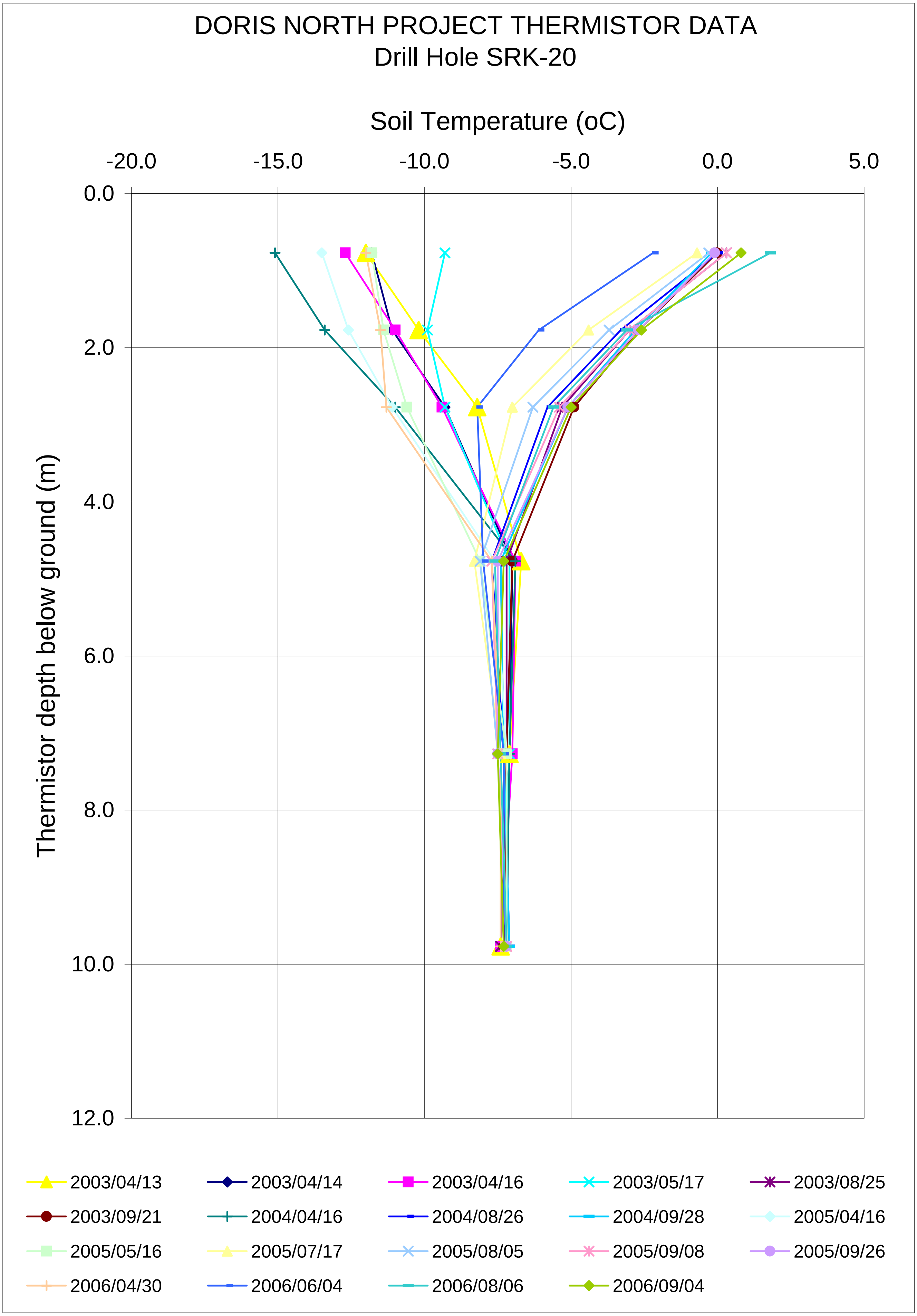

DORIS NORTH PROJECT THERMISTOR DATA
Drill Hole SRK-13
Soil Temperature (oC)
-30.0
-25.0
-20.0
-15.0
-10.0
-5.0
0.0
5.0
Thermistor depth below ground (m)
0.0
2.0
4.0
6.0
8.0
10.0
12.0
2002/09/14
2002/09/14
2002/09/15
2002/09/19
2003/02/16
2003/03/17
2003/03/24
2003/04/06
2003/04/13
2003/04/15
2003/04/16
2003/04/20
2003/05/16
2003/08/25

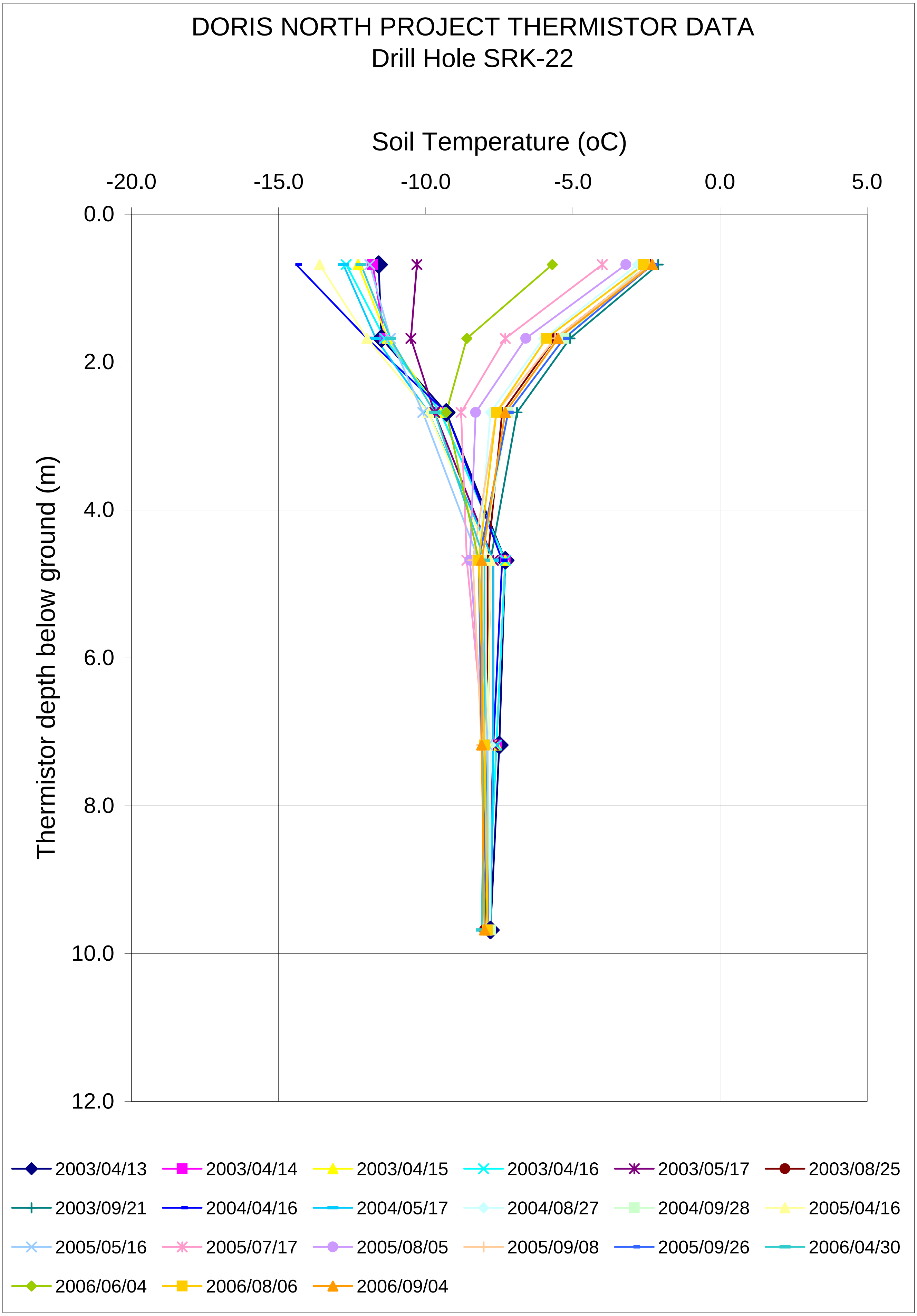

DORIS NORTH PROJECT THERMISTOR DATA
Drill Hole SRK-14
Soil Temperature (oC)
-20.0
-15.0
-10.0
-5.0
0.0
5.0
Thermistor depth below ground (m)
0.0
2.0
4.0
6.0
8.0
10.0
12.0
2003/04/06
2003/04/13
2003/04/15
2003/04/16
2003/04/20
2003/05/16
2003/08/25
2003/09/21
2004/04/11
2004/05/17
2004/08/27
2005/04/16
2005/05/16
2005/07/18
2005/09/08
2006/06/04
2004/09/26
2005/08/05
2005/09/26
2006/04/30
2006/08/06
2006/09/13


DORIS NORTH PROJECT THERMISTOR DATA
Drill Hole SRK-15
Soil Temperature (oC)
-20.0
-15.0
-10.0
-5.0
0.0
5.0
Thermistor depth below ground (m)
0.0
2.0
4.0
6.0
8.0
10.0
12.0
14.0
16.0
18.0
20.0
2003/04/06
2003/04/13
2003/04/15
2003/04/16
2003/04/20
2003/05/16
2003/08/25
2003/09/21
2004/04/11
2004/05/17
2004/08/27
2005/04/16
2005/05/16
2005/07/18
2005/09/08
2005/09/26
2006/04/30
2006/06/04
2006/08/06
2006/09/13

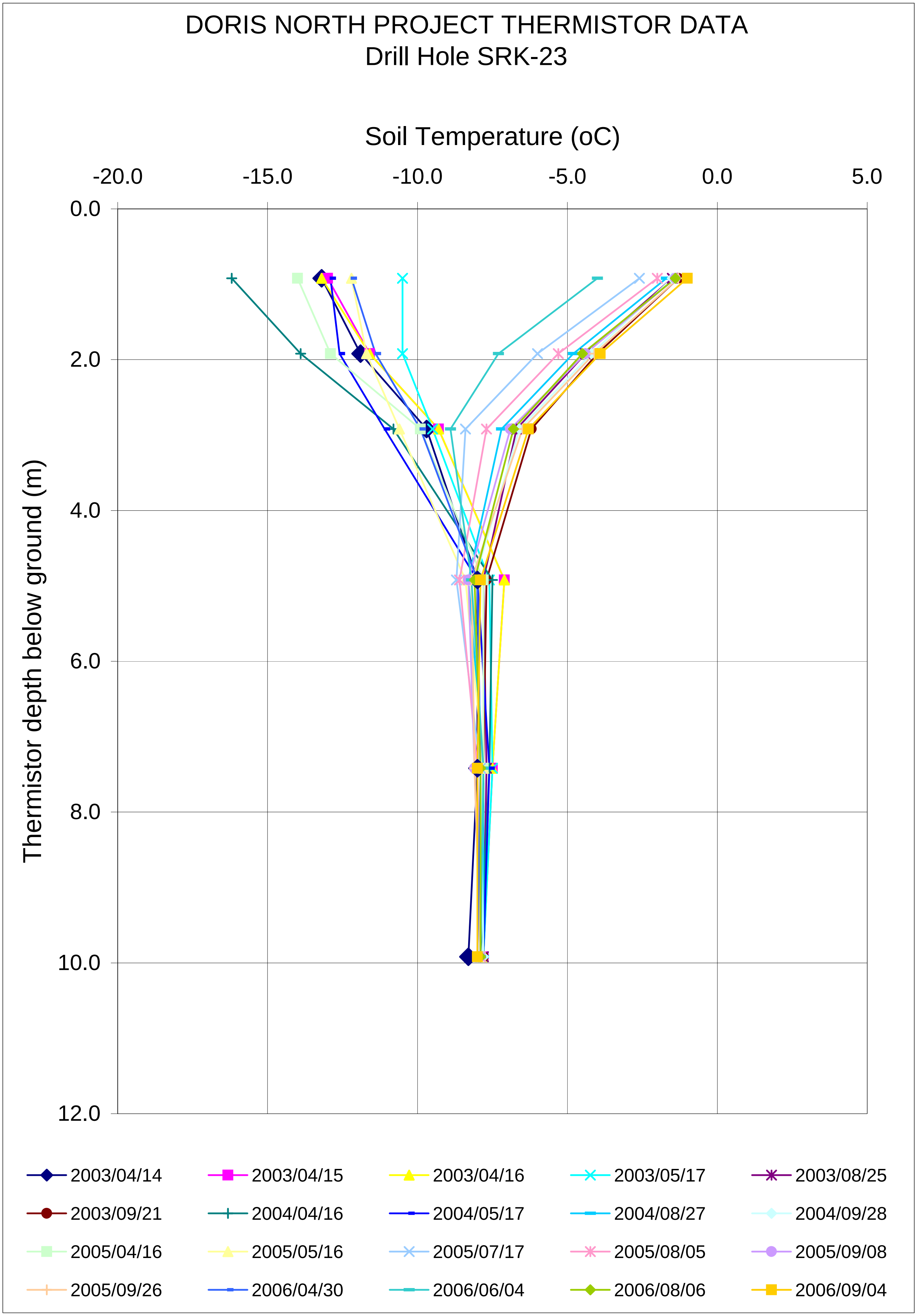

DORIS NORTH PROJECT THERMISTOR DATA
Drill Hole SRK-16
Soil Temperature (oC)
-20.0
-15.0
-10.0
-5.0
0.0
5.0
Thermistor depth below ground (m)
0.0
2.0
4.0
6.0
8.0
10.0
12.0
2002/09/14
2002/09/15
2002/09/19
2003/03/18
2003/03/24
2003/04/06
2003/04/13
2003/04/15
2003/04/16
2003/04/20
2003/05/16
2003/08/25
2003/09/21
2004/04/11
2004/05/17
2004/08/27
2004/09/26
2005/04/16
2005/05/16
2005/07/18
2005/08/05
2005/09/08
2005/09/26
2006/04/30
2006/06/04
2006/08/06
2006/09/13

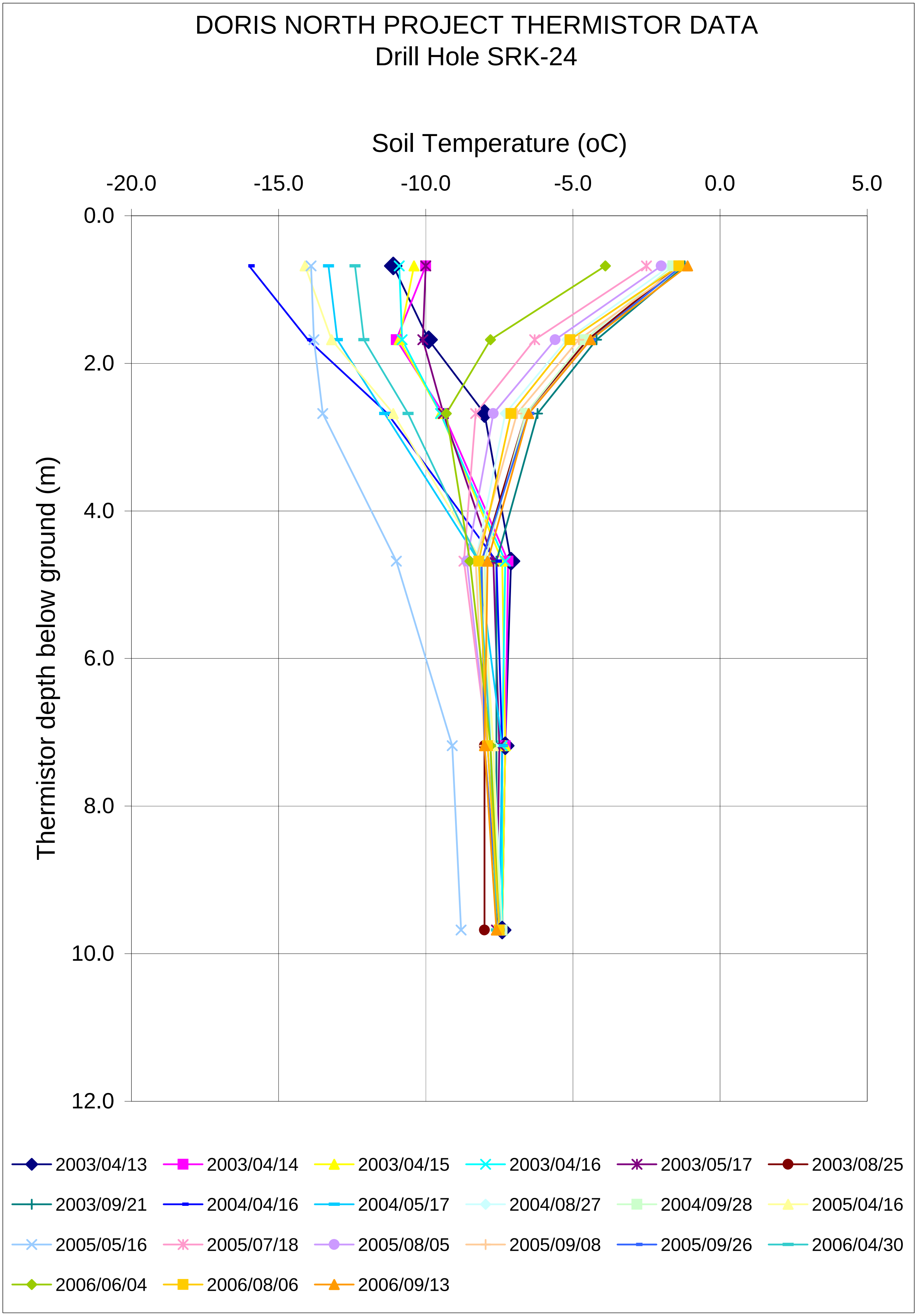

DORIS NORTH PROJECT THERMISTOR DATA
Drill Hole SRK-19
Soil Temperature (oC)
-20.0
-15.0
-10.0
-5.0
0.0
5.0
Thermistor depth below ground (m)
0.0
2.0
4.0
6.0
8.0
10.0
12.0
2003/04/14
2003/04/16
2003/05/17
2003/08/25
2003/09/21
2004/04/16
2004/08/26
2004/09/28
2005/04/16
2005/05/16
2005/07/17
2005/08/05
2005/09/08
2005/09/26
2006/04/30
2006/06/04
2006/08/06
2006/09/13

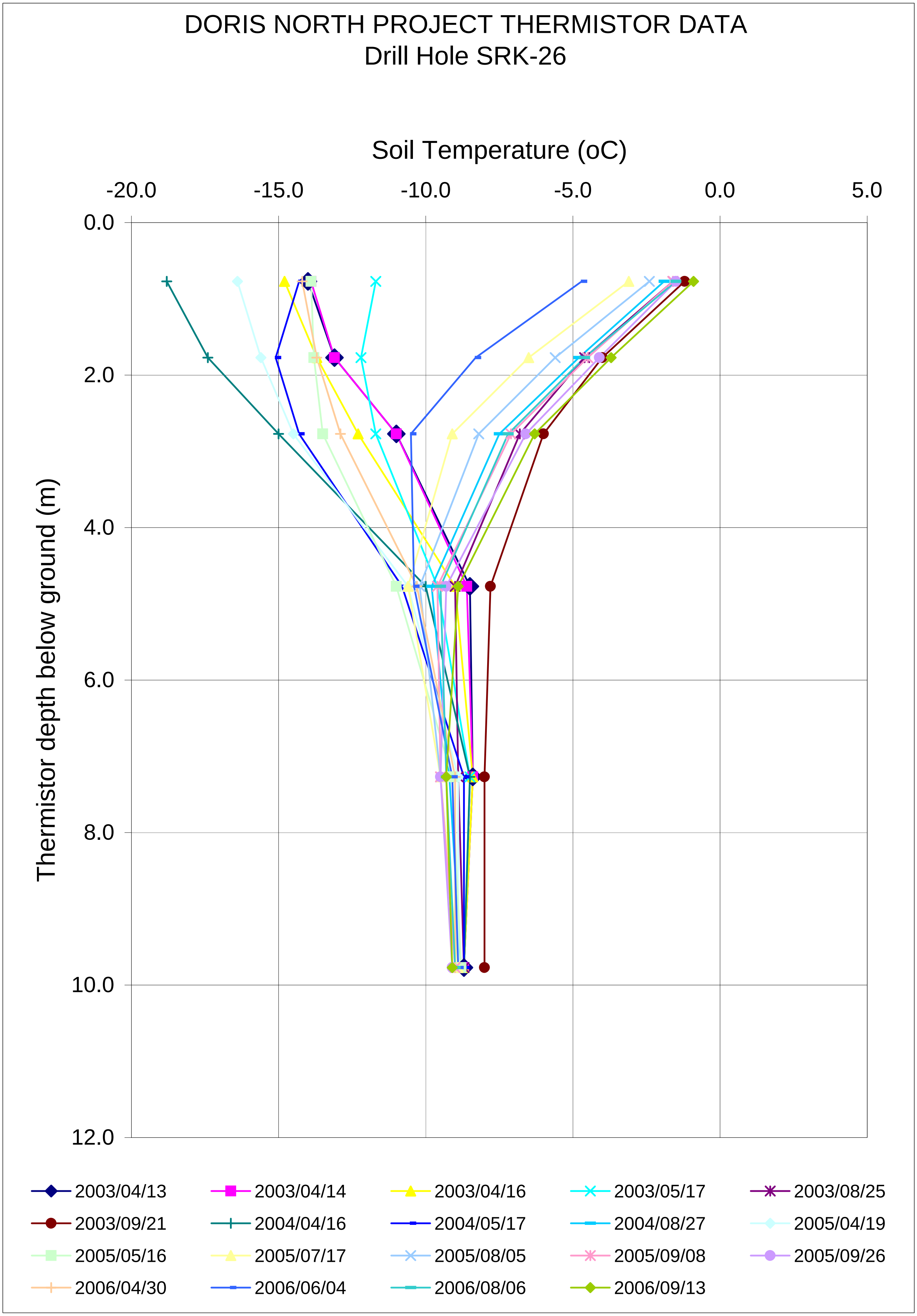

DORIS NORTH PROJECT THERMISTOR DATA
Drill Hole SRK-20



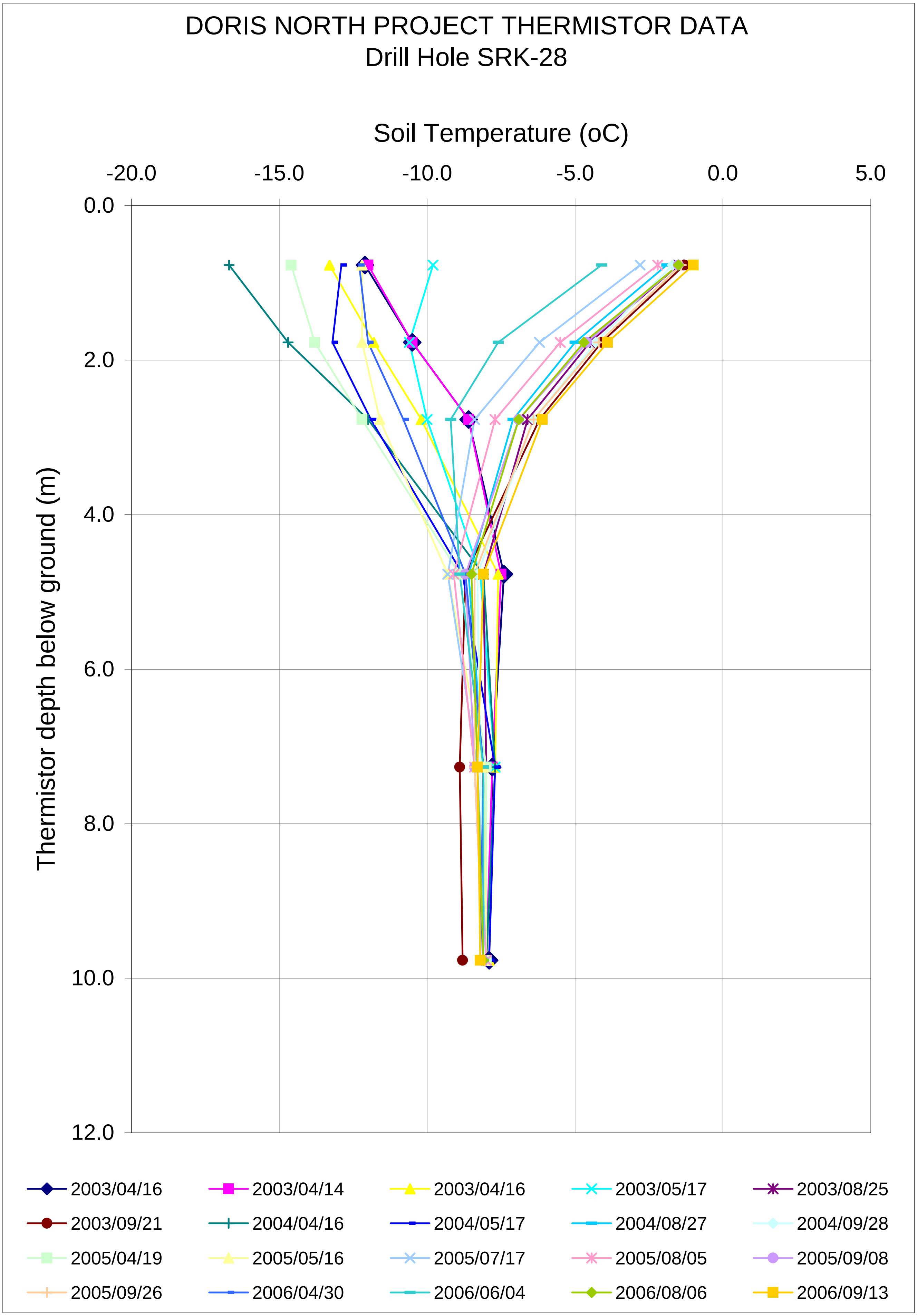
DORIS NORTH PROJECT THERMISTOR DATA
Drill Hole SRK-22

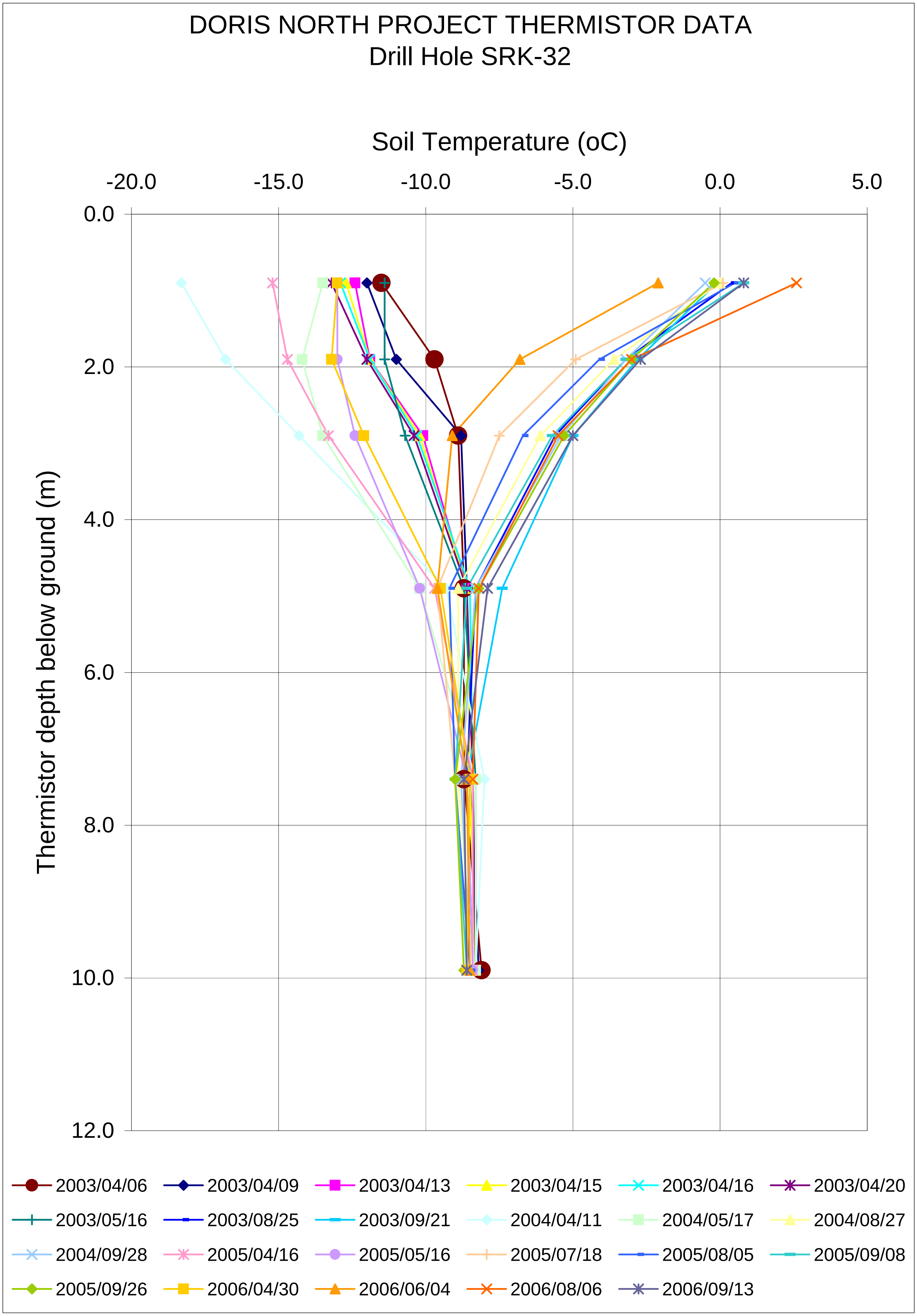

DORIS NORTH PROJECT THERMISTOR DATA
Drill Hole SRK-23
Soil Temperature (oC)
-20.0
-15.0
-10.0
-5.0
0.0
5.0
0.0
2.0
4.0
6.0
8.0
10.0
12.0
Thermistor depth below ground (m)
2003/04/14
2003/04/15
2003/04/16
2003/05/17
2003/08/25
2003/09/21
2004/04/16
2004/05/17
2004/08/27
2004/09/28
2005/04/16
2005/05/16
2005/07/17
2005/08/05
2005/09/08
2005/09/26
2006/04/30
2006/06/04
2006/08/06
2006/09/04


DORIS NORTH PROJECT THERMISTOR DATA
Drill Hole SRK-24
Soil Temperature (oC)
-20.0
-15.0
-10.0
-5.0
0.0
5.0
Thermistor depth below ground (m)
0.0
2.0
4.0
6.0
8.0
10.0
12.0
2003/04/13
2003/04/14
2003/04/15
2003/04/16
2003/05/17
2003/08/25
2003/09/21
2004/04/16
2004/05/17
2004/08/27
2004/09/28
2005/04/16
2005/05/16
2005/07/18
2005/08/05
2005/09/08
2005/09/26
2006/04/30
2006/06/04
2006/08/06
2006/09/13

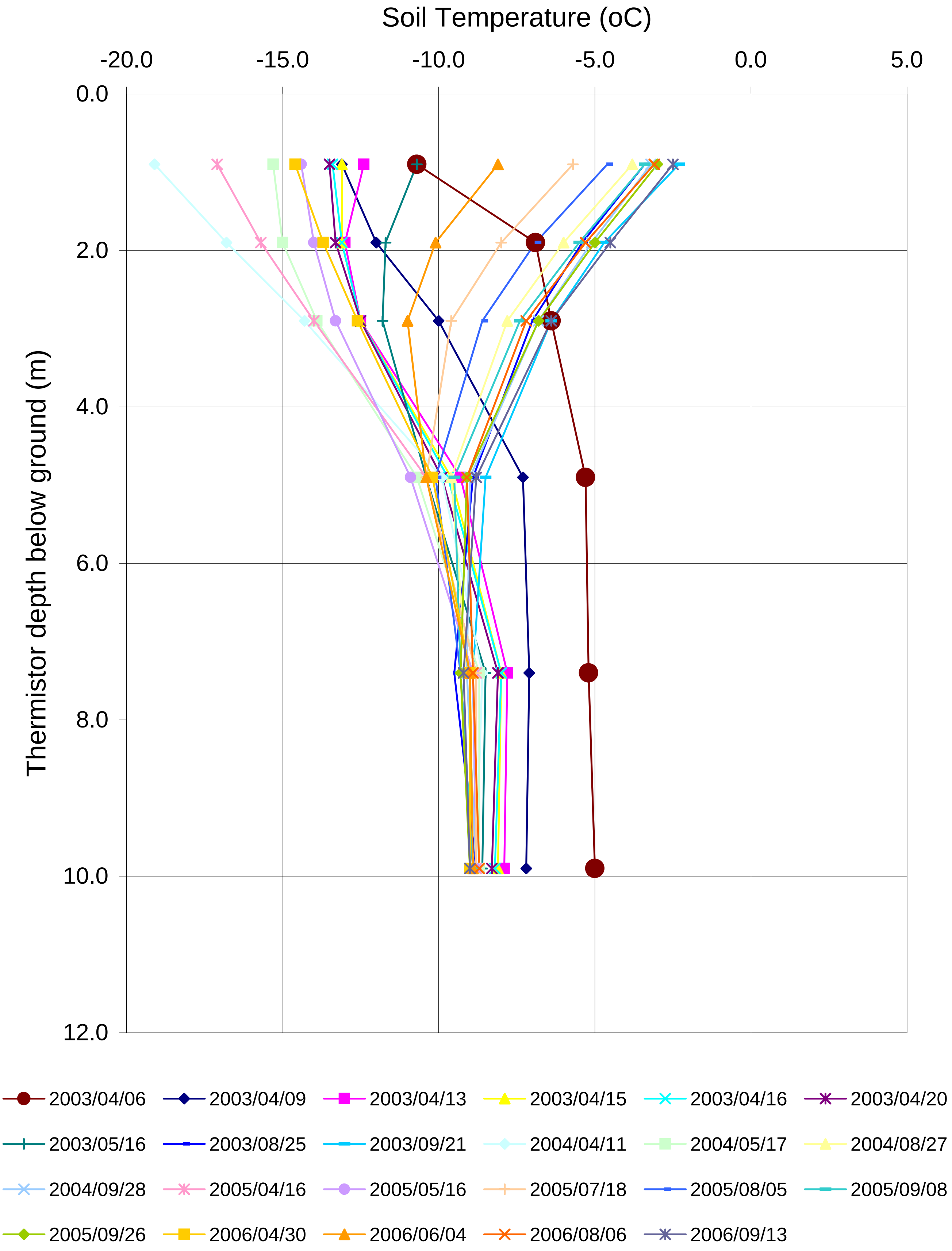


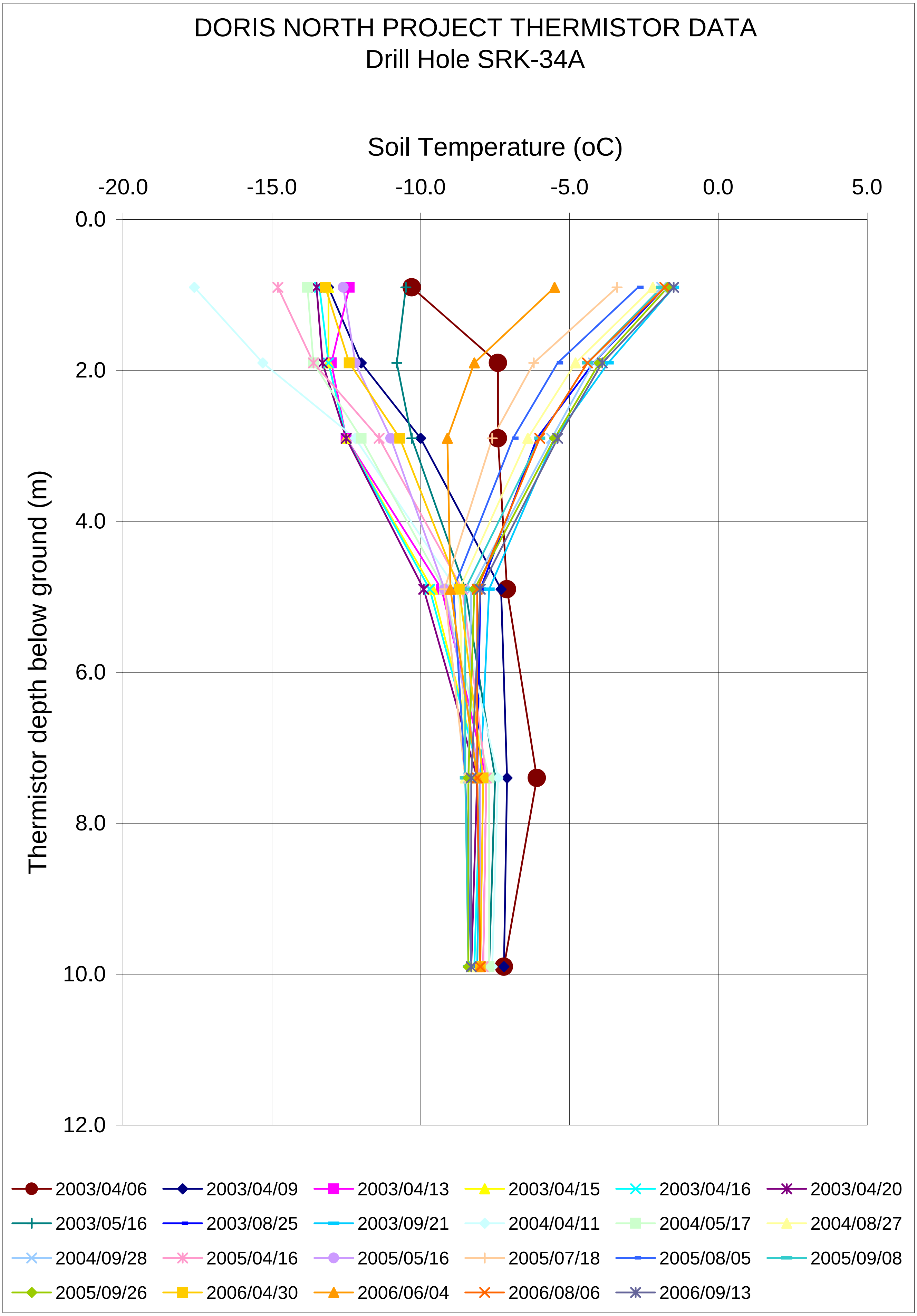
DORIS NORTH PROJECT THERMISTOR DATA
Drill Hole SRK-26
Soil Temperature (oC)
-20.0
-15.0
-10.0
-5.0
0.0
5.0
Thermistor depth below ground (m)
0.0
2.0
4.0
6.0
8.0
10.0
12.0
2003/04/13
2003/04/14
2003/04/16
2003/05/17
2003/08/25
2003/09/21
2004/04/16
2004/05/17
2004/08/27
2005/04/19
2005/05/16
2005/07/17
2005/08/05
2005/09/08
2005/09/26
2006/04/30
2006/06/04
2006/08/06
2006/09/13


DORIS NORTH PROJECT THERMISTOR DATA
Drill Hole SRK-28
Soil Temperature (oC)
-20.0
-15.0
-10.0
-5.0
0.0
5.0
Thermistor depth below ground (m)
0.0
2.0
4.0
6.0
8.0
10.0
12.0
2003/04/16
2003/04/14
2003/04/16
2003/05/17
2003/08/25
2003/09/21
2004/04/16
2004/05/17
2004/08/27
2004/09/28
2005/04/19
2005/05/16
2005/07/17
2005/08/05
2005/09/08
2005/09/26
2006/04/30
2006/06/04
2006/08/06
2006/09/13

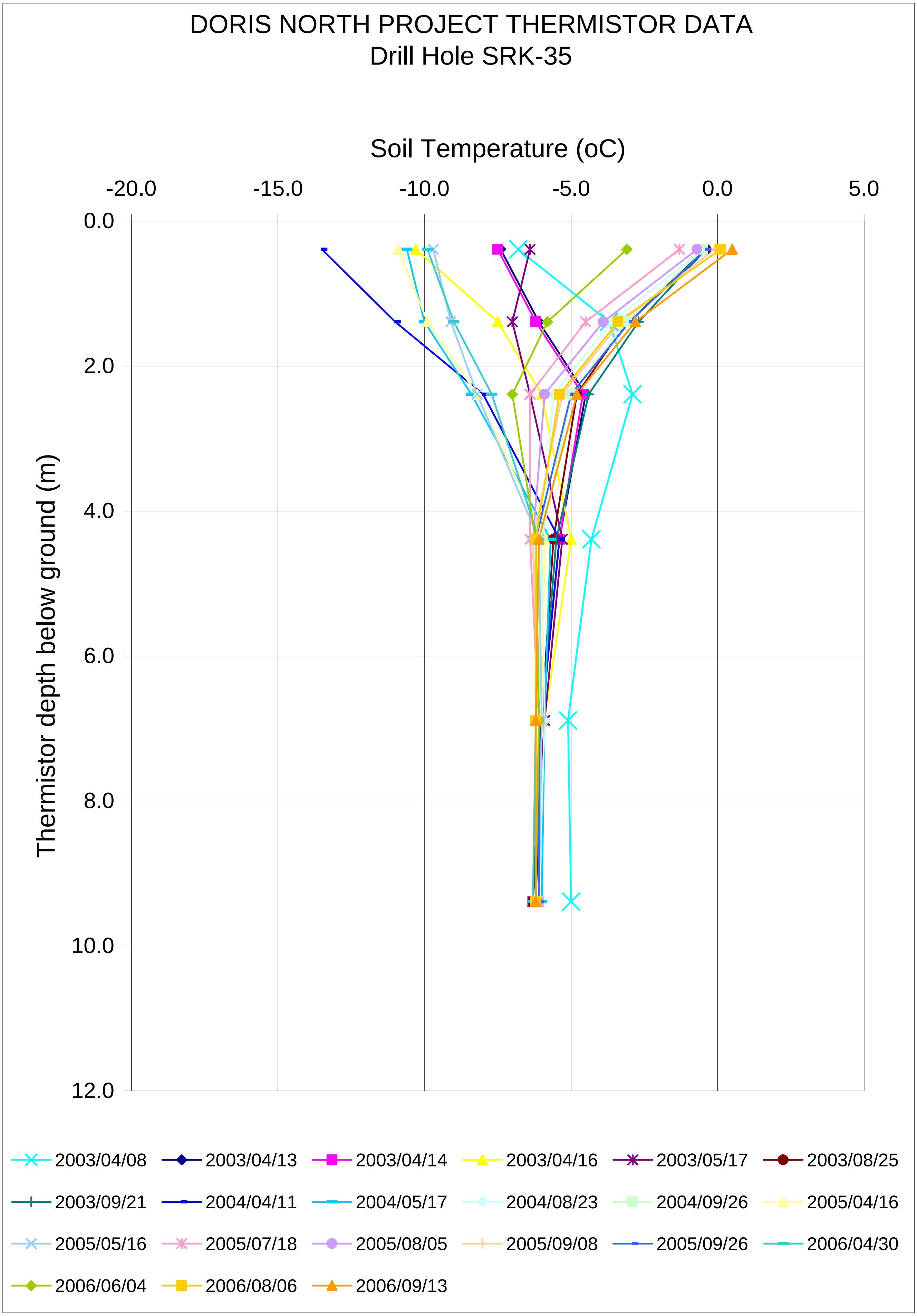

DORIS NORTH PROJECT THERMISTOR DATA
Drill Hole SRK-32
Soil Temperature (oC)
-20.0
-15.0
-10.0
-5.0
0.0
5.0
Thermistor depth below ground (m)
0.0
2.0
4.0
6.0
8.0
10.0
12.0
2003/04/06
2003/04/09
2003/04/13
2003/04/15
2003/04/16
2003/04/20
2003/05/16
2003/08/25
2003/09/21
2004/04/11
2004/05/17
2004/08/27
2004/09/28
2005/04/16
2005/05/16
2005/07/18
2005/08/05
2005/09/08
2005/09/26
2006/04/30
2006/06/04
2006/08/06
2006/09/13

DORIS NORTH PROJECT THERMISTOR DATA
Drill Hole SRK-33

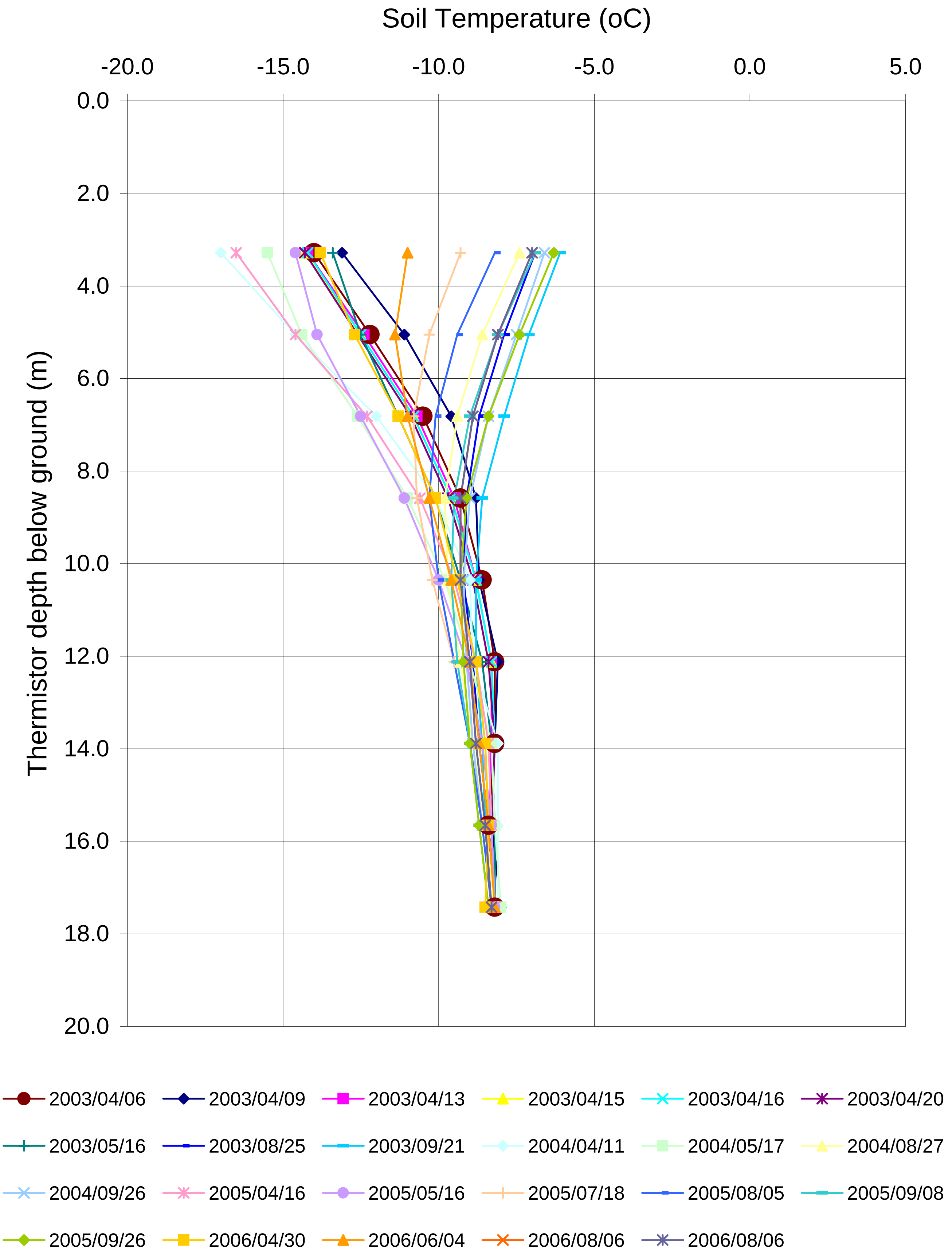


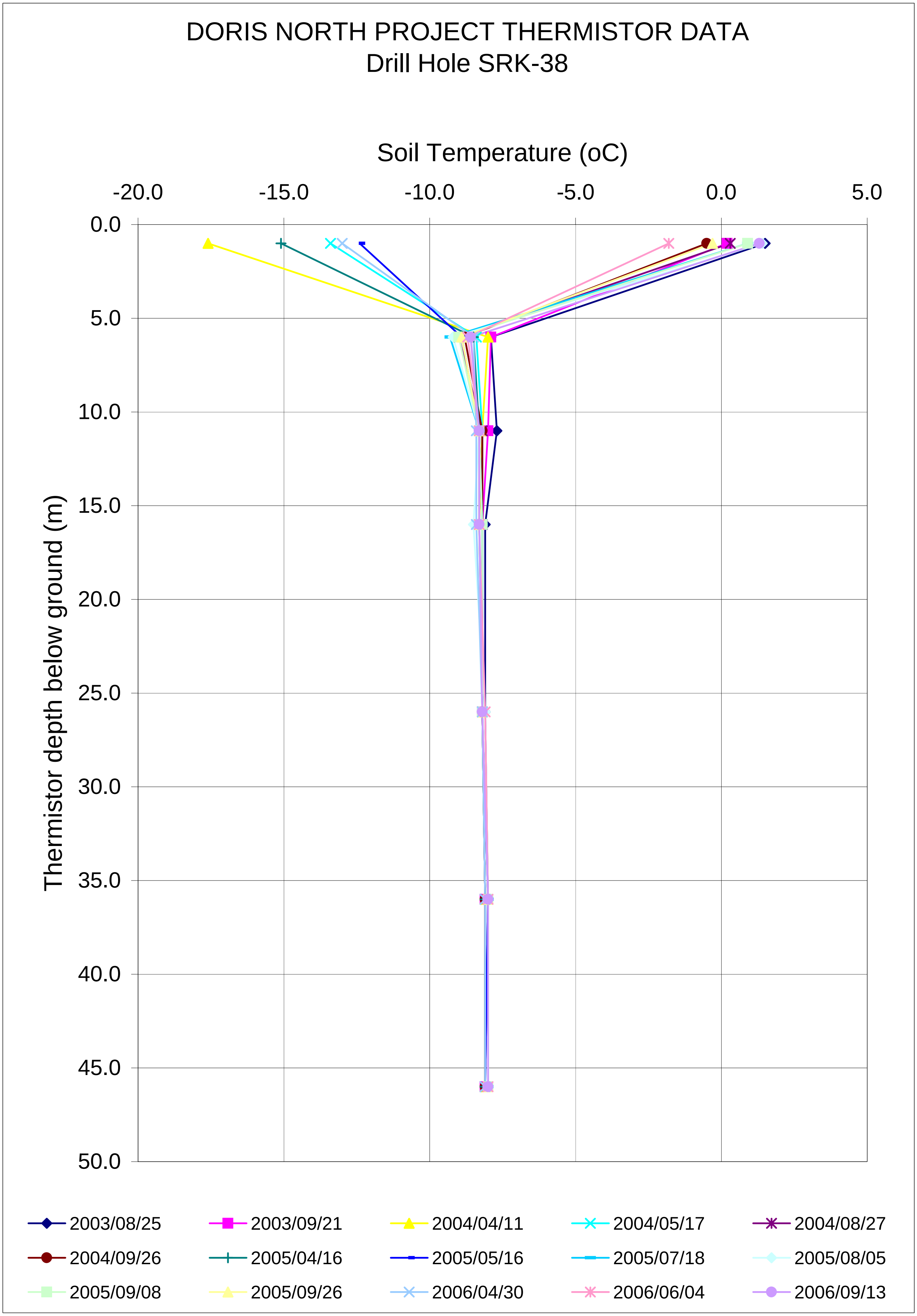


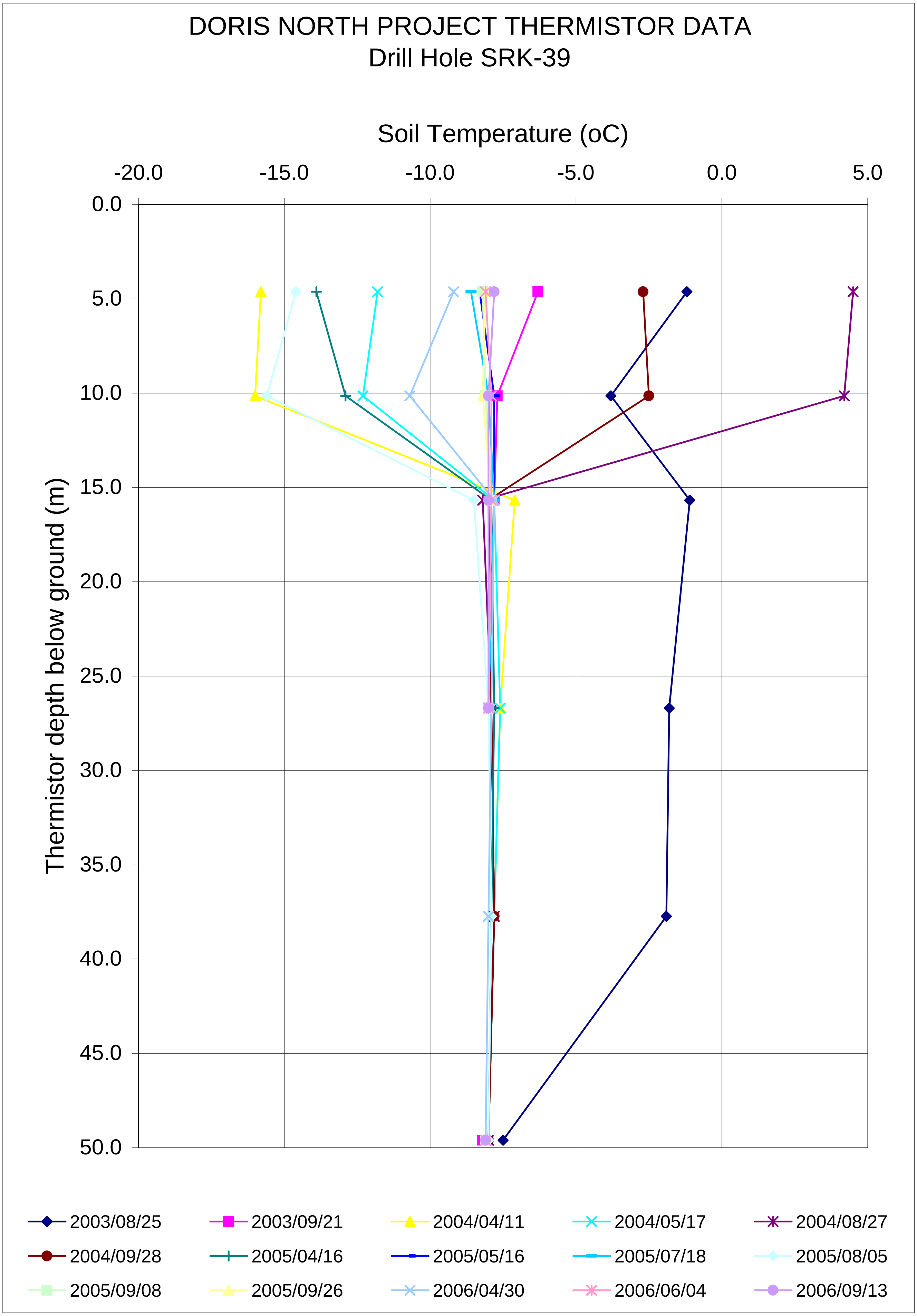
DORIS NORTH PROJECT THERMISTOR DATA
Drill Hole SRK-34A

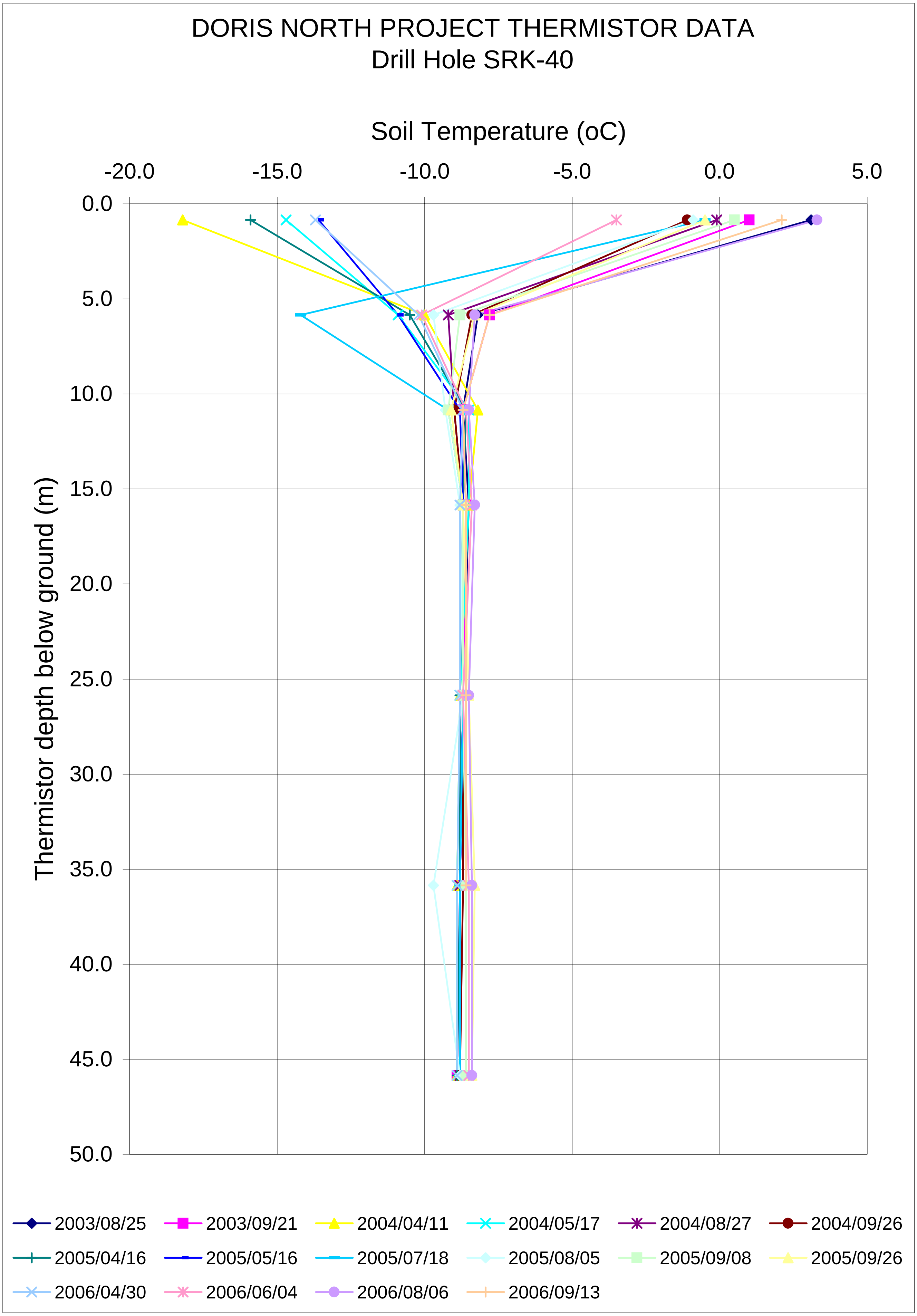

DORIS NORTH PROJECT THERMISTOR DATA
Drill Hole SRK-35
Soil Temperature (oC)
-20.0
-15.0
-10.0
-5.0
0.0
5.0
Thermistor depth below ground (m)
0.0
2.0
4.0
6.0
8.0
10.0
12.0
2003/04/08
2003/04/13
2003/04/14
2003/04/16
2003/05/17
2003/08/25
2003/09/21
2004/04/11
2004/05/17
2004/08/23
2004/09/26
2005/04/16
2005/05/16
2005/07/18
2005/08/05
2005/09/08
2005/09/26
2006/04/30
2006/06/04
2006/08/06
2006/09/13

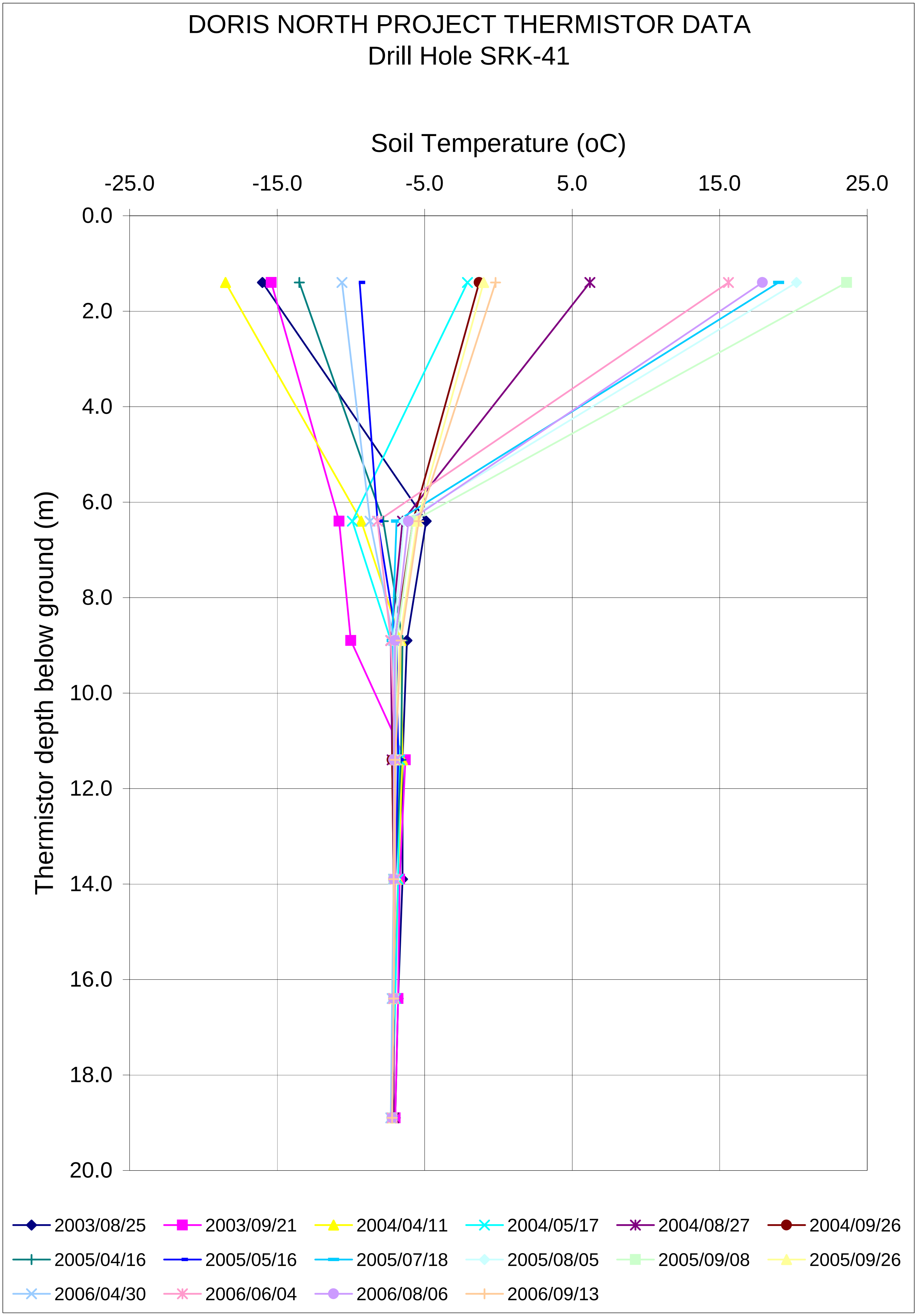
DORIS NORTH PROJECT THERMISTOR DATA
Drill Hole SRK-37



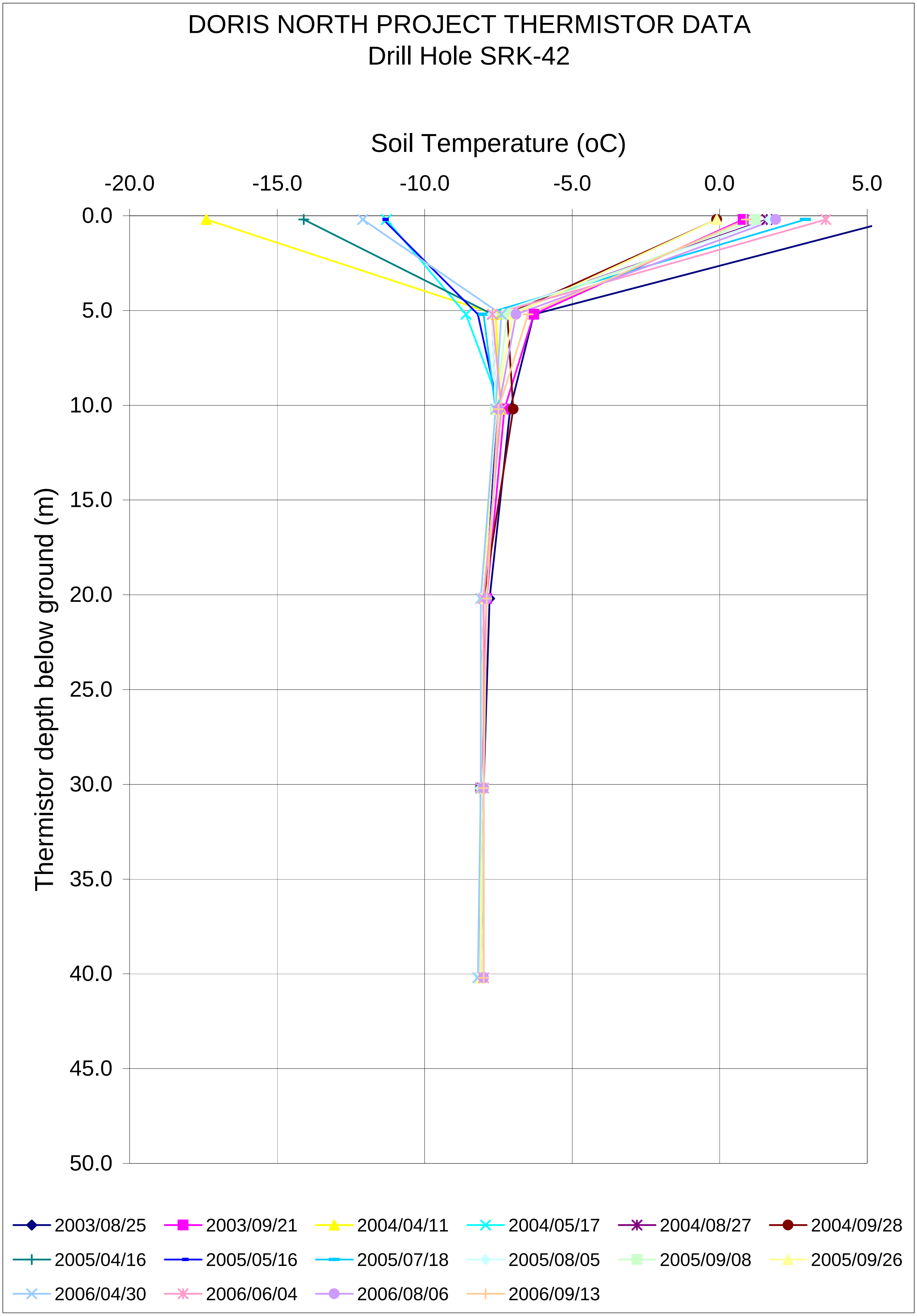

DORIS NORTH PROJECT THERMISTOR DATA
Drill Hole SRK-38
Soil Temperature (oC)
-20.0
-15.0
-10.0
-5.0
0.0
5.0
Thermistor depth below ground (m)
0.0
5.0
10.0
15.0
20.0
25.0
30.0
35.0
40.0
45.0
50.0
2003/08/25
2003/09/21
2004/04/11
2004/05/17
2004/08/27
2004/09/26
2005/04/16
2005/05/16
2005/07/18
2005/08/05
2005/09/08
2005/09/26
2006/04/30
2006/06/04
2006/09/13


DORIS NORTH PROJECT THERMISTOR DATA
Drill Hole SRK-39
Soil Temperature (oC)
-20.0
-15.0
-10.0
-5.0
0.0
5.0
0.0
5.0
10.0
15.0
20.0
25.0
30.0
35.0
40.0
45.0
50.0
Thermistor depth below ground (m)
2003/08/25
2003/09/21
2004/04/11
2004/05/17
2004/08/27
2004/09/28
2005/04/16
2005/05/16
2005/07/18
2005/08/05
2005/09/08
2005/09/26
2006/04/30
2006/06/04
2006/09/13

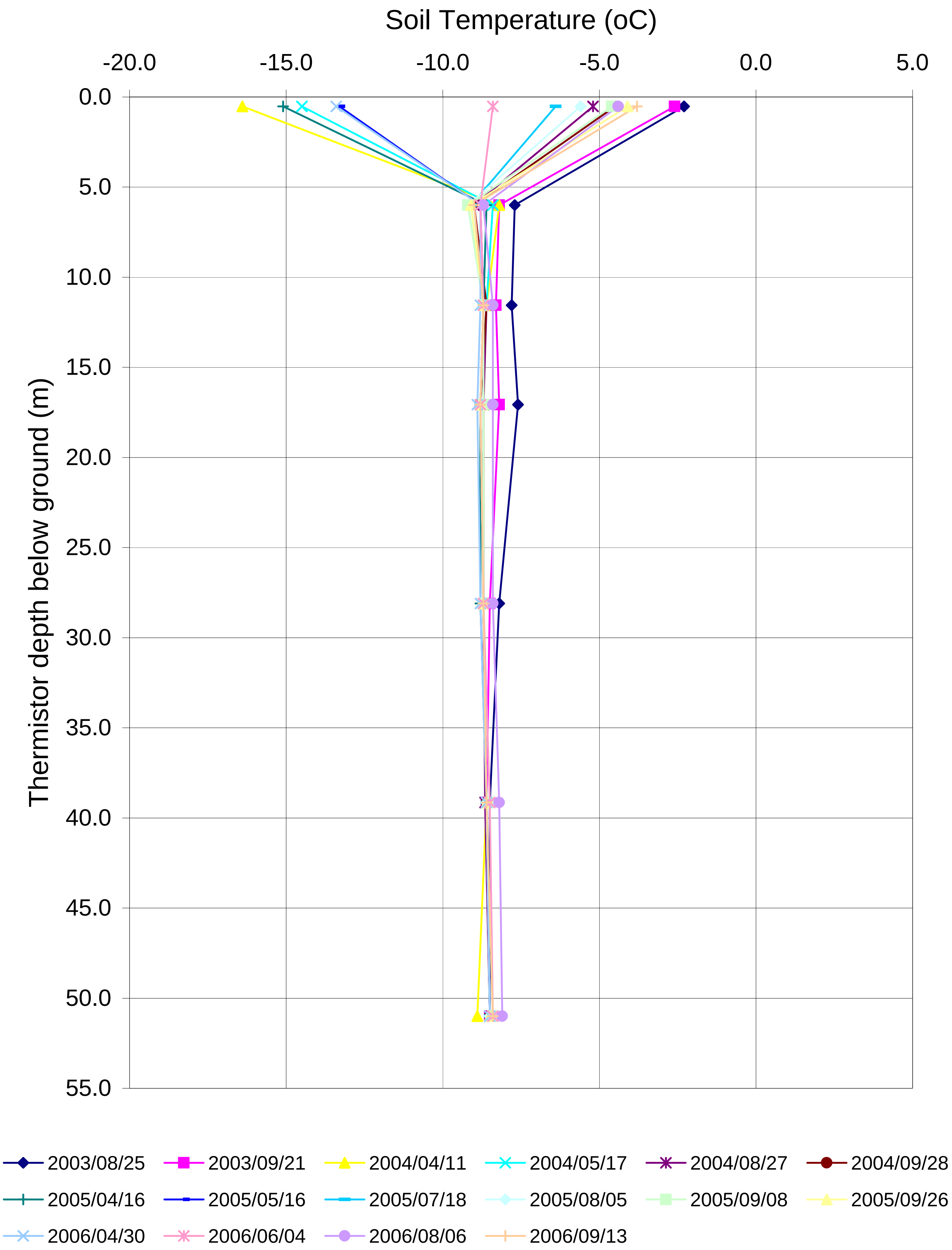

DORIS NORTH PROJECT THERMISTOR DATA
Drill Hole SRK-40
Soil Temperature (oC)
-20.0
-15.0
-10.0
-5.0
0.0
5.0
Thermistor depth below ground (m)
0.0
5.0
10.0
15.0
20.0
25.0
30.0
35.0
40.0
45.0
50.0
2003/08/25
2003/09/21
2004/04/11
2004/05/17
2004/08/27
2004/09/26
2005/04/16
2005/05/16
2005/07/18
2005/08/05
2005/09/08
2005/09/26
2006/04/30
2006/06/04
2006/08/06
2006/09/13

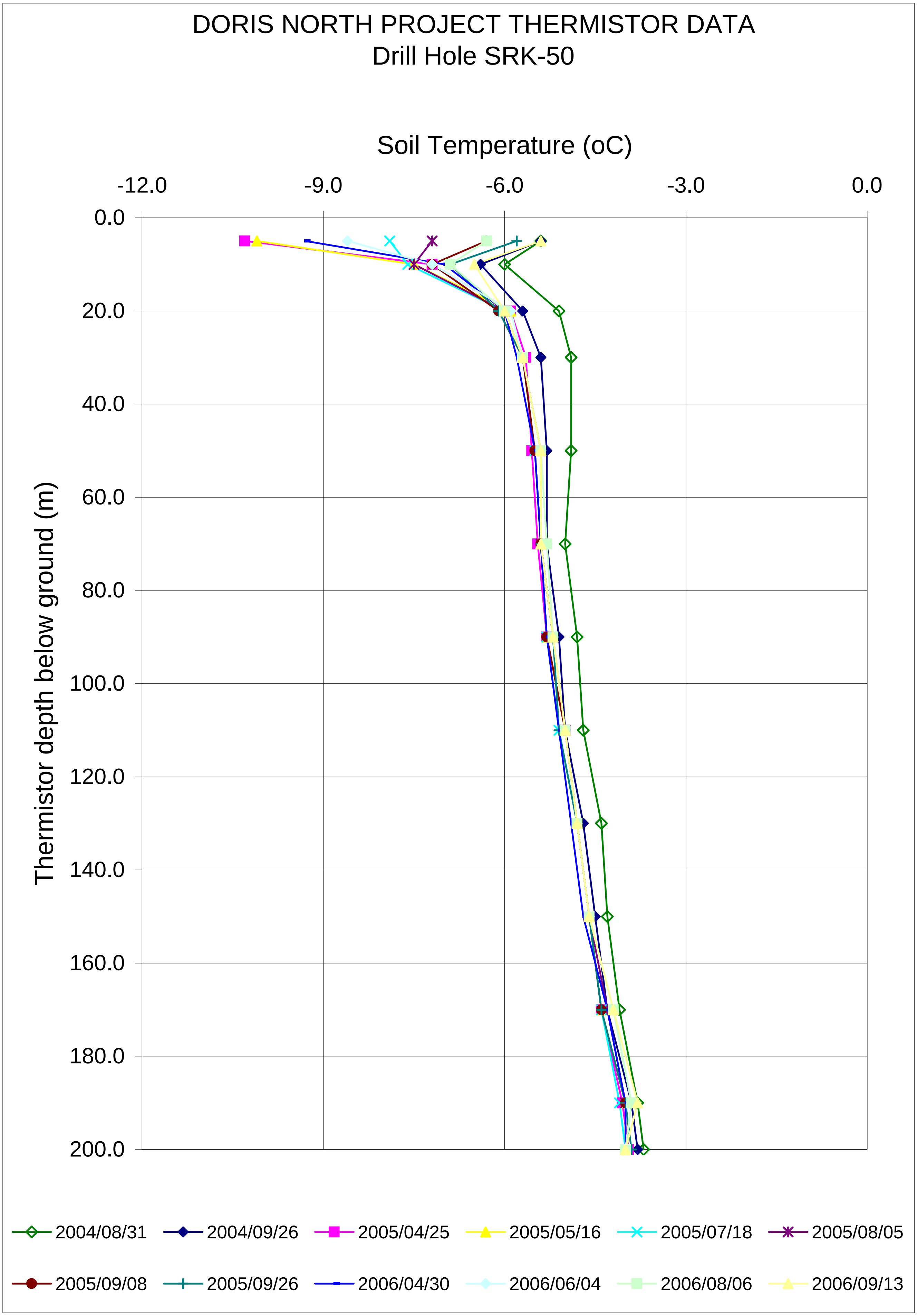


DORIS NORTH PROJECT THERMISTOR DATA
Drill Hole SRK-41

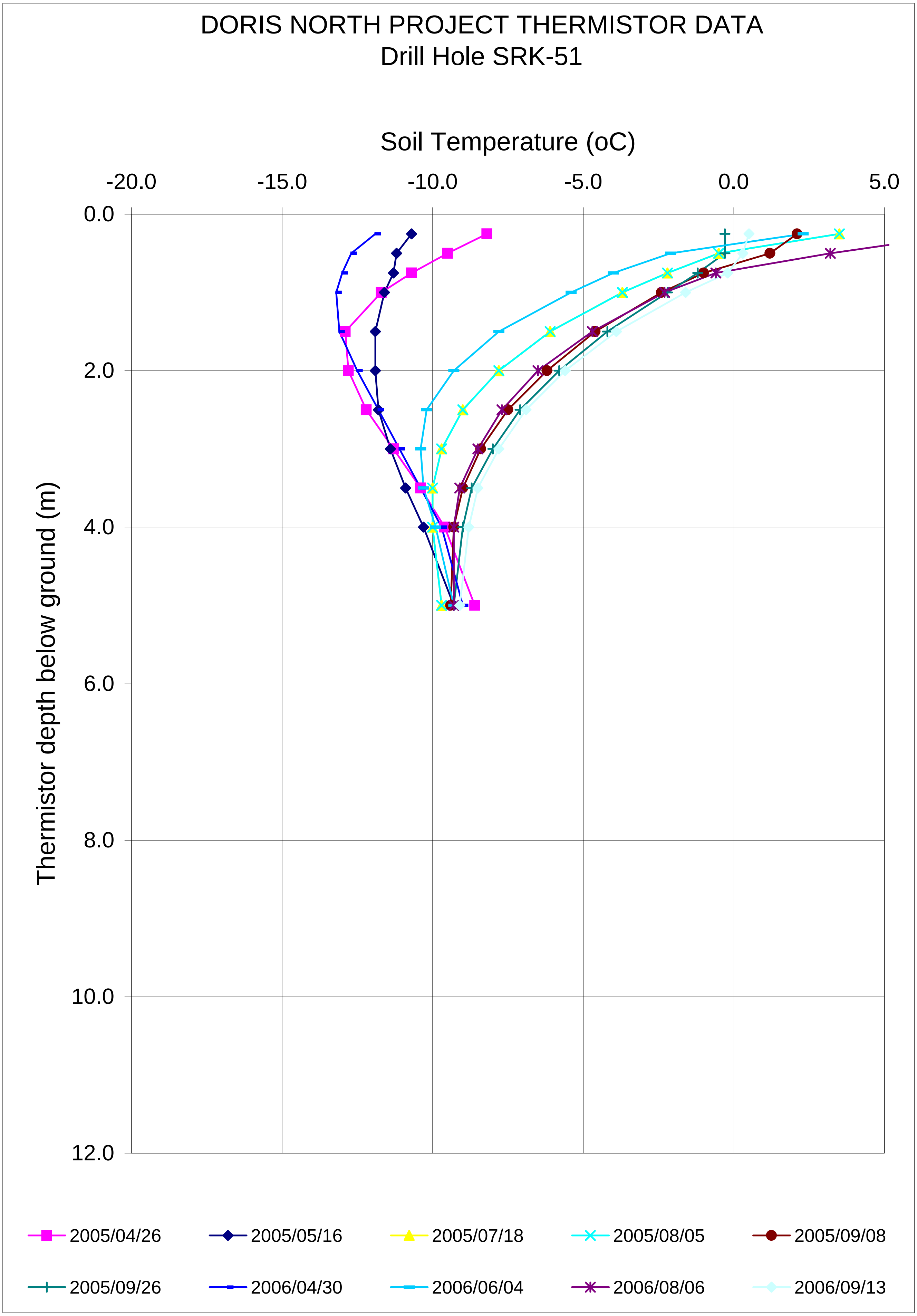

DORIS NORTH PROJECT THERMISTOR DATA
Drill Hole SRK-42
Soil Temperature (oC)
-20.0
-15.0
-10.0
-5.0
0.0
5.0
Thermistor depth below ground (m)
0.0
5.0
10.0
15.0
20.0
25.0
30.0
35.0
40.0
45.0
50.0
2003/08/25
2003/09/21
2004/04/11
2004/05/17
2004/08/27
2004/09/28
2005/04/16
2005/05/16
2005/07/18
2005/08/05
2005/09/08
2005/09/26
2006/04/30
2006/06/04
2006/08/06
2006/09/13

DORIS NORTH PROJECT THERMISTOR DATA
Drill Hole SRK-43

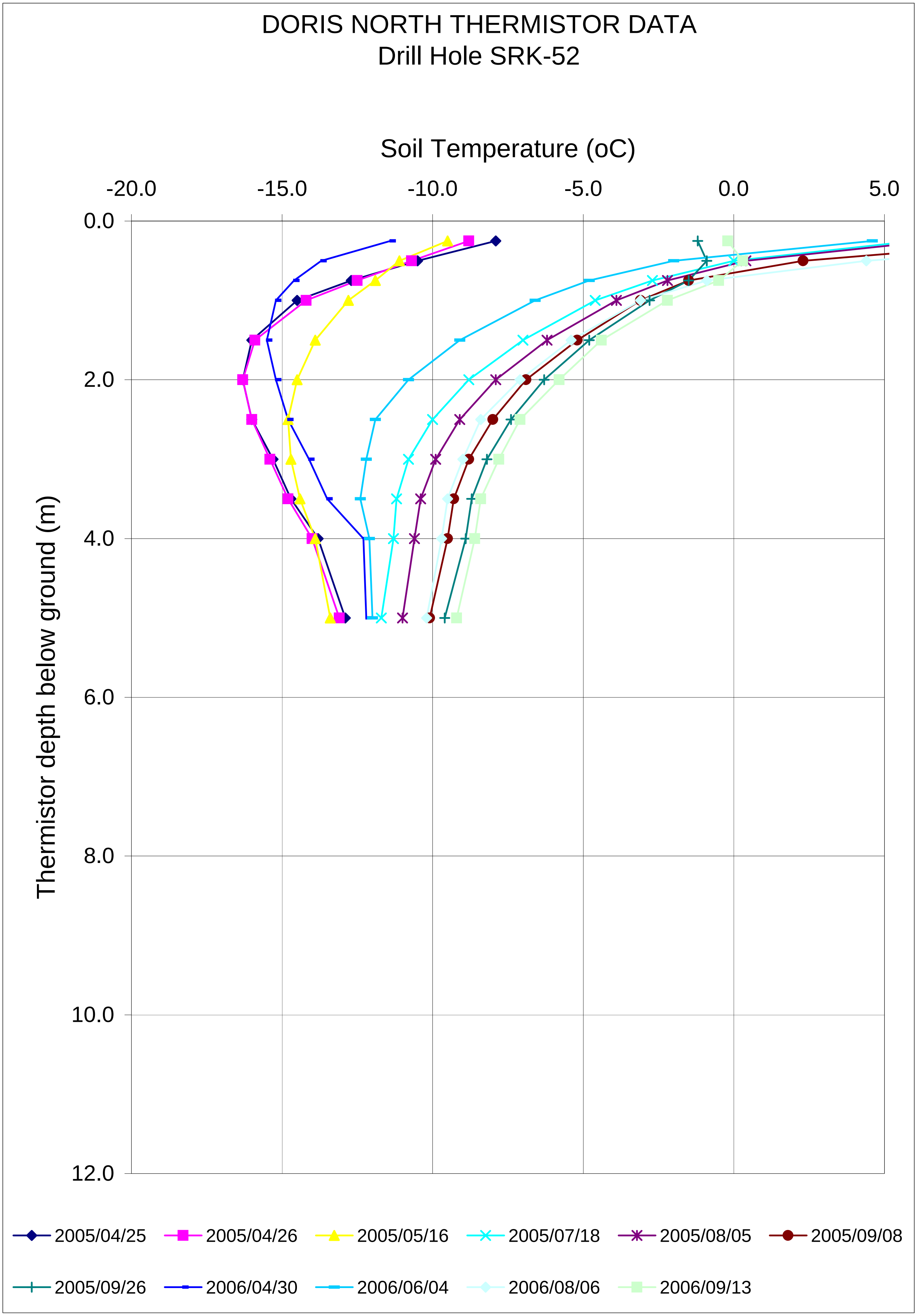


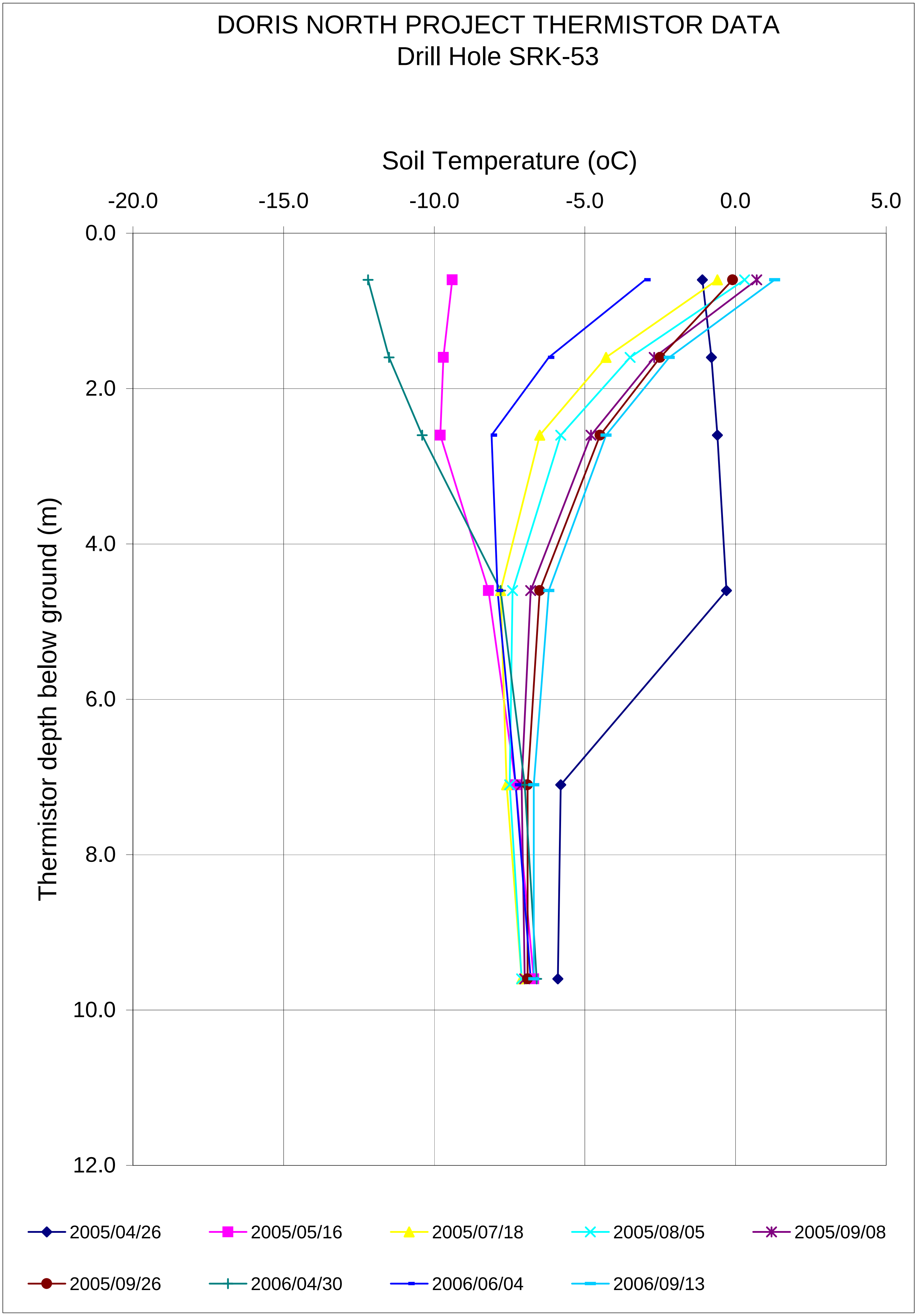


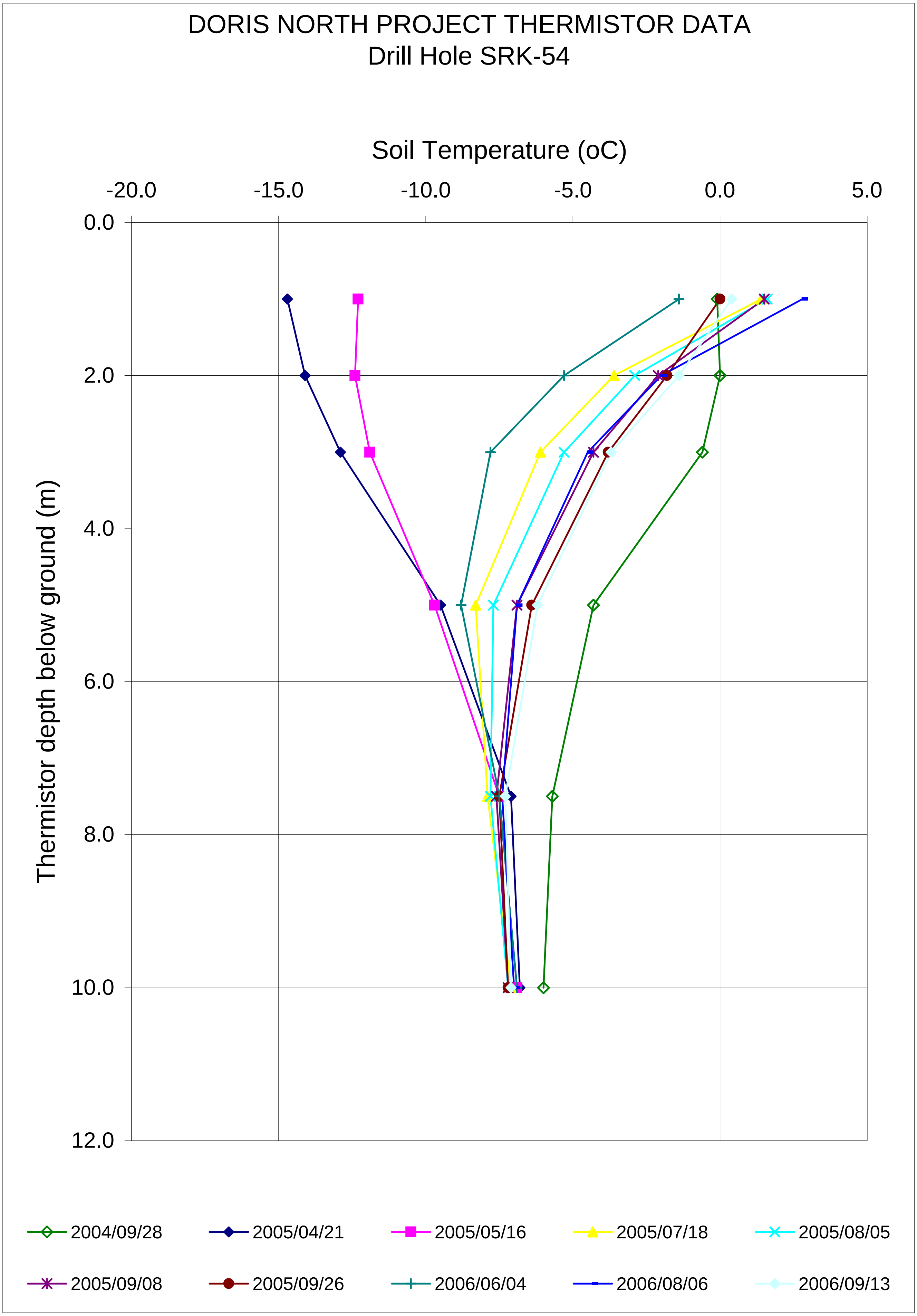
DORIS NORTH PROJECT THERMISTOR DATA
Drill Hole SRK-50

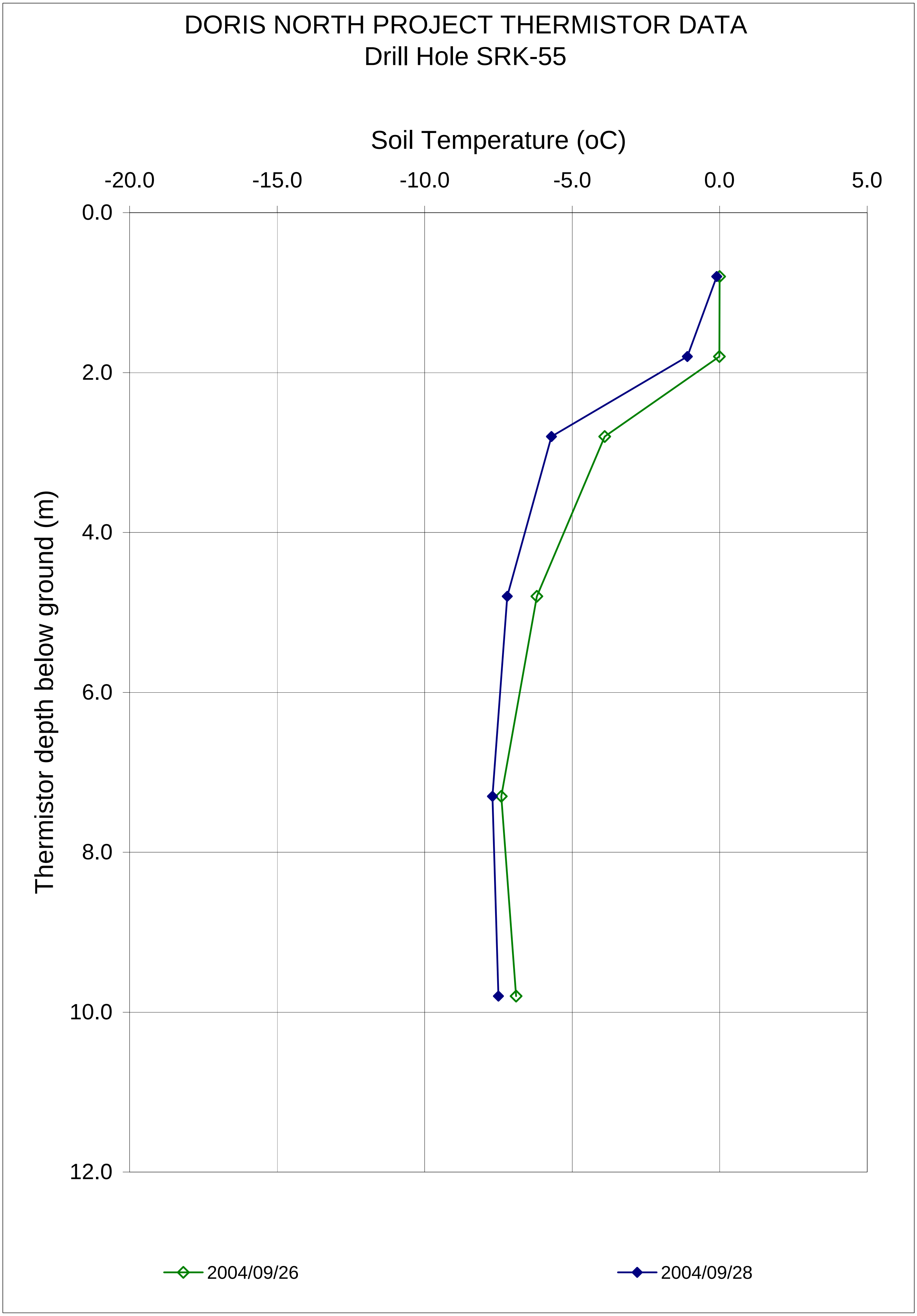


DORIS NORTH PROJECT THERMISTOR DATA
Drill Hole SRK-51

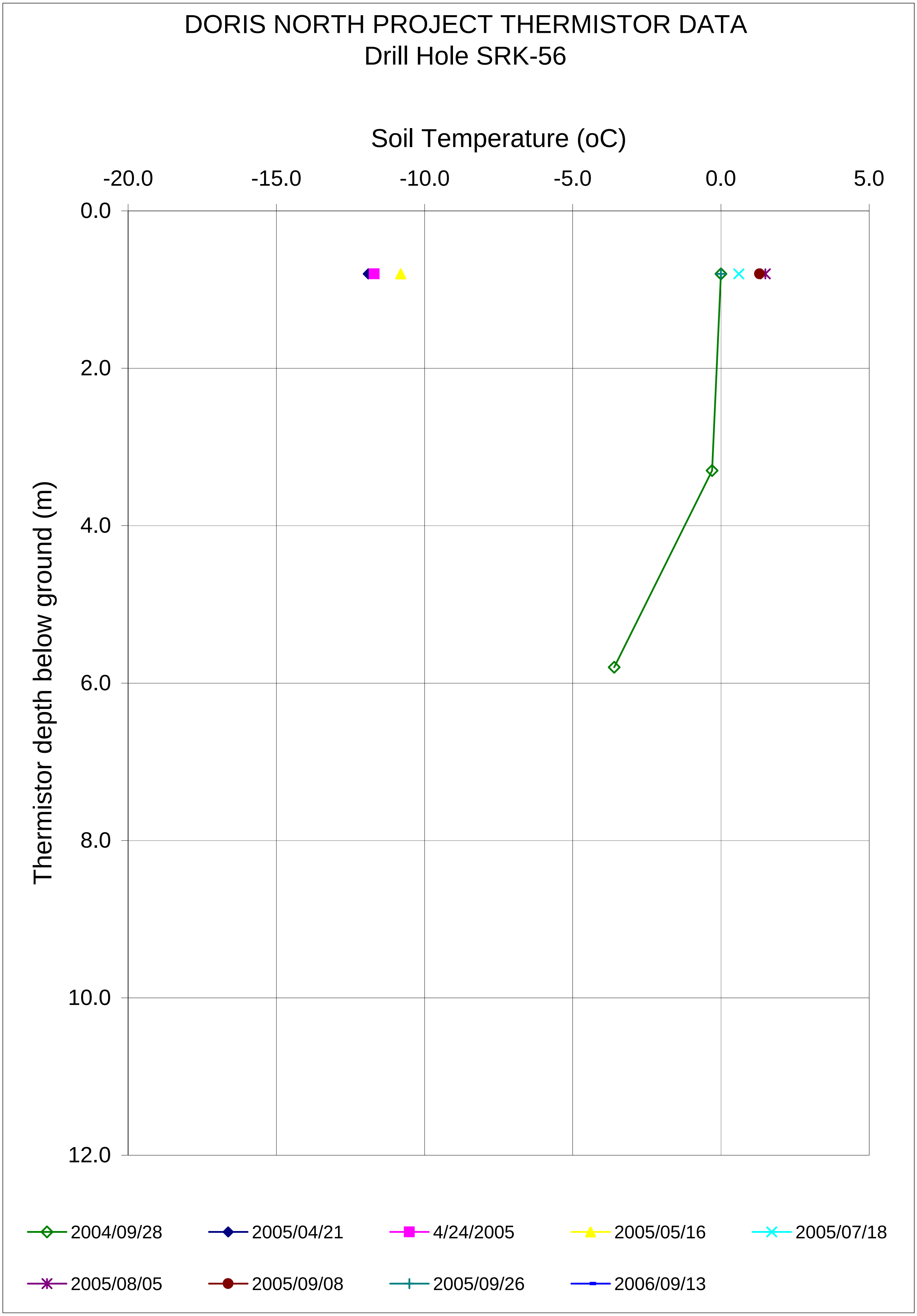

DORIS NORTH THERMISTOR DATA
Drill Hole SRK-52
Soil Temperature (oC)
-20.0
-15.0
-10.0
-5.0
0.0
5.0
Thermistor depth below ground (m)
0.0
2.0
4.0
6.0
8.0
10.0
12.0
2005/04/25
2005/04/26
2005/05/16
2005/07/18
2005/08/05
2005/09/08
2005/09/26
2006/04/30
2006/06/04
2006/08/06
2006/09/13

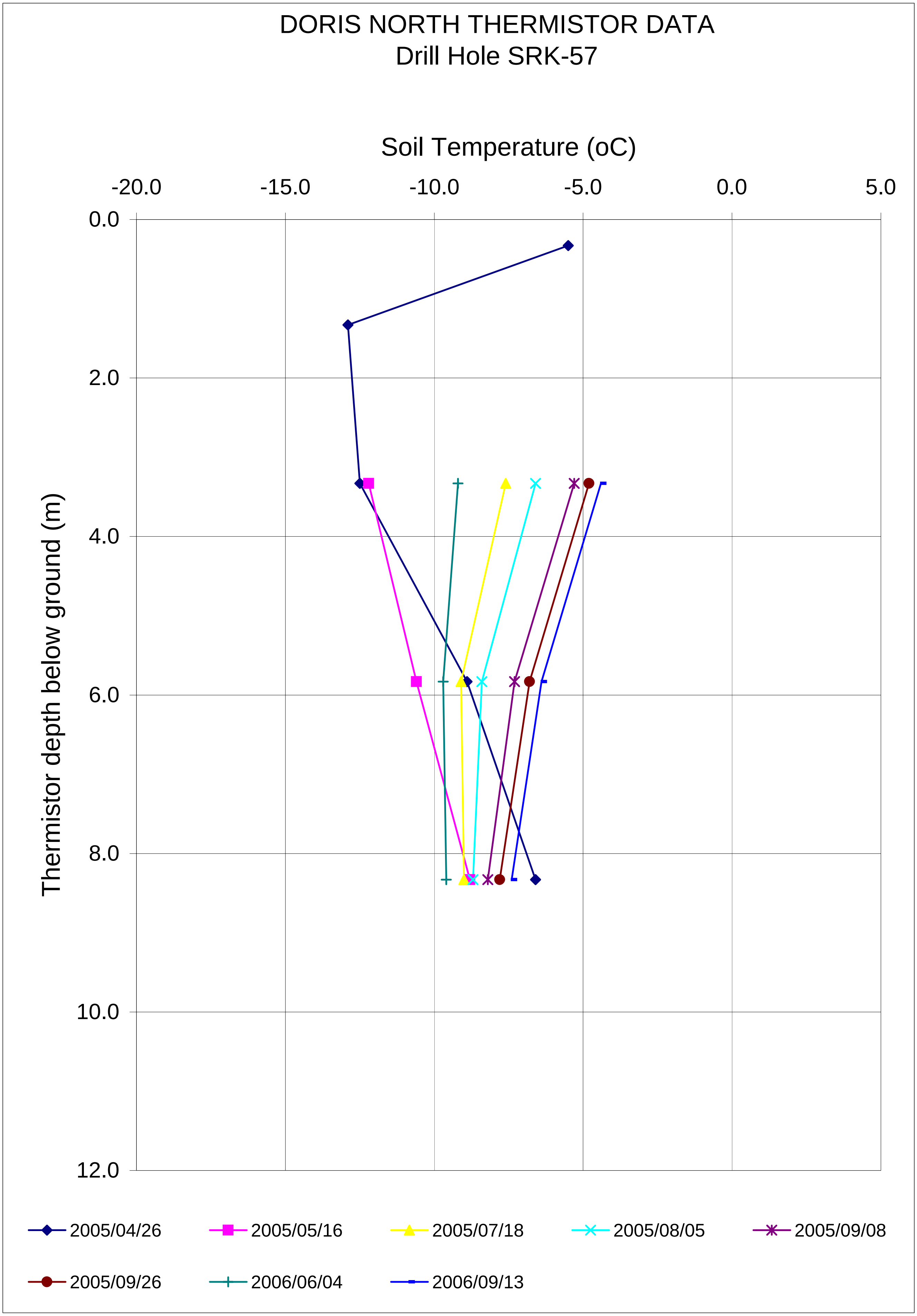

DORIS NORTH PROJECT THERMISTOR DATA
Drill Hole SRK-53
Soil Temperature (oC)
-20.0
-15.0
-10.0
-5.0
0.0
5.0
Thermistor depth below ground (m)
0.0
2.0
4.0
6.0
8.0
10.0
12.0
2005/04/26
2005/05/16
2005/07/18
2005/08/05
2005/09/08
2005/09/26
2006/04/30
2006/06/04
2006/09/13


DORIS NORTH PROJECT THERMISTOR DATA
Drill Hole SRK-54
Soil Temperature (oC)
-20.0
-15.0
-10.0
-5.0
0.0
5.0
Thermistor depth below ground (m)
0.0
2.0
4.0
6.0
8.0
10.0
12.0
2004/09/28
2005/04/21
2005/05/16
2005/07/18
2005/08/05
2005/09/08
2005/09/26
2006/06/04
2006/08/06
2006/09/13

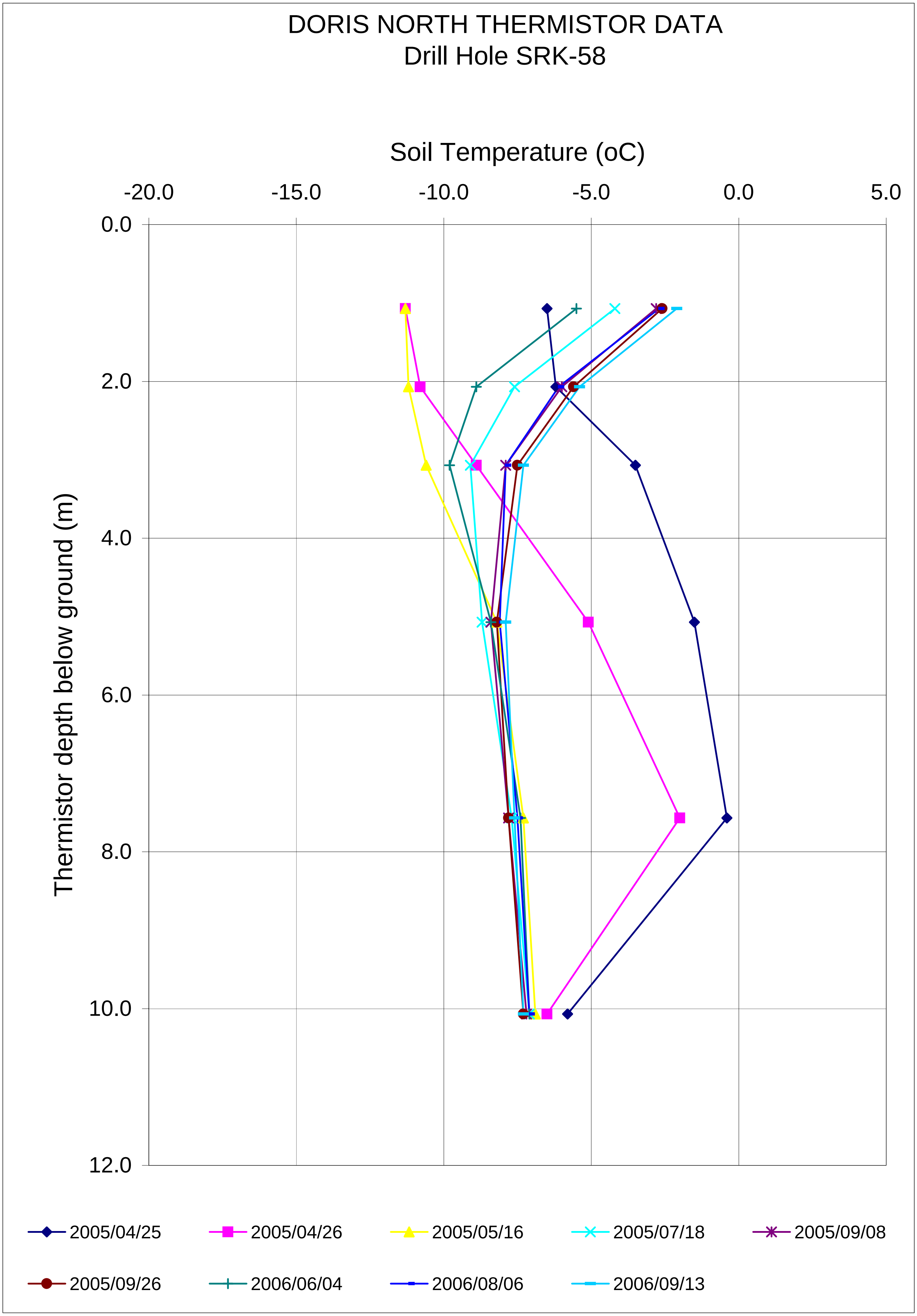


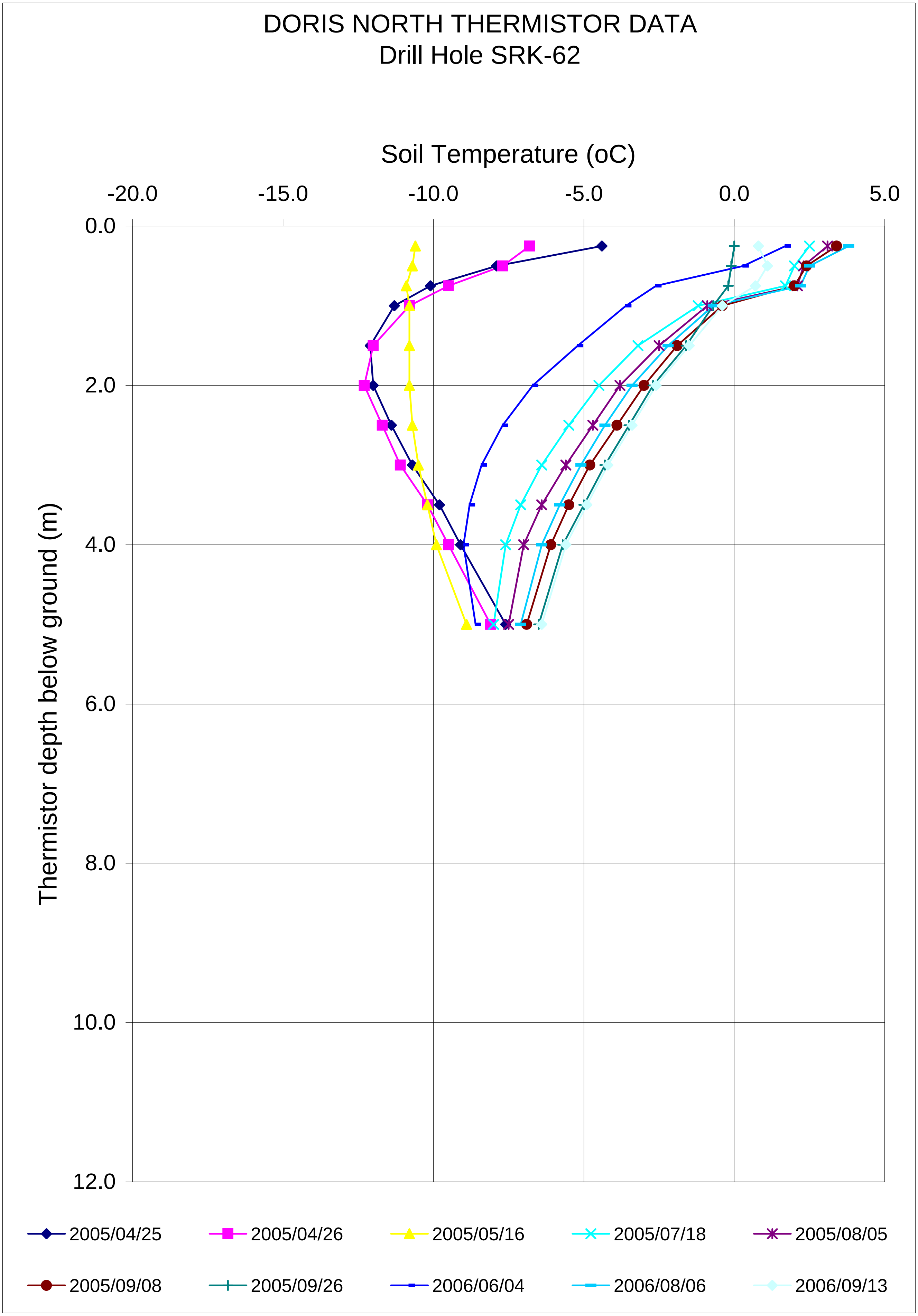
DORIS NORTH PROJECT THERMISTOR DATA
Drill Hole SRK-55


DORIS NORTH PROJECT THERMISTOR DATA
Drill Hole SRK-56
Soil Temperature (oC)
-20.0
-15.0
-10.0
-5.0
0.0
5.0
Thermistor depth below ground (m)
0.0
2.0
4.0
6.0
8.0
10.0
12.0
2004/09/28
2005/04/21
4/24/2005
2005/05/16
2005/07/18
2005/08/05
2005/09/08
2005/09/26
2006/09/13



DORIS NORTH THERMISTOR DATA
Drill Hole SRK-57


DORIS NORTH THERMISTOR DATA
Drill Hole SRK-58
Soil Temperature (oC)
-20.0
-15.0
-10.0
-5.0
0.0
5.0
Thermistor depth below ground (m)
0.0
2.0
4.0
6.0
8.0
10.0
12.0
2005/04/25
2005/04/26
2005/05/16
2005/07/18
2005/09/08
2005/09/26
2006/06/04
2006/08/06
2006/09/13


DORIS NORTH THERMISTOR DATA
Drill Hole SRK-62
Soil Temperature (oC)
-20.0
-15.0
-10.0
-5.0
0.0
5.0
Thermistor depth below ground (m)
0.0
2.0
4.0
6.0
8.0
10.0
12.0
2005/04/25
2005/04/26
2005/05/16
2005/07/18
2005/08/05
2005/09/08
2005/09/26
2006/06/04
2006/08/06
2006/09/13

Appendix C-2

Thermistor String Data - Tables

DORIS NORTH PROJECT THERMISTOR DATA

Drill Hole SRK-11		Bead No.	Bead 1	Bead 2	Bead 3	Bead 4	Bead 5
Read By	Date	Bead Location from Top (m)	5.0	6.0	7.0	8.5	10.0
		Bead Depth (m)	3.8	4.8	5.8	7.3	8.8
Andrew Doe (SRK)	2002/09/14	Temperature (Celsius)	-5.7	-6.3	-6.3	-6.2	-6.1
Andrew Doe (SRK)	2002/09/14		-6.5	-7.3	-7.4	-7.3	-7.0
Andrew Doe (SRK)	2002/09/15		-6.8	-7.4	-7.5	-7.4	-7.3
Dwayne Winsor (Miramar)	2002/09/19		-7.5	-7.8	-7.9	-7.9	-7.8
Dylan McGreggor (SRK)	2003/03/24					-7.9	-7.4
Dylan MacGregor (SRK)	2003/03/29		-8.1	-7.7	-7.7	-7.4	-7.6
Sebastian Fortin (SRK)	2003/04/06		-8.4	-7.9	-7.6	-7.4	-7.5
Dan Mackie (SRK)	2003/04/13		-8.7	-8.1	-7.7	-7.5	-7.5
Dan Mackie (SRK)	2003/04/15		-8.8	-8.5	-7.8	-7.6	-7.6
Dan Mackie (SRK)	2003/04/16		-8.8	-8.2	-7.8	-7.5	-7.5
Dan Mackie (SRK)	2003/04/20		-8.9	-8.2	-7.8	-7.5	-7.5
Jay Hallman (Miramar)	2003/05/16		-9.5	-8.9	-8.4	-7.9	-7.7
Dylan MacGregor (SRK)	2003/08/25		-8.3	-8.4	-8.4	-8.1	-8.1
Mike Cripps (Miramar)	2003/09/21		0.8	-8.0	-8.1	-8.0	-8.1
Dylan MacGregor (SRK)	2004/04/11		-8.7	-8.1	-7.8	-7.4	-7.5
Thorpe/Lindsay (Miramar)	2004/05/17		-9.9	-9.2	-8.6	-7.9	-7.8
Dylan MacGregor (SRK)	2004/08/27		-8.7	-8.8	-8.8	-8.5	-8.4
Quinn Jordan-Knox (SRK)	2004/09/26		-8.1	-8.3	-8.3	-8.3	-8.3
Dylan MacGregor (SRK)	2005/04/16		-9.9	-9.1	-8.6	-7.9	-7.9
D Kary (Miramar)	2005/05/16		-10.5	-9.8	-9.1	-8.4	-8.1
Jay Hallman (Miramar)	2005/07/18		-9.8	-9.6	-9.3	-8.8	-8.5
JRH (Miramar)	2005/08/05		-9.3	-9.2	-9.1	-8.7	-8.6
D. Kary (Miramar)	2005/09/08		-8.5	-8.7	-8.8	-8.5	-8.5
E Ballent (Miramar)	2005/09/26		-0.4	-2.1	-4.4	-8.4	-8.4
L.Wade / A. Tong (SRK)	2006/04/30		-9.1	-8.7	-8.4	-7.9	-7.9
Jay Hallman (Miramar)	2006/06/04		-9.4	-9.0	-8.7	-8.1	-8.1
S Gary/J Hugh (Miramar)	2006/08/29		-8.4	-8.5	-8.5	-8.2	-8.2
H.Johnson (Miramar)	2006/09/13		-7.7	-7.9	-8.1	-8	-8.1

DORIS NORTH PROJECT THERMISTOR DATA

Drill Hole SRK-13		Bead No.	Bead 1	Bead 2	Bead 3	Bead 4	Bead 5
Read By	**Date**	**Bead Location from Top (m)**	**5.0**	**6.0**	**7.0**	**8.5**	**10.0**
		Bead Depth (m)	**1.1**	**2.1**	**3.1**	**4.6**	**6.1**
Andrew Doe (SRK)	2002/09/14	**Temperature (Celsius)**	0.0	-1.5	-2.8	-4.3	-7.6
Andrew Doe (SRK)	2002/09/14		-1.1	-3.7	-5.6	-7.5	-8.0
Andrew Doe (SRK)	2002/09/15		-1.2	-3.9	-5.8	-7.5	-8.1
Dwayne Winsor (Miramar)	2002/09/19		-1.2	-4.2	-6.3	-7.6	-8.1
Dwayne Winsor (Miramar)	2003/02/16		-26.5	-16.9	-11.9	-7.4	-7.1
Maritz Rykaart (SRK)	2003/03/17		-21.5	-19.6	-14.8	-9.4	-8.1
Dylan MacGregor (SRK)	2003/03/24		-23.5	-18.1	-14.9	-9.9	-8.4
Sebastian Fortin (SRK)	2003/04/06		-17.4	-17.6	-14.8	-10.5	-9.0
Dan Mackie (SRK)	2003/04/13		-18.8	-16.8	-14.7	-10.8	-9.3
Dan Mackie (SRK)	2003/04/15		-21.7	-16.7	-14.7	-10.9	-9.4
Dan Mackie (SRK)	2003/04/16		-13.1	-17.0	-14.9	-11.2	-9.5
Dan Mackie (SRK)	2003/04/20		-13.3	-16.4	-14.8	-11.3	-9.6
Jay Hallman (Miramar)	2003/05/16		-3.3	-12.1	-12.6	-11.4	-10.1
Dylan MacGregor (SRK)	2003/08/25		16.4	1.0	-3.4	-7.4	-8.3
Thermistor Permanently Damaged - Not Repairable							

DORIS NORTH PROJECT THERMISTOR DATA

Drill Hole SRK-14		**Bead No.**	**Bead 1**	**Bead 2**	**Bead 3**	**Bead 4**	**Bead 5**	**Bead 6**
Read By	**Date**	**Bead Location from Top (m)**	**2.0**	**3.0**	**4.0**	**6.0**	**8.5**	**11.0**
		Bead Depth (m)	**1.2**	**2.2**	**3.2**	**5.2**	**7.7**	**10.2**
Sebastian Fortin (SRK)	2003/04/06	**Temperature (Celsius)**	-11.8	-11.1	-9.7	-7.6	-7.8	-8.3
Dan Mackie (SRK)	2003/04/13		-13.6	-12.6	-11.3	-9.3	-8.5	-8.6
Dan Mackie (SRK)	2003/04/15		-13.8	-12.8	-11.6	-9.6	-8.6	-8.6
Dan Mackie (SRK)	2003/04/16		-14.6	-13.7	-12.7	-10.5	-8.7	-8.6
Dan Mackie (SRK)	2003/04/20		-14.7	-13.9	-12.8	-10.8	-9.0	-8.7
Jay Hallman (Miramar)	2003/05/16		-10.8	-12.4	-12.5	-11.5	-9.8	-9.1
Dylan MacGregor (SRK)	2003/08/25		-0.1	-2.6	-4.9	-7.6	-9.2	-9.4
Mike Cripps (Miramar)	2003/09/21		-0.1	-2.1	-4.1	-6.8	-8.6	-9.2
Dylan MacGregor (SRK)	2004/04/11		-17.8	-16.3	-12.8	-9.8	-8.9	
Thorpe/Lindsay (Miramar)	2004/05/17		-13.8	-14.7	-14.9	-13.2	-10.8	-9.6
Dylan MacGregor (SRK)	2004/08/27		-0.5	-2.8	-5.5	-8.5	-9.9	-9.9
Quinn Jordan-Knox (SRK)	2004/09/26		-0.6	-2.3	-4.5	-7.3	-9.2	-9.6
Dylan MacGregor (SRK)	2005/04/16		-16.9	-17	-16.4	-13.8	-16.9	-9.6
D Kary (Miramar)	2005/05/16		-13.1	-13.9	-14.3	-13.5	-11.5	-10.1
Jay Hallman (Miramar)	2005/07/18		-1.4	-4.7	-7.7	-10.6	-11.1	-10.5
JRH (Miramar)	2005/08/05		-0.9	-3.6	-6.4	-9.5	-10.6	-10.4
D Kary (Miramar)	2005/09/08		-0.3	-2.5	-5.0	-8.1	-9.8	-10.1
E Ballent (Miramar)	2005/09/26		-0.4	-2.1	-4.4	-7.4	-9.3	-9.9
L. Wade / A. Tong (SRK)	2006/04/30		-13.9	-13.8	-13.3	-11.8	-10.2	-9.5
Jay Hallman (Miramar)	2006/06/04		-3.8	-7.2	-9.7	-11.1	-10.4	-9.7
Gary S/Hugh J. (Miramar)	2006/08/06		0.0	-2.5	-5.0	-7.9	-9.4	-9.7
H.Johnson (Miramar)	2006/09/13		0.4	-1.6	-3.9	-6.7	-8.6	-9.3

DORIS NORTH PROJECT THERMISTOR DATA

Drill Hole SRK-15		Bead No.	Bead 1	Bead 2	Bead 3	Bead 4	Bead 5	Bead 6	Bead 7	Bead 8	Bead 9	Bead 10
Read By	Date	Bead Location from Top (m)	6.0	11.0	13.5	16.0	18.5	21.0	23.5	26.0	28.5	31.0
		Inclined Bead Depth (m)	2.6	7.6	10.1	12.6	15.1	17.6	20.1	22.6	25.1	27.6
		Vert. Bead Depth (m)	1.8	5.4	7.1	8.9	10.7	12.4	14.2	16.0	17.7	19.5
Sebastian Fortin (SRK)	2003/04/06	Temperature (Celsius)	-13.2	-8.3	-8.0	-8.1	-8.1	-7.8	-8.1	-8.2	-8.1	-8.1
Dan Mackie (SRK)	2003/04/13		-13.5	-8.4	-8.2	-8.3	-8.2	-7.9	-8.2	-8.2	-8.2	-8.2
Dan Mackie (SRK)	2003/04/15		-13.6	-8.4	-8.4	-8.3	-8.3	-8.0	-8.2	-8.2	-8.2	-8.2
Dan Mackie (SRK)	2003/04/16		-13.6	-8.4	-8.2	-8.3	-8.3	-8.0	-8.2	-8.2	-8.2	-8.1
Dan Mackie (SRK)	2003/04/20		-13.6	-8.4	-8.2	-8.3	-8.3	-8.0	-8.2	-8.2	-8.2	-8.2
Jay Hallman (Miramar)	2003/05/16		-11.4	-8.6	-8.4	-8.4	-8.4	-8.2	-8.2	-8.3	-8.2	-8.2
Dylan MacGregor (SRK)	2003/08/25		-4.6	-8.9	-8.6	-8.5	-8.4	-8.3	-8.3	-8.3	-8.2	-8.2
Mike Cripps (Miramar)	2003/09/21		-4.1	-8.8	-8.6	-8.5	-8.4	-8.3	-8.3	-8.2	-8.1	-8.2
Dylan MacGregor (SRK)	2004/04/11		-17.4	-8.4	-8.9	-8.5	-8.5	-8.3	-8.2	-8.3	-8.0	-8.1
Thorpe/Lindsay (Miramar)	2004/05/17		-13.6	-8.8	-8.5	-8.4	-8.5	-8.3	-8.2	-8.3	-8.0	-8.0
Dylan MacGregor (SRK)	2004/08/27		-4.9	-9.5	-8.9	-8.4	-8.4	-8.2	-8.1	-8.2	-8.0	-8.0
Quinn Jordan-Knox (SRK)	2004/09/28		-4.3	-9.4	-9.0	-8.5	-8.4	-8.2	-8.1	-8.2	-8.0	-8.0
Dylan MacGregor (SRK)	2005/04/16		-15.2	-9.2	-8.8	-8.6	-8.5	-8.3	-8.2	-8.2	-8.0	-8.0
D Kary (Miramar)	2005/05/16		-12.8	-9.7	-8.9	-8.6	-8.5	-8.3	-8.2	-8.2	-8.0	-8.0
Jay Hallman (Miramar)	2005/07/18		-6.1	-10.0	-9.2	-8.6	-8.5	-8.3	-8.1	-8.2	-8.0	-8.0
JRH (Miramar)	2005/08/05		-5.4	-9.9	-9.2	-8.7	-8.5	-8.3	-8.1	-8.2	-8.0	-8.0
D Kary (Miramar)	2005/09/08		-4.5	-9.7	-9.3	-8.7	-8.5	-8.3	-8.1	-8.2	-7.9	-8.0
E Ballent (Miramar)	2005/09/26		4.2	-9.6	-9.3	-8.8	-8.6	-8.3	-8.2	-8.2	-8.0	-8.0
L. Wade / A. Tong (SRK)	2006/04/30		-12.7	-9.2	-9.0	-8.8	-8.7	-8.5	-8.3	-8.0	-8.1	
Jay Hallman (Miramar)	2006/06/04		-7.6	-9.4	-9.0	-8.7	-8.6	-8.4	-8.2	-8.3	-8.0	-8.0
Gary S./ Hugh J. (Miramar)	2006/08/06		-4.6	-9.4	-9.1	-8.8	-8.6	-8.4	-8.2	-8.3	-8.0	-8.0
H.Johnson (Miramar)	2006/09/13		-1.2	-5.7	-9.0	-8.7	-8.4	-8.2	-8.1	-8.1	-7.9	-8.0

DORIS NORTH PROJECT THERMISTOR DATA

Drill Hole SRK-16		Bead No.	Bead 1	Bead 2	Bead 3	Bead 4	Bead 5
Read By	**Date**	**Bead Location from Top (m)**	**5.0**	**6.0**	**7.0**	**8.5**	**10.0**
		Bead Depth (m)	**3.3**	**4.3**	**5.3**	**6.8**	**8.3**
Andrew Doe (SRK)	2002/09/14	**Temperature (Celsius)**	-0.6	-1.4	-1.6	-2.0	-1.6
Andrew Doe (SRK)	2002/09/15		-2.1	-2.4	-2.5	-3.1	-2.8
Dwayne Winsor (Miramar)	2002/09/19		-5.0	-6.5	-7.4	-7.9	-6.9
Dylan MacGregor (SRK)	2003/03/18		-13.6	-11.8	-10.5	-9.3	-8.9
Dylan MacGregor (SRK)	2003/03/24		-13.9	-12.2	-10.8	-9.5	-9.0
Sebastian Fontin (SRK)	2003/04/06		-14.3	-12.7	-11.4	-9.9	-9.2
Dan Mackie (SRK)	2003/04/13		-14.5	-13.0	-11.6	-10.2	-9.4
Dan Mackie (SRK)	2003/04/15		-14.5	-13.0	-11.7	-10.2	-9.5
Dan Mackie (SRK)	2003/04/16		-14.5	-13.0	-11.7	-10.3	-9.5
Dan Mackie (SRK)	2003/04/20		-14.5	-13.1	-11.8	-10.4	-9.5
Jay Hallman (Miramar)	2003/05/16		-13.6	-13.0	-12.1	-10.9	-10.0
Dylan MacGregor (SRK)	2003/08/25		-7.4	-8.4	-9.0	-9.6	-9.8
Mike Cripps (Miramar)	2003/09/21		-6.5	-7.6	-8.3	-9.0	-9.4
Dylan MacGregor (SRK)	2004/04/11		-16.7	-13.7	-13.0	-11.0	-9.8
Thorpe/Lindsay (Miramar)	2004/05/17		-15.3	-14.4	-13.5	-11.9	-10.7
Dylan MacGregor (SRK)	2004/08/27		-7.9	-8.9	-9.7	-10.2	-10.4
Quinn Jordan-Knox (SRK)	2004/09/26		-6.8	-7.9	-8.3	-9.4	-9.8
Dylan MacGregor (SRK)	2005/04/16		-16.3	-14.8	-13.5	-11.7	-10.6
D Kary (Miramar)	2005/05/16		-14.5	-14.0	-13.4	-12.2	-11.1
Jay Hallman (Miramar)	2005/07/18		-9.7	-10.6	-11.2	-11.3	-11.1
JRH (Miramar)	2005/08/05		-8.6	-9.7	-10.4	-10.8	-10.8
D Kary (Miramar)	2005/09/08		-7.4	-8.5	-9.3	-10.0	-10.3
E Ballent (Miramar)	2005/09/26		-6.7	-7.8	-8.7	-9.5	-9.9
L. Wade / A. Tong (SRK)	2006/04/30		-14.0	-13.0	-12.3	-11.1	-10.3
Jay Hallman (Miramar)	2006/06/04		-11.4	-11.7	-11.7	-11.2	-10.6
Gary S./Hugh J. (Miramar)	2006/08/06		-7.5	-8.5	-9.3	-9.8	-10.0
H.Johnson (Miramar)	2006/09/13		-6.3	-7.3	-8.2	-9.0	-9.4

DORIS NORTH PROJECT THERMISTOR DATA

Drill Hole SRK-19

		Bead No.	Bead 1	Bead 2	Bead 3	Bead 4	Bead 5	Bead 6
Read By	Date	Bead Location from Top (m)	2.0	3.0	4.0	6.0	8.5	11.0
		Bead Depth (m)	1.0	2.0	3.0	5.0	7.5	10.0
Dan Mackie (SRK)	2003/04/14	Temperature (Celsius)	-13.4	-11.8	-9.2	-7.9	-7.9	-8.0
Dan Mackie (SRK)	2003/04/16		-13.9	-11.5	-9.0	-7.0	-7.3	-7.6
Jay Hallman (Miramar)	2003/05/17		-9.7	-10.1	-9.3	-7.1	-7.3	-7.6
Dylan MacGregor (SRK)	2003/08/25		-0.9	-4.2	-6.2	-7.4	-7.5	-7.6
Mike Cripps (Miramar)	2003/09/21		-0.7	-3.7	-5.7	-7.3	-7.4	-7.6
Dylan MacGregor (SRK)	2004/04/16		-15.5	-13.4	-10.5	-6.7	-7.3	-7.5
Dylan MacGregor (SRK)	2004/08/26		-1.0	-4.3	-6.6	-7.7	-7.5	-7.5
Quinn Jordan-Knox (SRK)	2004/09/28		-0.9	-3.7	-5.9	-7.4	-7.6	-7.5
Dylan MacGregor (SRK)	2005/04/16		-14.6	-13.5	-11.7	-7.6	-7.5	-7.6
D Kary (Miramar)	2005/05/16		-12.4	-11.9	-11.2	-8.2	-7.5	-7.6
Gabrielle (Miramar)	2005/07/17		-1.9	-5.1	-8.0	-8.5	-7.7	-7.6
JRH (Miramar)	2005/08/05		-1.3	-4.8	-7.3	-8.3	-7.8	-7.6
D Kary (Miramar)	2005/09/08		-0.7	-4.0	-6.4	-8.0	-7.9	-7.6
E Ballent (Miramar)	2005/09/26		-0.7	-3.7	-6.1	-7.8	-7.9	-7.6
L. Wade / A. Tong (SRK)	2006/04/30		-12.6	-11.7	-10.6	-7.8	-7.7	-7.8
Jay Hallman (Miramar)	2006/06/04		-3.5	-7.1	-9.0	-8.1	-7.7	-7.7
Gary S./ Hugh J. (Miramar)	2006/08/06		-0.5	-4.1	-6.5	-7.8	-7.8	-7.6
J.Hallman (Miramar)	2006/09/13		0.0	-3.5	-6.0	-7.6	-7.9	-7.7

DORIS NORTH PROJECT THERMISTOR DATA

Drill Hole SRK-20		**Bead No.**	**Bead 1**	**Bead 2**	**Bead 3**	**Bead 4**	**Bead 5**	**Bead 6**
Read By	**Date**	**Bead Location from Top (m)**	**2.0**	**3.0**	**4.0**	**6.0**	**8.5**	**11.0**
		Bead Depth (m)	**0.8**	**1.8**	**2.8**	**4.8**	**7.3**	**9.8**
Dan Mackie (SRK)	2003/04/13	**Temperature (Celsius)**	-12.0	-10.2	-8.2	-6.7	-7.1	-7.4
Dan Mackie (SRK)	2003/04/14		-11.8	-11.1	-9.3	-7.0	-7.1	-7.4
Dan Mackie (SRK)	2003/04/16		-12.7	-11.0	-9.4	-6.9	-7.0	-7.4
Jay Hallman (Miramar)	2003/05/17		-9.3	-9.9	-9.3	-7.1	-7.1	-7.4
Dylan MacGregor (SRK)	2003/08/25		0.3	-3.0	-5.3	-7.2	-7.2	-7.4
Mike Cripps (Miramar)	2003/09/21		0.0	-2.7	-4.9	-7.0	-7.2	-7.3
Dylan MacGregor (SRK)	2004/04/16		-15.1	-13.4	-11	-6.9	-7.1	-7.2
Dylan MacGregor (SRK)	2004/08/26		0.0	-3.3	-5.8	-7.7	-7.3	-7.1
Quinn Jordan-Knox (SRK)	2004/09/28		-0.2	-2.8	-5.1	-7.4	-7.3	-7.1
Dylan MacGregor (SRK)	2005/04/16		-13.5	-12.6	-11.1	-7.6	-7.2	-7.2
D Kary (Miramar)	2005/05/16		-11.8	-11.4	-10.6	-8.1	-7.2	-7.2
Gabrielle (Miramar)	2005/07/17		-0.7	-4.4	-7.0	-8.3	-7.4	-7.2
JRH (Miramar)	2005/08/05		-0.3	-3.7	-6.3	-8.1	-7.5	-7.2
D Kary (Miramar)	2005/09/08		0.3	-3.0	-5.4	-7.7	-7.5	-7.2
E Ballent (Miramar)	2005/09/26		-0.1	-2.7	-5.1	-7.5	-7.5	-7.2
L. Wade / A. Tong (SRK)	2006/04/30		-12	-11.5	-11.3	-7.7	-7.4	-7.4
Jay Hallman (Miramar)	2006/06/04		-2.2	-6.1	-8.2	-8.0	-7.3	-7.3
Gary S./Hugh J. (Miramar)	2006/08/06		1.8	-3.1	-5.6	-7.6	-7.4	-7.2
J.Hallman (Miramar)	2006/09/04		0.8	-2.6	-5.0	-7.3	-7.5	-7.3

DORIS NORTH PROJECT THERMISTOR DATA

Drill Hole SRK-22		Bead No.	Bead 1	Bead 2	Bead 3	Bead 4	Bead 5	Bead 6
Read By	**Date**	**Bead Location from Top (m)**	**2.0**	**3.0**	**4.0**	**6.0**	**8.5**	**11.0**
		Bead Depth (m)	**0.7**	**1.7**	**2.7**	**4.7**	**7.2**	**9.7**
Dan Mackie (SRK)	2003/04/13	**Temperature (Celsius)**	-11.6	-11.5	-9.3	-7.3	-7.5	-7.8
Dan Mackie (SRK)	2003/04/14		-11.8	-11.4	-9.5	-7.3	-7.6	-7.9
Dan Mackie (SRK)	2003/04/15		-12.3	-11.3	-9.5	-7.3	-7.6	-7.9
Dan Mackie (SRK)	2003/04/16		-12.7	-11.4	-9.5	-7.3	-7.6	-7.9
Jay Hallman (Miramar)	2003/05/17		-10.3	-10.5	-9.7	-7.7	-7.7	-7.9
Dylan MacGregor (SRK)	2003/08/25		-2.4	-5.6	-7.4	-7.9	-7.9	-8.0
Mike Cripps (Miramar)	2003/09/21		-2.1	-5.1	-6.9	-7.8	-7.9	-8.0
Dylan MacGregor (SRK)	2004/04/16		-14.4	-12.0	-9.3	-7.4	-7.7	-7.8
Thorpe/Lindsay (Miramar)	2004/05/17		-12.8	-11.7	-9.9	-7.7	-7.7	-7.8
Dylan MacGregor (SRK)	2004/08/27		-2.8	-6.0	-7.8	-8.2	-7.8	-7.8
Quinn Jordan-Knox (SRK)	2004/09/28		-2.6	-5.3	-7.2	-8.1	-7.9	-7.8
Dylan MacGregor (SRK)	2005/04/16		-13.6	-12.0	-9.9	-7.8	-7.9	-7.9
D Kary (Miramar)	2005/05/16		-11.9	-11.2	-10.1	-8.2	-7.9	-7.9
Gabrielle (Miramar)	2005/07/17		-4.0	-7.3	-8.8	-8.6	-8.0	-7.9
JRH (Miramar)	2005/08/05		-3.2	-6.6	-8.3	-8.5	-8.0	-7.9
D Kary (Miramar)	2005/09/08		-2.4	-5.6	-7.6	-8.4	-8.1	-7.9
E Ballent (Miramar)	2005/09/26		-2.3	-5.3	-7.2	-8.2	-8.1	-7.9
L. Wade / A. Tong (SRK)	2006/04/30		-12.2	-11.2	-9.7	-8.0	-8.0	-8.1
Jay Hallman (Miramar)	2006/06/04		-5.7	-8.6	-9.3	-8.2	-8.0	-7.9
Gary S./Hugh J. (Miramar)	2006/08/06		-2.6	-5.9	-7.6	-8.2	-8.0	-7.9
J. Hallman (Miramar)	2006/09/04		-2.3	-5.5	-7.3	-8.1	-8.1	-8.0

DORIS NORTH PROJECT THERMISTOR DATA

Drill Hole SRK-23		**Bead No.**	**Bead 1**	**Bead 2**	**Bead 3**	**Bead 4**	**Bead 5**	**Bead 6**
Read By	**Date**	**Bead Location from Top (m)**	**2.0**	**3.0**	**4.0**	**6.0**	**8.5**	**11.0**
		Bead Depth (m)	**0.9**	**1.9**	**2.9**	**4.9**	**7.4**	**9.9**
Dan Mackie (SRK)	2003/04/14	**Temperature (Celsius)**	-13.2	-11.9	-9.7	-8.0	-8.0	-8.3
Dan Mackie (SRK)	2003/04/15		-13.0	-11.6	-9.3	-7.1	-7.5	-7.8
Dan Mackie (SRK)	2003/04/16		-13.2	-11.6	-9.3	-7.1	-7.5	-7.8
Jay Hallman (Miramar)	2003/05/17		-10.5	-10.5	-9.5	-7.6	-7.5	-7.8
Dylan MacGregor (SRK)	2003/08/25		-1.5	-4.4	-6.7	-7.8	-7.7	-7.8
Mike Cripps (Miramar)	2003/09/21		-1.3	-4.0	-6.2	-7.7	-7.8	-7.8
Dylan MacGregor (SRK)	2004/04/16		-16.2	-13.9	-10.8	-7.5	-7.6	-7.9
Thorpe/Lindsay (Miramar)	2004/05/17		-12.9	-12.6	-11.1	-8.0	-7.6	-7.8
Dylan MacGregor (SRK)	2004/08/27		-1.7	-4.8	-7.2	-8.3	-7.8	-7.8
Quinn Jordan-Knox (SRK)	2004/09/28		-1.5	-4.2	-6.6	-8.1	-7.9	-7.8
Dylan MacGregor (SRK)	2005/04/16		-14.0	-12.9	-9.9	-7.8	-7.8	-7.9
D Kary (Miramar)	2005/05/16		-12.2	-11.7	-10.6	-8.4	-7.8	-7.9
Gabrielle (Miramar)	2005/07/17		-2.6	-6.0	-8.4	-8.7	-7.9	-7.9
JRH (Miramar)	2005/08/05		-2.0	-5.3	-7.7	-8.6	-8.0	-7.9
D Kary (Miramar)	2005/09/08		-1.4	-4.4	-6.9	-8.3	-8.1	-7.9
E Ballent (Miramar)	2005/09/26		-1.3	-4.1	-6.5	-8.2	-8.1	-7.9
L. Wade / A. Tong (SRK)	2006/04/30		-12.2	-11.4	-9.9	-8.0	-7.9	-8.0
Jay Hallman (Miramar)	2006/06/04		-4.0	-7.3	-8.9	-8.2	-7.8	-7.9
Gary S./ Hugh J.	2006/08/06		-1.4	-4.5	-6.8	-8.1	-7.9	-7.9
J.Hallman (Miramar)	2006/09/04		-1.0	-3.9	-6.3	-7.9	-8.0	-8.0

DORIS NORTH PROJECT THERMISTOR DATA

Drill Hole SRK-24		Bead No.	Bead 1	Bead 2	Bead 3	Bead 4	Bead 5	Bead 6
Read By	**Date**	**Bead Location from Top (m)**	**2.0**	**3.0**	**4.0**	**6.0**	**8.5**	**11.0**
		Bead Depth (m)	**0.7**	**1.7**	**2.7**	**4.7**	**7.2**	**9.7**
Dan Mackie (SRK)	2003/04/13	**Temperature (Celsius)**	-11.1	-9.9	-8.0	-7.1	-7.3	-7.4
Dan Mackie (SRK)	2003/04/14		-10.0	-11.0	-9.4	-7.2	-7.3	-7.4
Dan Mackie (SRK)	2003/04/15		-10.4	-10.9	-9.5	-7.4	-7.3	-7.4
Dan Mackie (SRK)	2003/04/16		-10.9	-10.8	-9.5	-7.3	-7.4	-7.4
Jay Hallman (Miramar)	2003/05/17		-10.0	-10.1	-9.4	-7.7	-7.5	-7.6
Dylan MacGregor (SRK)	2003/08/25		-1.4	-4.5	-6.6	-8.1	-8.0	-8.0
Mike Cripps (Miramar)	2003/09/21		-1.2	-4.2	-6.2	-7.6	-7.6	-7.6
Dylan MacGregor (SRK)	2004/04/16		-16.0	-14.0	-11.3	-7.6	-7.4	-7.5
Thorpe/Lindsay (Miramar)	2004/05/17		-13.3	-13.0	-11.4	-8.2	-7.4	-7.5
Dylan MacGregor (SRK)	2004/08/27		-1.7	-5.2	-7.3	-8.3	-7.7	-7.4
Quinn Jordan-Knox (SRK)	2004/09/28		-1.6	-4.6	-6.6	-8.0	-7.8	-7.4
Dylan MacGregor (SRK)	2005/04/16		-14.1	-13.2	-11.1	-8.1	-7.7	-7.5
D Kary (Miramar)	2005/05/16		-13.9	-13.8	-13.5	-11.0	-9.1	-8.8
Jay Hallman (Miramar)	2005/07/18		-2.5	-6.3	-8.3	-8.7	-7.9	-7.5
JRH (Miramar)	2005/08/05		-2.0	-5.6	-7.7	-8.6	-7.9	-7.5
D Kary (Miramar)	2005/09/08		-1.3	-4.8	-6.9	-8.3	-8.0	-7.5
E Ballent (Miramar)	2005/09/26		-1.3	-4.4	-6.5	-8.1	-8.0	-7.5
L. Wade / A. Tong (SRK)	2006/04/30		-12.4	-12.1	-10.6	-8.2	-7.8	-7.6
Jay Hallman (Miramar)	2006/06/04		-3.9	-7.8	-9.3	-8.5	-7.8	-7.5
Gary S./Hugh J. (Miramar)	2006/08/06		-1.4	-5.1	-7.1	-8.2	-7.9	-7.5
H.Johnson (Miramar)	2006/09/13		-1.1	-4.4	-6.5	-7.9	-8.0	-7.6

DORIS NORTH PROJECT THERMISTOR DATA

Drill Hole SRK-26		Bead No.	Bead 1	Bead 2	Bead 3	Bead 4	Bead 5	Bead 6
Read By	**Date**	**Bead Location from Top (m)**	**2.0**	**3.0**	**4.0**	**6.0**	**8.5**	**11.0**
		Bead Depth (m)	**0.8**	**1.8**	**2.8**	**4.8**	**7.3**	**9.8**
Dan Mackie (SRK)	2003/04/13	**Temperature (Celsius)**	-14.0	-13.1	-11.0	-8.5	-8.4	-8.7
Dan Mackie (SRK)	2003/04/14		-13.9	-13.1	-11.0	-8.6	-8.4	-8.7
Dan Mackie (SRK)	2003/04/16		-14.8	-13.7	-12.3	-9.0	-8.4	-8.7
Jay Hallman (Miramar)	2003/05/17		-11.7	-12.2	-11.7	-9.6	-8.5	-8.7
Dylan MacGregor (SRK)	2003/08/25		-1.6	-4.6	-6.8	-9.0	-8.9	-8.7
Mike Cripps (Miramar)	2003/09/21		-1.2	-4.0	-6.0	-7.8	-8.0	-8.0
Dylan MacGregor (SRK)	2004/04/16		-18.8	-17.4	-15.0	-10.0	-8.5	-8.7
Thorpe/Lindsay (Miramar)	2004/05/17		-14.3	-15.1	-14.3	-10.8	-8.7	-8.7
Dylan MacGregor (SRK)	2004/08/27		-1.9	-4.8	-7.5	-9.8	-9.2	-8.8
Dylan MacGregor (SRK)	2005/04/19		-16.4	-15.6	-14.5	-10.6	-8.9	-8.9
D Kary (Miramar)	2005/05/16		-13.9	-13.8	-13.5	-11.0	-9.1	-8.8
Gabrielle (Miramar)	2005/07/17		-3.1	-6.5	-9.1	-10.6	-9.5	-8.9
JRH (Miramar)	2005/08/05		-2.4	-5.6	-8.2	-10.2	-9.5	-9.0
D Kary (Miramar)	2005/09/08		-1.6	-4.5	-7.1	-9.6	-9.5	-9.0
E Ballent (Miramar)	2005/09/26		-1.5	-4.1	-6.6	-9.3	-9.5	-9.1
L. Wade / A. Tong (SRK)	2006/04/30		-14.2	-13.7	-12.9	-10.3	-9.0	-9.0
Jay Hallman (Miramar)	2006/06/04		-4.7	-8.3	-10.5	-10.4	-9.1	-8.9
Gary S./Hugh J. (Miramar)	2006/08/06		-1.5	-4.6	-7.2	-9.5	-9.3	-9.0
H.Johnson (Miramar)	2006/09/13		-0.9	-3.7	-6.3	-8.9	-9.3	-9.1

DORIS NORTH PROJECT THERMISTOR DATA

Drill Hole SRK-28		Bead No.	Bead 1	Bead 2	Bead 3	Bead 4	Bead 5	Bead 6
Read By	**Date**	**Bead Location from Top (m)**	**2.0**	**3.0**	**4.0**	**6.0**	**8.5**	**11.0**
		Bead Depth (m)	**0.8**	**1.8**	**2.8**	**4.8**	**7.3**	**9.8**
Dan Mackie (SRK)	2003/04/13	**Temperature (Celsius)**	-12.1	-10.5	-8.6	-7.4	-7.8	-7.9
Dan Mackie (SRK)	2003/04/14		-12.0	-10.5	-8.6	-7.5	-7.8	-8.0
Dan Mackie (SRK)	2003/04/16		-13.3	-11.8	-10.2	-7.6	-7.7	-7.9
Jay Hallman (Miramar)	2003/05/17		-9.8	-10.6	-10.0	-8.2	-7.7	-8.0
Dylan McGreggor (SRK)	2003/08/25		-1.5	-4.5	-6.6	-8.1	-8.0	-8.0
Mike Cripps (Miramar)	2003/09/21		-1.3	-4.1	-6.2	-8.7	-8.9	-8.8
Dylan MacGregor (SRK)	2004/04/16		-16.7	-14.7	-12.0	-8.1	-7.7	-8.0
Thorpe/Lindsay (Miramar)	2004/05/17		-12.9	-13.2	-11.9	-8.8	-7.7	-7.9
Dylan MacGregor (SRK)	2004/08/27		-1.9	-5.0	-7.1	-8.6	-8.1	-8.0
Quinn Jordan-Knox (SRK)	2004/09/28		-1.7	-4.3	-6.4	-8.3	-8.2	-8.0
Dylan MacGregor (SRK)	2005/04/19		-14.6	-13.8	-12.2	-8.9	-8.0	-8.0
D Kary (Miramar)	2005/05/16		-12.2	-12.2	-11.6	-9.3	-8.1	-8.0
Gabrielle (Miramar)	2005/07/17		-2.8	-6.2	-8.4	-9.3	-8.3	-8.0
JRH (Miramar)	2005/08/05		-2.2	-5.5	-7.7	-9.1	-8.4	-8.0
D Kary (Miramar)	2005/09/08		-1.5	-4.6	-6.9	-8.7	-8.4	-8.1
E Ballent (Miramar)	2005/09/26		-1.4	-4.2	-6.4	-8.4	-8.4	-8.1
L. Wade / A. Tong (SRK)	2006/04/30		-12.3	-12.0	-10.8	-8.7	-8.1	-8.2
Jay Hallman (Miramar)	2006/06/04		-4.1	-7.6	-9.2	-8.9	-8.1	-8.1
Gary S./Hugh J. (Miramar)	2006/08/06		-1.5	-4.7	-6.9	-8.5	-8.3	-8.1
H.Johnson (Miramar)	2006/09/13		-1.0	-3.9	-6.1	-8.1	-8.3	-8.2

DORIS NORTH PROJECT THERMISTOR DATA

Drill Hole SRK-32		Bead No.	Bead 1	Bead 2	Bead 3	Bead 4	Bead 5	Bead 6
Read By	**Date**	**Bead Location from Top (m)**	**2.0**	**3.0**	**4.0**	**6.0**	**8.5**	**11.0**
		Bead Depth (m)	**0.9**	**1.9**	**2.9**	**4.9**	**7.4**	**9.9**
Sebastian Fortin (SRK)	2003/04/06	**Temperature (Celsius)**	-11.5	-9.7	-8.9	-8.7	-8.7	-8.1
Dylan MacGregor (SRK)	2003/04/09		-12.0	-11.0	-8.8	-8.6	-8.5	-8.2
Dan Mackie (SRK)	2003/04/13		-12.4	-11.9	-10.1	-8.6	-8.5	-8.3
Dan Mackie (SRK)	2003/04/15		-12.7	-11.9	-10.2	-8.6	-8.5	-8.3
Dan Mackie (SRK)	2003/04/16		-12.9	-11.9	-10.3	-8.5	-8.4	-8.3
Dan Mackie (SRK)	2003/04/20		-13.2	-12.0	-10.4	-8.6	-8.4	-8.3
Jay Hallman (Miramar)	2003/05/16		-11.4	-11.4	-10.7	-8.7	-8.3	-8.3
Dylan MacGregor (SRK)	2003/08/25		0.4	-3.2	-5.6	-8.3	-8.6	-8.4
Mike Cripps (Miramar)	2003/09/21		-0.1	-2.8	-5.0	-7.4	-8.6	-8.4
Dylan MacGregor (SRK)	2004/04/11		-18.3	-16.8	-14.3	-9.3	-8.0	-8.3
Thorpe/Lindsay (Miramar)	2004/05/17		-13.5	-14.2	-13.5	-10.2	-8.3	-8.3
Dylan MacGregor (SRK)	2004/08/27		0.0	-3.6	-6.1	-8.9	-8.9	-8.4
Quinn Jordan-Knox (SRK)	2004/09/28		-0.5	-3.2	-5.4	-8.3	-8.8	-8.5
Dylan MacGregor (SRK)	2005/04/16		-15.2	-14.7	-13.3	-9.7	-8.4	-8.4
D Kary (Miramar)	2005/05/16		-13.0	-13.0	-12.4	-10.2	-8.6	-8.4
Jay Hallman (Miramar)	2005/07/18		0.1	-4.9	-7.5	-9.6	-9.0	-8.5
JRH (Miramar)	2005/08/05		0.5	-4.1	-6.7	-9.2	-9.0	-8.5
D Kary (Miramar)	2005/09/08		0.8	-3.2	-5.7	-8.6	-9.0	-8.6
E Ballent (Miramar)	2005/09/26		-0.2	-3.0	-5.3	-8.2	-9.0	-8.7
L. Wade / A. Tong (SRK)	2006/04/30		-13.0	-13.2	-12.1	-9.5	-8.5	-8.6
Jay Hallman (Miramar)	2006/06/04		-2.1	-6.8	-9.1	-9.6	-8.6	-8.5
Gary S./Hugh J. (Miramar)	2006/08/06		2.6	-3.0	-5.5	-8.2	-8.4	
H.Johnson (Miramar)	2006/09/13		0.8	-2.7	-5.0	-7.9	-8.7	-8.6

DORIS NORTH PROJECT THERMISTOR DATA

Drill Hole SRK-33		**Bead No.**	**Bead 1**	**Bead 2**	**Bead 3**	**Bead 4**	**Bead 5**	**Bead 6**
Read By	**Date**	**Bead Location from Top (m)**	**2.0**	**3.0**	**4.0**	**6.0**	**8.5**	**11.0**
		Bead Depth (m)	**0.9**	**1.9**	**2.9**	**4.9**	**7.4**	**9.9**
Sebastian Fortin (SRK)	2003/04/06	**Temperature (Celsius)**	-10.7	-6.9	-6.4	-5.3	-5.2	-5.0
Dylan MacGregor (SRK)	2003/04/09		-13.1	-12.0	-10.0	-7.3	-7.1	-7.2
Dan Mackie (SRK)	2003/04/13		-12.4	-13.0	-12.5	-9.3	-7.8	-7.9
Dan Mackie (SRK)	2003/04/15		-13.1	-13.1	-12.5	-9.6	-8.0	-8.1
Dan Mackie (SRK)	2003/04/16		-13.4	-13.1	-12.5	-9.7	-8.0	-8.2
Dan Mackie (SRK)	2003/04/20		-13.5	-13.3	-12.5	-9.9	-8.1	-8.3
Jay Hallman (Miramar)	2003/05/16		-10.7	-11.7	-11.8	-10.4	-8.5	-8.6
Dylan MacGregor (SRK)	2003/08/25		-3.4	-5.4	-7.0	-8.9	-9.5	-8.8
Mike Cripps (Miramar)	2003/09/21		-2.3	-4.7	-6.4	-8.5	-8.9	-8.8
Dylan MacGregor (SRK)	2004/04/11		-19.1	-16.8	-14.3	-9.9	-8.6	-8.8
Thorpe/Lindsay (Miramar)	2004/05/17		-15.3	-15.0	-13.9	-10.7	-8.7	-8.7
Dylan MacGregor (SRK)	2004/08/27		-3.8	-6.0	-7.8	-9.6	-9.1	-8.8
Quinn Jordan-Knox (SRK)	2004/09/28		-3.2	-5.1	-6.8	-9.0	-9.1	-8.8
Dylan MacGregor (SRK)	2005/04/16		-17.1	-15.7	-14.0	-10.4	-8.8	-8.8
D Kary (Miramar)	2005/05/16		-14.4	-14.0	-13.3	-10.9	-8.9	-8.8
Jay Hallman (Miramar)	2005/07/18		-5.7	-8.0	-9.6	-10.4	-8.8	-8.8
JRH (Miramar)	2005/08/05		-4.6	-6.9	-8.6	-10.1	-9.3	-8.9
D Kary (Miramar)	2005/09/08		-3.4	-5.5	-7.4	-9.5	-9.3	-8.9
E Ballent (Miramar)	2005/09/26		-3.0	-5.0	-6.8	-9.1	-9.3	-9.0
L. Wade / A. Tong (SRK)	2006/04/30		-14.6	-13.7	-12.6	-10.2	-9.0	-9.0
Jay Hallman (Miramar)	2006/06/04		-8.1	-10.1	-11.0	-10.4	-9.0	-8.9
Gary S./Hugh J. (Miramar)	2006/08/06		-3.1	-5.3	-7.2	-9.1	-8.9	-8.7
H.Johnson (Miramar)	2006/09/13		-2.5	-4.5	-6.4	-8.8	-9.2	-9.0

DORIS NORTH PROJECT THERMISTOR DATA

Drill Hole SRK-34A

Read By	Date	Bead No.	Bead 1	Bead 2	Bead 3	Bead 4	Bead 5	Bead 6
		Bead Location from Top (m)	2.0	3.0	4.0	6.0	8.5	11.0
		Bead Depth (m)	0.9	1.9	2.9	4.9	7.4	9.9
Sebastian Fortin (SRK)	2003/04/06	Temperature (Celsius)	-10.3	-7.4	-7.4	-7.1	-6.1	-7.2
Dylan MacGregor (SRK)	2003/04/09		-13.1	-12.0	-10.0	-7.3	-7.1	-7.2
Dan Mackie (SRK)	2003/04/13		-12.4	-13.0	-12.5	-9.3	-7.8	-7.9
Dan Mackie (SRK)	2003/04/15		-13.1	-13.1	-12.5	-9.6	-8.0	-8.1
Dan Mackie (SRK)	2003/04/16		-13.4	-13.1	-12.5	-9.7	-8.0	-8.2
Dan Mackie (SRK)	2003/04/20		-13.5	-13.3	-12.5	-9.9	-8.1	-8.3
Jay Hallman (Miramar)	2003/05/16		-10.5	-10.8	-10.3	-8.5	-7.5	-7.7
Dylan MacGregor (SRK)	2003/08/25		-1.8	-4.2	-6.1	-8.0	-8.1	-8.0
Mike Cripps (Miramar)	2003/09/21		-1.5	-3.7	-5.5	-7.7	-8.0	-8.1
Dylan MacGregor (SRK)	2004/04/11		-17.6	-15.3	-12.2	-8.8	-7.4	-7.6
Thorpe/Lindsay (Miramar)	2004/05/17		-13.8	-13.6	-12.0	-9.2	-7.7	-7.7
Dylan MacGregor (SRK)	2004/08/27		-2.2	-4.8	-6.4	-8.7	-8.5	-8.3
Quinn Jordan-Knox (SRK)	2004/09/28		-1.9	-4.2	-5.6	-8.3	-8.4	-8.3
Dylan MacGregor (SRK)	2005/04/16		-14.8	-13.6	-11.4	-8.6	-7.8	-7.9
D Kary (Miramar)	2005/05/16		-12.6	-12.2	-11.0	-9.2	-8.0	-8.0
Jay Hallman (Miramar)	2005/07/18		-3.4	-6.2	-7.6	-9.2	-8.5	-8.3
JRH (Miramar)	2005/08/05		-2.7	-5.4	-6.9	-8.9	-8.5	-8.3
D Kary (Miramar)	2005/09/08		-1.9	-4.4	-6.0	-8.5	-8.5	-8.4
E Ballent (Miramar)	2005/09/26		-1.7	-4.0	-5.5	-8.2	-8.4	-8.4
L. Wade / A. Tong (SRK)	2006/04/30		-13.2	-12.4	-10.7	-8.7	-7.9	-8.0
Jay Hallman (Miramar)	2006/06/04		-5.5	-8.2	-9.1	-9.0	-8.1	-8.0
Gary S./Hugh J. (Miramar)	2006/08/06		-1.8	-4.4	-6.0	-8.1	-8.1	-8.0
H.Johnson (Miramar)	2006/09/13		-1.5	-3.9	-5.4	-8.0	-8.3	-8.3

DORIS NORTH PROJECT THERMISTOR DATA

Drill Hole SRK-35		**Bead No.**	**Bead 1**	**Bead 2**	**Bead 3**	**Bead 4**	**Bead 5**	**Bead 6**
Read By	**Date**	**Bead Location from Top (m)**	**2.0**	**3.0**	**4.0**	**6.0**	**8.5**	**11.0**
		Bead Depth (m)	**0.4**	**1.4**	**2.4**	**4.4**	**6.9**	**9.4**
Sebastian Fortin (SRK)	2003/04/08	**Temperature (Celsius)**	-6.8	-3.7	-2.9	-4.3	-5.1	-5.0
Dan Mackie (SRK)	2003/04/13		-7.4	-6.1	-4.5	-5.4	-6.0	-6.3
Dan Mackie (SRK)	2003/04/14		-7.5	-6.2	-4.6	-5.4	-6.0	-6.3
Dan Mackie (SRK)	2003/04/16		-10.3	-7.5	-6.0	-5.0	-5.9	-6.2
Jay Hallman (Miramar)	2003/05/17		-6.4	-7.0	-6.4	-5.3	-5.9	-6.3
Dylan MacGregor (SRK)	2003/08/25		-0.4	-3.1	-4.8	-5.6	-6.0	-6.3
Mike Cripps (Miramar)	2003/09/21		-0.4	-2.7	-4.4	-5.5	-6.0	-6.3
Dylan MacGregor (SRK)	2004/04/11		-13.5	-11.0	-8.0	-5.4	-6.0	-6.2
Thorpe/Lindsay (Miramar)	2004/05/17		-10.6	-10.0	-8.4	-5.7	-5.9	-6.0
Dylan MacGregor (SRK)	2004/08/23		-0.5	-3.6	-5.6	-6.0	-5.9	-6.1
Quinn Jordan-Knox (SRK)	2004/09/26		-0.5	-3.1	-5.0	-6.0	-6.1	-6.1
Dylan MacGregor (SRK)	2005/04/16		-10.9	-9.9	-8.2	-5.9	-6.0	-6.1
D Kary (Miramar)	2005/05/16		-9.7	-9.1	-8.2	-6.1	-6.0	-6.1
Jay Hallman (Miramar)	2005/07/18		-1.3	-4.5	-6.4	-6.4	-6.1	-6.1
JRH (Miramar)	2005/08/05		-0.7	-3.9	-5.9	-6.3	-6.1	-6.1
D Kary (Miramar)	2005/09/08		-0.1	-3.3	-5.3	-6.3	-6.1	-6.1
E Ballent (Miramar)	2005/09/26		-0.4	-3.0	-5.0	-6.2	-6.1	-6.1
L. Wade / A. Tong (SRK)	2006/04/30		-9.9	-9.0	-7.7	-6.1	-6.2	-6.3
Jay Hallman (Miramar)	2006/06/04		-3.1	-5.8	-7.0	-6.2	-6.1	-6.2
Gary S./Hugh J. (Miramar)	2006/08/06		0.1	-3.4	-5.4	-6.2	-6.2	-6.2
H.Johnson (Miramar)	2006/09/13		0.5	-2.8	-4.8	-6.1	-6.2	-6.2

DORIS NORTH PROJECT THERMISTOR DATA

Drill Hole SRK-37		Bead No.	Bead 1	Bead 2	Bead 3	Bead 4	Bead 5	Bead 6	Bead 7	Bead 8	Bead 9	Bead 10
Read By	**Date**	**Bead Location from Top (m)**	**6.0**	**11.0**	**13.5**	**16.0**	**18.5**	**21.0**	**23.5**	**26.0**	**28.5**	**31.0**
		Inclined Bead Depth (m)	**0.0**	**4.6**	**7.1**	**9.6**	**12.1**	**14.6**	**17.1**	**19.6**	**22.1**	**24.6**
		Vert. Bead Depth (m)	**0.0**	**3.3**	**5.0**	**6.8**	**8.6**	**10.4**	**12.1**	**13.9**	**15.7**	**17.4**
Sebastian Fortin (SRK)	2003/04/06	**Temperature (Celsius)**	-14.7	-14.0	-12.2	-10.5	-9.3	-8.6	-8.2	-8.2	-8.4	-8.2
Dylan MacGregor (SRK)	2003/04/09		-20.9	-13.1	-11.1	-9.6	-8.8	-8.7	-8.1	-8.2	-8.3	-8.0
Dan Mackie (SRK)	2003/04/13		-23.0	-14.2	-12.4	-10.7	-9.5	-8.8	-8.3	-8.3	-8.4	-8.2
Dan Mackie (SRK)	2003/04/15		-20.9	-14.3	-12.5	-10.8	-9.6	-8.8	-8.3	-8.2	-8.3	-8.2
Dan Mackie (SRK)	2003/04/16		-13.3	-14.2	-12.5	-10.8	-9.6	-8.8	-8.3	-8.2	-8.3	-8.2
Dan Mackie (SRK)	2003/04/20		-10.3	-14.3	-12.6	-10.9	-9.7	-8.9	-8.4	-8.2	-8.3	-8.2
Jay Hallman (Miramar)	2003/05/16		-1.8	-13.4	-12.5	-11.3	-10.1	-9.3	-8.6	-8.3	-8.2	-8.2
Dylan MacGregor (SRK)	2003/08/25		13.0	-6.9	-7.9	-8.7	-9.1	-9.2	-8.9	-8.7	-8.4	-8.2
Mike Cripps (Miramar)	2003/09/21		-5.2	-6.1	-7.1	-7.9	-8.6	-8.8	-8.8	-8.6	-8.5	-8.3
Dylan MacGregor (SRK)	2004/04/11		-19.9	-17.0	-14.6	-12.0	-10.1	-9.0	-8.9	-8.1	-8.1	-8.1
Thorpe/Lindsay (Miramar)	2004/05/17		-3.2	-15.5	-14.4	-12.6	-11.0	-9.8	-9.0	-8.3	-8.2	-8.0
Dylan MacGregor (SRK)	2004/08/27		4.7	-7.4	-8.6	-9.4	-9.8	-9.7	-9.3	-8.9	-8.5	-8.2
Quinn Jordan-Knox (SRK)	2004/09/26		-1.4	-6.6	-7.5	-8.4	-9.0	-9.2	-9.1	-8.9	-8.6	-8.3
Dylan MacGregor (SRK)	2005/04/16		-13.8	-16.5	-14.6	-12.3	-10.6	-9.5	-8.8	-8.4	-8.3	-8.2
D Kary (Miramar)	2005/05/16		-4.1	-14.6	-13.9	-12.5	-11.1	-10.0	-9.1	-8.6	-8.3	-8.2
Jay Hallman (Miramar)	2005/07/18		11.9	-9.3	-10.3	-10.8	-10.7	-10.2	-9.5	-9.0	-8.6	-8.3
JRH (Miramar)	2005/08/05		14.0	-8.2	-9.4	-10.1	-10.3	-10.0	-9.5	-9.0	-8.6	-8.3
D Kary (Miramar)	2005/09/08		15.6	-6.9	-8.1	-9.0	-9.5	-9.6	-9.4	-9.0	-8.7	-8.4
E Ballent (Miramar)	2005/09/26		-1.7	-6.3	-7.4	-8.4	-9.1	-9.3	-9.2	-9.0	-8.7	-8.4
L. Wade / A. Tong (SRK)	2006/04/30		-7.4	-13.8	-12.7	-11.3	-10.1	-9.4	-8.8	-8.5	-8.4	-8.5
Jay Hallman (Miramar)	2006/06/04		9.6	-11.0	-11.4	-11.0	-10.3	-9.6	-9.0	-8.7	-8.4	-8.2
Gary S./Hugh J. (Miramar)	2006/08/06		16.6	-7.0	-8.1	-8.9	-9.3	-9.3	-9.0	-8.8	-8.5	-8.3

DORIS NORTH PROJECT THERMISTOR DATA

Drill Hole SRK-38		Bead No.	Bead 1	Bead 2	Bead 3	Bead 4	Bead 5	Bead 6	Bead 7	Bead 8
Read By	Date	Bead Location from Top (m)	above ground	6.0	11.0	16.0	21.0	31.0	41.0	51.0
		Bead Depth (m)	n/a	1.0	6.0	11.0	16.0	26.0	36.0	46.0
Dylan MacGregor (SRK)	2003/08/25	Temperature (Celsius)		1.5	-7.9	-7.7	-8.1	-8.1	-8.0	-8.1
Mike Cripps (Miramar)	2003/09/21			0.2	-7.9	-8.0	-8.2	-8.2	-8.1	-8.1
Dylan MacGregor (SRK)	2004/04/11			-17.6	-8.0	-8.2	-8.3	-8.2	-8.1	-8.1
Thorpe/Lindsay (Miramar)	2004/05/17			-13.4	-8.4	-8.2	-8.2	-8.2	-8.1	-8.1
Dylan MacGregor (SRK)	2004/08/27			0.3	-9.0	-8.2	-8.2	-8.2	-8.1	-8.1
Quinn Jordan-Knox (SRK)	2004/09/26			-0.5	-8.8	-8.2	-8.2	-8.2	-8.1	-8.1
Dylan MacGregor (SRK)	2005/04/16			-15.1	-8.5	-8.3	-8.3	-8.2	-8.1	-8.1
D Kary (Miramar)	2005/05/16			-12.4	-8.9	-8.3	-8.2	-8.2	-8.0	-8.1
Jay Hallman (Miramar)	2005/07/18			0.9	-9.3	-8.3	-8.2	-8.2	-8.0	-8.0
JRH (Miramar)	2005/08/05		12.0	1.4	-9.2	-8.3	-8.5	-8.1	-8.0	-8.0
D Kary (Miramar)	2005/09/08			0.9	-9.0	-8.3	-8.2	-8.2	-8.0	-8.0
E Ballent (Miramar)	2005/09/26		-2.1	-0.3	-8.9	-8.3	-8.3	-8.2	-8.0	-8.0
L. Wade / A. Tong (SRK)	2006/04/30		-8.1	-13.0	-8.5	-8.4	-8.4	-8.2	-8.1	-8.1
Jay Hallman (Miramar)	2006/06/04		10.7	-1.8	-8.7	-8.3	-8.3	-8.1	-8.0	-8.0
H.Johnson (Miramar)	2006/09/13		-1.7	1.3	-8.6	-8.3	-8.3	-8.2	-8.0	-8.0

DORIS NORTH PROJECT THERMISTOR DATA

Drill Hole SRK-39		Bead No.	Bead 1	Bead 2	Bead 3	Bead 4	Bead 5	Bead 6
Read By	**Date**	**Bead Location from Top (m)**	**21.0**	**26.6**	**32.1**	**43.1**	**54.1**	**66.0**
		Bead Depth (m)	**4.6**	**10.2**	**15.7**	**26.7**	**37.7**	**49.6**
Dylan MacGregor (SRK)	2003/08/25	**Temperature (Celsius)**	-1.2	-3.8	-1.1	-1.8	-1.9	-7.5
Mike Cripps (Miramar)	2003/09/21		-6.3	-7.7	-7.8	-8.0		-8.2
Dylan MacGregor (SRK)	2004/04/11		-15.8	-16.0	-7.1	-7.6	-7.8	-8.0
Thorpe/Lindsay (Miramar)	2004/05/17		-11.8	-12.3	-7.8	-7.6	-7.8	-8.0
Dylan MacGregor (SRK)	2004/08/27		4.5	4.2	-8.2	-7.9	-7.8	-8.0
Quinn Jordan-Knox (SRK)	2004/09/28		-2.7	-2.5	-8.0	-7.9	-7.8	-8.0
Dylan MacGregor (SRK)	2005/04/16		-13.9	-12.9	-7.9	-7.8	-7.9	-8.0
D Kary (Miramar)	2005/05/16		-8.3	-7.8	-7.8	-8.0		-8.1
Jay Hallman (Miramar)	2005/07/18		-8.6	-8.0	-7.8	-8.0		-8.1
JRH (Miramar)	2005/08/05		-14.6	-15.6	-8.5	-8.0	-7.9	-8.0
D Kary (Miramar)	2005/09/08		-8.2	-8.1	-7.9	-8.0		-8.1
E Ballent (Miramar)	2005/09/26		-8.1	-8.2	-7.9	-8.0		-8.1
L. Wade / A. Tong (SRK)	2006/04/30		-9.2	-10.7	-7.8	-7.9	-8.0	-8.1
Jay Hallman (Miramar)	2006/06/04		-8.1	-7.9	-7.9	-8.0		-8.1
H.Johnson (Miramar)	2006/09/13		-7.8	-8.0	-8.0	-8.0		-8.1

DORIS NORTH PROJECT THERMISTOR DATA

Drill Hole SRK-40

Read By	Date	Bead No.	Bead 1	Bead 2	Bead 3	Bead 4	Bead 5	Bead 6	Bead 7	Bead 8
		Bead Location from Top (m)	**above ground**	**6.0**	**11.0**	**16.0**	**21.0**	**31.0**	**41.0**	**51.0**
		Bead Depth (m)	**n/a**	**0.9**	**5.9**	**10.9**	**15.9**	**25.9**	**35.9**	**45.9**
Dylan McGreggor (SRK)	2003/08/25	**Temperature (Celsius)**		3.1	-8.2	-8.7	-8.5	-8.7	-8.8	-8.8
Mike Cripps (Miramar)	2003/09/21			1.0	-7.8	-8.7	-8.6	-8.7	-8.8	-8.9
Dylan MacGregor (SRK)	2004/04/11			-18.2	-10.0	-8.2	-8.5	-8.8	-8.9	-8.9
Thorpe/Lindsay (Miramar)	2004/05/17			-14.7	-10.9	-8.5	-8.5	-8.8	-8.9	-8.9
Dylan MacGregor (SRK)	2004/08/27			-0.1	-9.2	-9.0	-8.7	-8.7	-8.8	-8.8
Quinn Jordan-Knox (SRK)	2004/09/26			-1.1	-8.4	-9.0	-8.7	-8.7	-8.7	-8.8
Dylan MacGregor (SRK)	2005/04/16			-15.9	-10.5	-8.6	-8.7	-8.8	-8.8	-8.9
D Kary (Miramar)	2005/05/16			-13.6	-10.9	-8.8	-8.7	-8.7	-8.8	-8.8
Jay Hallman (Miramar)	2005/07/18			-0.5	-14.2	-9.2	-8.7	-8.7	-8.8	-8.8
JRH (Miramar)	2005/08/05		13.8	-0.9	-9.7	-9.3	-8.8	-8.7	-9.7	-8.8
D Kary (Miramar)	2005/09/08			0.5	-8.8	-9.2	-8.7	-8.7	-8.6	-8.6
E Ballent (Miramar)	2005/09/26			-0.5	-8.3	-9.1	-8.7	-8.5	-8.3	-8.4
L. Wade / A. Tong (SRK)	2006/04/30		-6.7	-13.7	-10.2	-8.7	-8.8	-8.8	-8.9	-8.9
Jay Hallman (Miramar)	2006/06/04		2.9	-3.5	-10.1	-8.6	-8.4	-8.7	-8.5	-8.5
Gary S./Hugh J. (Miramar)	2006/08/06		24.0	3.3	-8.3	-8.5	-8.3	-8.5	-8.4	-8.4
H.Johnson (Miramar)	2006/09/13		-1.1	2.1	-7.8	-8.7	-8.6	-8.6	-8.6	

DORIS NORTH PROJECT THERMISTOR DATA

Drill Hole SRK-41		Bead No.	Bead 1	Bead 2	Bead 3	Bead 4	Bead 5	Bead 6	Bead 7	Bead 8
Read By	**Date**	**Bead Location from Top (m)**	**3.5**	**6.0**	**8.5**	**11.0**	**13.5**	**16.0**	**18.5**	**21.0**
		Bead Depth (m)	**1.4**	**3.9**	**6.4**	**8.9**	**11.4**	**13.9**	**16.4**	**18.9**
Dylan MacGregor (SRK)	2003/08/25	**Temperature (Celsius)**	-16.0		-4.9	-6.2	-6.5	-6.5	-6.8	-7.0
Mike Cripps (Miramar)	2003/09/21		-15.4	-10.8	-10.0	-6.3	-6.7	-6.8	-7.0	-7.8
Dylan MacGregor (SRK)	2004/04/11		-18.5		-9.3	-6.6	-6.5	-6.8	-7.1	-7.2
Thorpe/Lindsay (Miramar)	2004/05/17		-2.1		-9.9	-7.3	-6.7	-6.8	-7.0	-7.2
Dylan MacGregor (SRK)	2004/08/27		6.2		-6.5	-7.3	-7.2	-7.1	-7.1	-7.1
Quinn Jordan-Knox (SRK)	2004/09/26		-1.3		-5.8	-7.0	-7.2	-7.1	-7.1	-7.2
Dylan MacGregor (SRK)	2005/04/16		-13.5		-7.8	-6.5	-6.6	-7.0	-7.1	-7.2
D Kary (Miramar)	2005/05/16		-9.4		-8.2	-6.9	-6.8	-7.0	-7.1	-7.2
Jay Hallman (Miramar)	2005/07/18		19.0		-6.9	-7.2	-7.0	-7.0	-7.1	-7.2
JRH (Miramar)	2005/08/05		20.2		-6.4	-7.1	-7.1	-7.0	-7.1	-7.2
D Kary (Miramar)	2005/09/08		23.6		-5.8	-6.9	-7.0	-7.1	-7.1	-7.2
E Ballent (Miramar)	2005/09/26		-1.0		-5.5	-6.7	-7.0	-7.1	-7.1	-7.2
L. Wade / A. Tong (SRK)	2006/04/30		-10.6		-8.7	-7.1	-6.9	-7.1	-7.2	-7.3
Jay Hallman (Miramar)	2006/06/04		15.6		-8.2	-7.3	-7.0	-7.0	-7.1	-7.2
Gary S./Hugh J. (Miramar)	2006/08/06		17.9		-6.1	-7.0	-7.1	-7.1	-7.1	-7.2
H.Johnson (Miramar)	2006/09/13		-0.2		-5.4	-6.6	-7.0	-7.1	-7.1	-7.2

DORIS NORTH PROJECT THERMISTOR DATA

Drill Hole SRK-42		Bead No.	Bead 1	Bead 2	Bead 3	Bead 4	Bead 5	Bead 6	Bead 7	Bead 8
Read By	**Date**	**Bead Location from Top (m)**	**above ground**	**above ground**	**11.0**	**16.0**	**21.0**	**31.0**	**41.0**	**51.0**
		Bead Depth (m)	**n/a**	**n/a**	**0.2**	**5.2**	**10.2**	**20.2**	**30.2**	**40.2**
Dylan McGreggor (SRK)	2003/08/25	**Temperature (Celsius)**			6.0	-6.3	-7.1	-7.8	-8.0	-8.1
Mike Cripps (Miramar)	2003/09/21				0.8	-6.3	-7.3	-7.9	-8.1	-8.1
Dylan MacGregor (SRK)	2004/04/11				-17.4	-7.6	-7.4	-8.0	-8.1	-8.1
Thorpe/Lindsay (Miramar)	2004/05/17				-11.3	-8.6	-7.4	-8.0	-8.1	-8.1
Dylan MacGregor (SRK)	2004/08/27				1.6	-7.7	-7.6	-8.0	-8.1	-8.1
Quinn Jordan-Knox (SRK)	2004/09/28				-0.1	-7.2	-7.0	-8.0	-8.1	-8.1
Dylan MacGregor (SRK)	2005/04/16				-14.1	-7.7	-7.6	-8.0	-8.1	-8.1
D Kary (Miramar)	2005/05/16				-11.4	-8.2	-7.5	-8.0	-8.1	-8.1
Jay Hallman (Miramar)	2005/07/18				2.9	-8.0	-7.6	-8.0	-8.1	-8.1
JRH (Miramar)	2005/08/05		16.4	18.9	1.7	-7.7	-7.6	-8.0	-8.1	-8.1
D Kary (Miramar)	2005/09/08				1.2	-7.2	-7.6	-8.0	-8.0	-8.1
E Ballent (Miramar)	2005/09/26				-0.1	-7.0	-7.6	-8.0	-8.1	-8.1
L. Wade / A. Tong (SRK)	2006/04/30		-5.5	-8.4	-12.1	-7.4	-7.6	-8.1	-8.1	-8.2
Jay Hallman (Miramar)	2006/06/04		14.8	12.2	3.6	-7.7	-7.4	-8.0	-8.0	-8.0
Gary S./Hugh J. (Miramar)	2006/08/06		26.4	27.1	1.9	-6.9	-7.5	-7.9	-8.0	-8.0
H.Johnson (Miramar)	2006/09/13		-0.2	-0.1	0.9	-6.5	-7.5	-7.9	-8.0	-8.0

DORIS NORTH PROJECT THERMISTOR DATA

Drill Hole SRK-43		**Bead No.**	**Bead 1**	**Bead 2**	**Bead 3**	**Bead 4**	**Bead 5**	**Bead 6**	**Bead 7**	**Bead 8**
Read By	**Date**	**Bead Location from Top (m)**	**above ground**	**15.5**	**21.0**	**26.6**	**32.1**	**43.1**	**54.1**	**66.0**
		Bead Depth (m)	**n/a**	**0.5**	**6.0**	**11.6**	**17.1**	**28.1**	**39.1**	**51.0**
Dylan MacGregor (SRK)	2003/08/25	**Temperature (Celsius)**		-2.3	-7.7	-7.8	-7.6	-8.2	-8.5	-8.5
Mike Cripps (Miramar)	2003/09/21			-2.6	-8.2	-8.3	-8.2	-8.5	-8.6	-8.5
Dylan MacGregor (SRK)	2004/04/11			-16.4	-8.2	-8.6	-8.7	-8.7	-8.6	-8.9
Thorpe/Lindsay (Miramar)	2004/05/17			-14.5	-8.4	-8.6	-8.8	-8.7	-8.6	-8.5
Dylan MacGregor (SRK)	2004/08/27			-5.2	-9.0	-8.6	-8.7	-8.7	-8.7	-8.5
Quinn Jordan-Knox (SRK)	2004/09/28			-4.5	-9.0	-8.6	-8.8	-8.7	-8.6	-8.5
Dylan MacGregor (SRK)	2005/04/16			-15.1	-8.6	-8.7	-8.8	-8.8	-8.6	-8.5
D Kary (Miramar)	2005/05/16			-13.3	-8.8	-8.7	-8.8	-8.7	-8.6	-8.5
Jay Hallman (Miramar)	2005/07/18			-6.4	-9.1	-8.7	-8.8	-8.7	-8.6	-8.5
JRH (Miramar)	2005/08/05		14.9	-5.6	-9.2	-8.7	-8.8	-8.7	-8.6	-8.5
D Kary (Miramar)	2005/09/08			-4.6	-9.2	-8.7	-8.7	-8.7	-8.5	-8.4
E Ballent (Miramar)	2005/09/26			-4.1	-9.1	-8.7	-8.8	-8.7	-8.6	-8.5
L. Wade / A. Tong (SRK)	2006/04/30		-10.5	-13.4	-8.8	-8.8	-8.9	-8.8	-8.6	-8.5
Jay Hallman (Miramar)	2006/06/04		13.1	-8.4	-8.8	-8.7	-8.8	-8.7	-8.5	-8.4
Gary S./Hugh J. (Miramar)	2006/08/06		21.0	-4.4	-8.7	-8.4	-8.4	-8.4	-8.2	-8.1
H.Johnson (Miramar)	2006/09/13		-0.7	-3.8	-9.0	-8.7	-8.8	-8.7	-8.6	-8.4

DORIS NORTH PROJECT THERMISTOR DATA

Drill Hole SRK-50		Bead No.	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
Read By	**Date**	**Bead Location from Top (m)**	**5.0**	**10.0**	**20.0**	**30.0**	**50.0**	**70.0**	**90.0**	**110.0**	**130.0**	**150.0**	**170.0**	**190.0**	**200.0**
		Bead Depth (m)	**5.0**	**10.0**	**20.0**	**30.0**	**50.0**	**70.0**	**90.0**	**110.0**	**130.0**	**150.0**	**170.0**	**190.0**	**200.0**
Dylan MacGregor (SRK)	2004/08/31	**Temperature (Celsius)**	-5.4	-6.0	-5.1	-4.9	-4.9	-5.0	-4.8	-4.7	-4.4	-4.3	-4.1	-3.8	-3.7
Quinn Jordan-Knox (SRK)	2004/09/26		-5.4	-6.4	-5.7	-5.4	-5.3	-5.3	-5.1	-5.0	-4.7	-4.5	-4.3	-3.9	-3.8
Dylan MacGregor (SRK)	2005/04/25		-10.3	-7.2	-5.9	-5.7	-5.6	-5.5	-5.3	-5.0	-4.8	-4.6	-4.4	-4.1	-4.0
D Kary (Miramar)	2005/05/16		-10.1	-7.5	-5.9	-5.7	-5.4	-5.4	-5.3	-5.0	-4.8	-4.6	-4.4	-4.0	-4.0
Jay Hallman (Miramar)	2005/07/18		-7.9	-7.6	-6.0	-5.7	-5.5	-5.4	-5.3	-5.1	-4.8	-4.6	-4.4	-4.1	-4.0
JRH (Miramar)	2005/08/05		-7.2	-7.5	-6.0	-5.7	-5.4	-5.4	-5.2	-5.0	-4.8	-4.6	-4.3	-4.0	-4.0
D Kary (Miramar)	2005/09/08		-6.3	-7.2	-6.1	-5.7	-5.5	-5.4	-5.3	-5.0	-4.8	-4.6	-4.4	-4.0	-4.0
E Ballent (Miramar)	2005/09/26		-5.8	-6.9	-6.1	-5.7	-5.4	-5.4	-5.2	-5.1	-4.8	-4.6	-4.4	-4.0	-3.9
L. Wade / A. Tong (SRK)	2006/04/30		-9.3	-7.0	-6.0	-5.8	-5.5	-5.4	-5.3	-5.1	-4.9	-4.7	-4.3	-4.0	-4.0
Jay Hallman (Miramar)	2006/06/04		-8.6	-7.2	-5.9	-5.7	-5.4	-5.3	-5.2	-5.0	-4.8	-4.6	-4.2	-3.9	-4.0
Gary S./Hugh J. (Miramar)	2006/08/06		-6.3	-6.9	-6.0	-5.7	-5.4	-5.3	-5.2	-5.0	-4.8	-4.6	-4.2	-3.9	-4.0
H.Johnson (Miramar)	2006/09/13		-5.4	-6.5	-6.0	-5.7	-5.4	-5.4	-5.2	-5.0	-4.8	-4.6	-4.2	-3.8	-4.0

DORIS NORTH PROJECT THERMISTOR DATA

Drill Hole SRK-51		**Bead No.**	**Bead 1**	**Bead 2**	**Bead 3**	**Bead 4**	**Bead 5**	**Bead 6**	**Bead 7**	**Bead 8**	**Bead 9**	**Bead 10**	**Bead 11**	**Bead 12**
Read By	**Date**	**Bead Location from Top (m)**	**above ground**	**1.25**	**1.50**	**1.75**	**2.00**	**2.50**	**3.00**	**3.50**	**4.00**	**4.50**	**5.00**	**6.00**
		Bead Depth (m)	**n/a**	**0.25**	**0.50**	**0.75**	**1.00**	**1.50**	**2.00**	**2.50**	**3.00**	**3.50**	**4.00**	**5.00**
Dylan MacGregor (SRK)	2005/04/26	**Temperature (Celsius)**	0.3	-8.2	-9.5	-10.7	-11.7	-12.9	-12.8	-12.2	-11.3	-10.4	-9.6	-8.6
D Kary (Miramar)	2005/05/16		-21.8	-10.7	-11.2	-11.3	-11.6	-11.9	-11.9	-11.8	-11.4	-10.9	-10.3	-9.3
Jay Hallman (Miramar)	2005/07/18		21.9	3.5	-0.5	-2.2	-3.7	-6.1	-7.8	-9.0	-9.7	-10.0	-10.0	-9.7
JRH (Miramar)	2005/08/05		20.9	4.0	0.4	-1.7	-3.1	-5.4	-7.1	-8.4	-9.1	-9.6	-9.8	-9.6
D Kary (Miramar)	2005/09/08		27.6	2.1	1.2	-1.0	-2.4	-4.6	-6.2	-7.5	-8.4	-9.0	-9.3	-9.4
E Ballent (Miramar)	2005/09/26		-5.1	-0.3	-0.3	-1.2	-2.2	-4.2	-5.8	-7.1	-8.0	-8.7	-9.0	-9.3
L. Wade / A. Tong (SRK)	2006/04/30		-2.9	-11.9	-12.7	-13.0	-13.2	-13.1	-12.5	-11.8	-11.1	-10.4	-9.7	-9.0
Jay Hallman (Miramar)	2006/06/04		18.6	2.3	-2.1	-4.0	-5.4	-7.8	-9.3	-10.2	-10.4	-10.3	-9.9	-9.3
Gary S./Hugh J. (Miramar)	2006/08/06		18.1	7.7	3.2	-0.6	-2.3	-4.7	-6.5	-7.7	-8.5	-9.1	-9.3	-9.3
H.Johnson (Miramar)	2006/09/13		-0.9	0.5	0.3	-0.2	-1.6	-3.9	-5.6	-6.9	-7.8	-8.5	-8.8	-9.1

DORIS NORTH PROJECT THERMISTOR DATA

Drill Hole SRK-52

Read By	Date	Bead No.	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 Location from Top (m)	above ground	1.25	1.50	1.75	2.00	2.50	3.00	3.50	4.00	4.50	5.00	6.00
		Bead Depth (m)	n/a	0.25	0.50	0.75	1.00	1.50	2.00	2.50	3.00	3.50	4.00	5.00
Dylan MacGregor (SRK)	2005/04/25	Temperature (Celsius)	6.0	-7.9	-10.5	-12.7	-14.5	-16.0	-16.3	-16.0	-15.3	-14.7	-13.8	-12.9
Dylan MacGregor (SRK)	2005/04/26		9.2	-8.8	-10.7	-12.5	-14.2	-15.9	-16.3	-16.0	-15.4	-14.8	-14.0	-13.1
D Kary (Miramar)	2005/05/16		-9.8	-9.5	-11.1	-11.9	-12.8	-13.9	-14.5	-14.8	-14.7	-14.4	-13.9	-13.4
Jay Hallman (Miramar)	2005/07/18		21.4	6.0	0.0	-2.7	-4.6	-7.0	-8.8	-10.0	-10.8	-11.2	-11.3	-11.7
JRH (Miramar)	2005/08/05		23.8	6.6	0.4	-2.2	-3.9	-6.2	-7.9	-9.1	-9.9	-10.4	-10.6	-11.0
D Kary (Miramar)	2005/09/08		21.2	10.3	2.3	-1.5	-3.1	-5.2	-6.9	-8.0	-8.8	-9.3	-9.5	-10.1
E Ballent (Miramar)	2005/09/26		-1.4	-1.2	-0.9	-1.5	-2.8	-4.8	-6.3	-7.4	-8.2	-8.7	-8.9	-9.6
L. Wade / A. Tong (SRK)	2006/04/30		-3.9	-11.4	-13.7	-14.6	-15.2	-15.5	-15.2	-14.8	-14.1	-13.5	-12.3	-12.2
Jay Hallman (Miramar)	2006/06/04		13.1	4.6	-2.0	-4.8	-6.6	-9.1	-10.8	-11.9	-12.2	-12.4	-12.1	-12.0
Gary S./Hugh J. (Miramar)	2006/08/06		17.9	12.9	4.4	-0.9	-3.1	-5.4	-7.1	-8.4	-9.0	-9.5	-9.7	-10.2
H.Johnson (Miramar)	2006/09/13		0.1	-0.2	0.3	-0.5	-2.2	-4.4	-5.8	-7.1	-7.8	-8.4	-8.6	-9.2

DORIS NORTH PROJECT THERMISTOR DATA

Drill Hole SRK-53		Bead No.	Bead 1	Bead 2	Bead 3	Bead 4	Bead 5	Bead 6
Read By	**Date**	**Bead Location from Top (m)**	**2.00**	**3.00**	**4.00**	**6.00**	**8.50**	**11.00**
		Bead Depth (m)	**0.60**	**1.60**	**2.60**	**4.60**	**7.10**	**9.60**
Dylan MacGregor (SRK)	2005/04/26	**Temperature (Celsius)**	-1.1	-0.8	-0.6	-0.3	-5.8	-5.9
D Kary (Miramar)	2005/05/16		-9.4	-9.7	-9.8	-8.2	-7.3	-6.7
Jay Hallman (Miramar)	2005/07/18		-0.6	-4.3	-6.5	-7.8	-7.6	-7.1
JRH (Miramar)	2005/08/05		0.3	-3.5	-5.8	-7.4	-7.5	-7.1
D Kary (Miramar)	2005/09/08		0.7	-2.7	-4.8	-6.8	-7.1	-7.0
E Ballent (Miramar)	2005/09/26		-0.1	-2.5	-4.5	-6.5	-6.9	-6.9
L. Wade / A. Tong (SRK)	2006/04/30		-12.2	-11.5	-10.4	-7.8	-7.0	-6.6
Jay Hallman (Miramar)	2006/06/04		-3.0	-6.2	-8.1	-7.9	-7.3	-6.8
H.Johnson (Miramar)	2006/09/13		1.3	-2.2	-4.3	-6.2	-6.7	-6.7

DORIS NORTH PROJECT THERMISTOR DATA

Drill Hole SRK-54		Bead No.	Bead 1	Bead 2	Bead 3	Bead 4	Bead 5	Bead 6
Read By	**Date**	**Bead Location from Top (m)**	**2.0**	**3.0**	**4.0**	**6.0**	**8.5**	**11.0**
		Bead Depth (m)	**1.0**	**2.0**	**3.0**	**5.0**	**7.5**	**10.0**
Quinn Jordan-Knox (SRK)	2004/09/28	**Temperature (Celsius)**	-0.1	0.0	-0.6	-4.3	-5.7	-6.0
Dylan MacGregor (SRK)	2005/04/21		-14.7	-14.1	-12.9	-9.5	-7.1	-6.8
D Kary (Miramar)	2005/05/16		-12.3	-12.4	-11.9	-9.7	-7.5	-6.9
Jay Hallman (Miramar)	2005/07/18		1.4	-3.6	-6.1	-8.3	-7.9	-7.1
JRH (Miramar)	2005/08/05		1.6	-2.9	-5.3	-7.7	-7.8	-7.2
D Kary (Miramar)	2005/09/08		1.5	-2.1	-4.3	-6.9	-7.6	-7.2
E Ballent (Miramar)	2005/09/26		0.0	-1.8	-3.8	-6.4	-7.5	-7.2
Jay Hallman (Miramar)	2006/06/04		-1.4	-5.3	-7.8	-8.8	-7.5	-6.9
Gary S./Hugh J. (Miramar)	2006/08/06		2.8	-2.0	-4.5	-6.9	-7.4	-7.0
H.Johnson (Miramar)	2006/09/13		0.4	-1.4	-3.7	-6.2	-7.3	-7.1

DORIS NORTH PROJECT THERMISTOR DATA

Drill Hole SRK-55		Bead No.	Bead 1	Bead 2	Bead 3	Bead 4	Bead 5	Bead 6
Read By	**Date**	**Bead Location from Top (m)**	**2.0**	**3.0**	**4.0**	**6.0**	**8.5**	**11.0**
		Bead Depth (m)	**0.8**	**1.8**	**2.8**	**4.8**	**7.3**	**9.8**
Quinn Jordan-Knox (SRK)	2004/09/26	**Temperature (Celsius)**	0.0	0.0	-3.9	-6.2	-7.4	-6.9
Quinn Jordan-Knox (SRK)	2004/09/28		-0.1	-1.1	-5.7	-7.2	-7.7	-7.5
Thermistor Permanently Damaged - Not Repairable								

DORIS NORTH PROJECT THERMISTOR DATA

Drill Hole SRK-56		Bead No.	Bead 1	Bead 2	Bead 3	Bead 4	Bead 5	Bead 6
Read By	**Date**	**Bead Location from Top (m)**	**2.0**	**3.0**	**4.0**	**6.0**	**8.5**	**11.0**
		Bead Depth (m)	**above ground**	**above ground**	**above ground**	**0.8**	**3.3**	**5.8**
Quinn Jordan-Knox (SRK)	2004/09/28	**Temperature (Celsius)**	-2.9	-2.9	-2.6	0.0	-0.3	-3.6
Dylan MacGregor (SRK)	2005/04/21		-12.7	-13.4	-10.7	-11.9		
Dylan MacGregor (SRK)	4/24/2005		1.4	1.7	1.6	-11.7		
D Kary (Miramar)	2005/05/16		-12.8	-8.8	-10.4	-10.8		
Jay Hallman (Miramar)	2005/07/18		20.2	18.4	23.7	0.6		
JRH (Miramar)	2005/08/05		18.9	17.2	22.1	1.5		
D Kary (Miramar)	2005/09/08		26.3	24.0	28.6	1.3		
E Ballent (Miramar)	2005/09/26		-1.4	-1.5	-1.1	0.0		
Jay Hallman (Miramar)	2006/06/04		3.1	3.1	2.3			
Gary S./Hugh J. (Miramar)	2006/08/06		27.0	35.5	31.6			
H.Johnson (Miramar)	2006/09/13		-0.9	-1.5	-0.3			

DORIS NORTH PROJECT THERMISTOR DATA

Drill Hole SRK-57		Bead No.	Bead 1	Bead 2	Bead 3	Bead 4	Bead 5	Bead 6
Read By	**Date**	**Bead Location from Top (m)**	**above ground**	**3.00**	**4.00**	**6.00**	**8.50**	**11.00**
		Bead Depth (m)	**n/a**	**0.33**	**1.33**	**3.33**	**5.83**	**8.33**
Dylan MacGregor (SRK)	2005/04/26	**Temperature (Celsius)**	-6.1	-5.5	-12.9	-12.5	-8.9	-6.6
D Kary (Miramar)	2005/05/16					-12.2	-10.6	-8.8
Jay Hallman (Miramar)	2005/07/18		13.5			-7.6	-9.1	-9.0
JRH (Miramar)	2005/08/05		12.7			-6.6	-8.4	-8.7
D Kary (Miramar)	2005/09/08		7.8			-5.3	-7.3	-8.2
E Ballent (Miramar)	2005/09/26		-1.4			-4.8	-6.8	-7.8
Jay Hallman (Miramar)	2006/06/04		13.5			-9.2	-9.7	-9.6
Gary S./Hugh J. (Miramar)	2006/08/06		33.6	0.9	-0.9	-1.2		
H.Johnson (Miramar)	2006/09/13		0.0			-4.4	-6.4	-7.4

DORIS NORTH PROJECT THERMISTOR DATA

Drill Hole SRK-58		Bead No.	Bead 1	Bead 2	Bead 3	Bead 4	Bead 5	Bead 6
Read By	**Date**	**Bead Location from Top (m)**	**2.00**	**3.00**	**4.00**	**6.00**	**8.50**	**11.00**
		Bead Depth (m)	**1.07**	**2.07**	**3.07**	**5.07**	**7.57**	**10.07**
Dylan MacGregor (SRK)	2005/04/25	**Temperature (Celsius)**	-6.5	-6.2	-3.5	-1.5	-0.4	-5.8
Dylan MacGregor (SRK)	2005/04/26		-11.3	-10.8	-8.9	-5.1	-2	-6.5
D Kary (Miramar)	2005/05/16		-11.3	-11.2	-10.6	-8.2	-7.3	-6.9
Jay Hallman (Miramar)	2005/07/18		-4.2	-7.6	-9.1	-8.7	-7.7	-7.1
D Kary (Miramar)	2005/09/08		-2.8	-6	-7.9	-8.4	-7.8	-7.2
E Ballent (Miramar)	2005/09/26		-2.6	-5.6	-7.5	-8.2	-7.8	-7.3
Jay Hallman (Miramar)	2006/06/04		-5.5	-8.9	-9.8	-8.4	-7.4	-7.1
Gary S./Hugh J. (Miramar)	2006/08/06		-2.7	-6.1	-7.9	-8.1	-7.5	-7.1
H.Johnson (Miramar)	2006/09/13		-2.1	-5.4	-7.3	-7.9	-7.6	-7.3

DORIS NORTH PROJECT THERMISTOR DATA

Drill Hole SRK-62		**Bead No.**	**Bead 1**	**Bead 2**	**Bead 3**	**Bead 4**	**Bead 5**	**Bead 6**	**Bead 7**	**Bead 8**	**Bead 9**	**Bead 10**	**Bead 11**	**Bead 12**
Read By	**Date**	**Bead Location from Top (m)**	**above ground**	**1.25**	**1.50**	**1.75**	**2.00**	**2.50**	**3.00**	**3.50**	**4.00**	**4.50**	**5.00**	**6.00**
		Bead Depth (m)	**n/a**	**0.25**	**0.50**	**0.75**	**1.00**	**1.50**	**2.00**	**2.50**	**3.00**	**3.50**	**4.00**	**5.00**
Dylan MacGregor (SRK)	2005/04/25	**Temperature (Celsius)**	7.4	-4.4	-7.9	-10.1	-11.3	-12.1	-12.0	-11.4	-10.7	-9.8	-9.1	-7.6
Dylan MacGregor (SRK)	2005/04/26		-7.8	-6.8	-7.7	-9.5	-10.8	-12.0	-12.3	-11.7	-11.1	-10.2	-9.5	-8.1
D Kary (Miramar)	2005/05/16		-6.6	-10.6	-10.7	-10.9	-10.8	-10.8	-10.8	-10.7	-10.5	-10.2	-9.9	-8.9
Jay Hallman (Miramar)	2005/07/18		13.3	2.5	2.0	1.7	-1.2	-3.2	-4.5	-5.5	-6.4	-7.1	-7.6	-8.0
JRH (Miramar)	2005/08/05		14.6	3.1	2.3	2.1	-0.9	-2.5	-3.8	-4.7	-5.6	-6.4	-7.0	-7.5
D Kary (Miramar)	2005/09/08		17.6	3.4	2.4	2.0	-0.4	-1.9	-3.0	-3.9	-4.8	-5.5	-6.1	-6.9
E Ballent (Miramar)	2005/09/26		-1.7	0.0	-0.1	-0.2	-0.7	-1.6	-2.7	-3.5	-4.3	-5.0	-5.7	-6.5
Jay Hallman (Miramar)	2006/06/04		12.2	1.7	0.3	-2.6	-3.6	-5.2	-6.7	-7.7	-8.4	-8.8	-9	-8.6
Gary S./Hugh J. (Miramar)	2006/08/06		21.8	3.8	2.5	2.2	-0.7	-2.2	-3.4	-4.3	-5.1	-5.8	-6.4	-7.1
H.Johnson (Miramar)	2006/09/13		-1.0	0.8	1.1	0.7	-0.4	-1.5	-2.6	-3.4	-4.2	-4.9	-5.6	-6.4

Appendix D
AMCL Geophysical Study Report

IMC

Associated Mining Consultants Ltd.

Geophysical Study for Anomalous Stratigraphy and/or Ice Content Tail Lake, Hope Bay - Nunavut

Prepared for
SRK Consulting Inc.
Vancouver, British Columbia

Submitted by
Associated Mining Consultants Ltd.
Vancouver, British Columbia

06PW43 **June 2006**

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June 29, 2006

SRK Consulting Inc. (Canada)
Suite 800, 1066 West Hastings Street
Vancouver, British Columbia
V6E 3X2

Attention: Maritz Rykaart

Dear Maritz,

Associated Mining Consultants Ltd. (AMCL) is pleased to submit the following report entitled:

Geophysical Study for Anomalous Stratigraphy
and/or Ice Content
Trail Lake, Hope Bay - Nunavut

We would like to express our thanks to SRK Consulting Inc. for the opportunity to provide our services in relation to this project.

If you have any questions, or require any additional information, please do not hesitate to contact our office.

Yours sincerely,

ASSOCIATED MINING CONSULTANTS LTD.

Claude Robillard

Dave Butler, Ph.D., P.Geo.
Manager, Vancouver Office

Claude Robillard, géoph.
Senior Geophysicist

/cew

TABLE OF CONTENTS

1.0 INTRODUCTION 1
1.1 Site Description 1

2.0 METHODS 3
2.1 Seismic Refraction 3
2.2 Ground-Penetrating Radar (GPR) 5
2.3 Transient Electromagnetic (TDEM) 6
2.4 Capacitively Coupled Resistivity Profiling (OhmMapper) 8

3.0 RESULTS 10

4.0 CONCLUSIONS 12

1.0 INTRODUCTION

A geophysical survey was undertaken by Associated Mining Consultants Ltd. (AMCL) from April 4, 13, 2006 at the Hope Bay site in Nunavut to provide information in support of mine facilities/infrastructures design. The primary objectives of the survey were to guide a proposed drilling programme by identifying regions of anomalous stratigraphy and/or ice content, and to map bedrock topography around the perimeter of Tail Lake.

In accordance with AMCL's proposal (AMP 1786) to SRK Consulting Inc., a two-phase approach has been taken to fulfill the objectives. A test survey to determine the most appropriate geophysical methods was conducted first which included electrical imaging, time domain electromagnetics, seismic refraction and ground-penetrating radar. The test survey was conducted at the proposed sites of both the south and north dams, where geophysical data could be readily correlated with drill-hole data.

After phase one was completed, and data analysed in the field, it was decided to proceed only with ground-penetrating radar for phase two of the study, which covered the entire perimeter of Tail Lake.

This report presents the results of both phases of the survey.

1.1 Site Description

Hope Bay is located some 750 km northeast of Yellowknife and 130 km southwest of Cambridge Bay. Figure 1 shows the general location of the area. The survey site at Tail Lake is located approximately 20 km east of the camp and was accessible by skidoo. The lake itself is 3.3 km long and 400 m wide on average.

Figure 1 shows the location and outline of the survey area.

At the time of the survey, the snow cover varied between 0.5 to 1.5 m in the survey area. The two proposed dam areas were easily accessible. The 33.5 m contour line that had to be surveyed around the perimeter of the lake could not be surveyed over some short sections where cliffs and snow drifts were too steep to allow access.

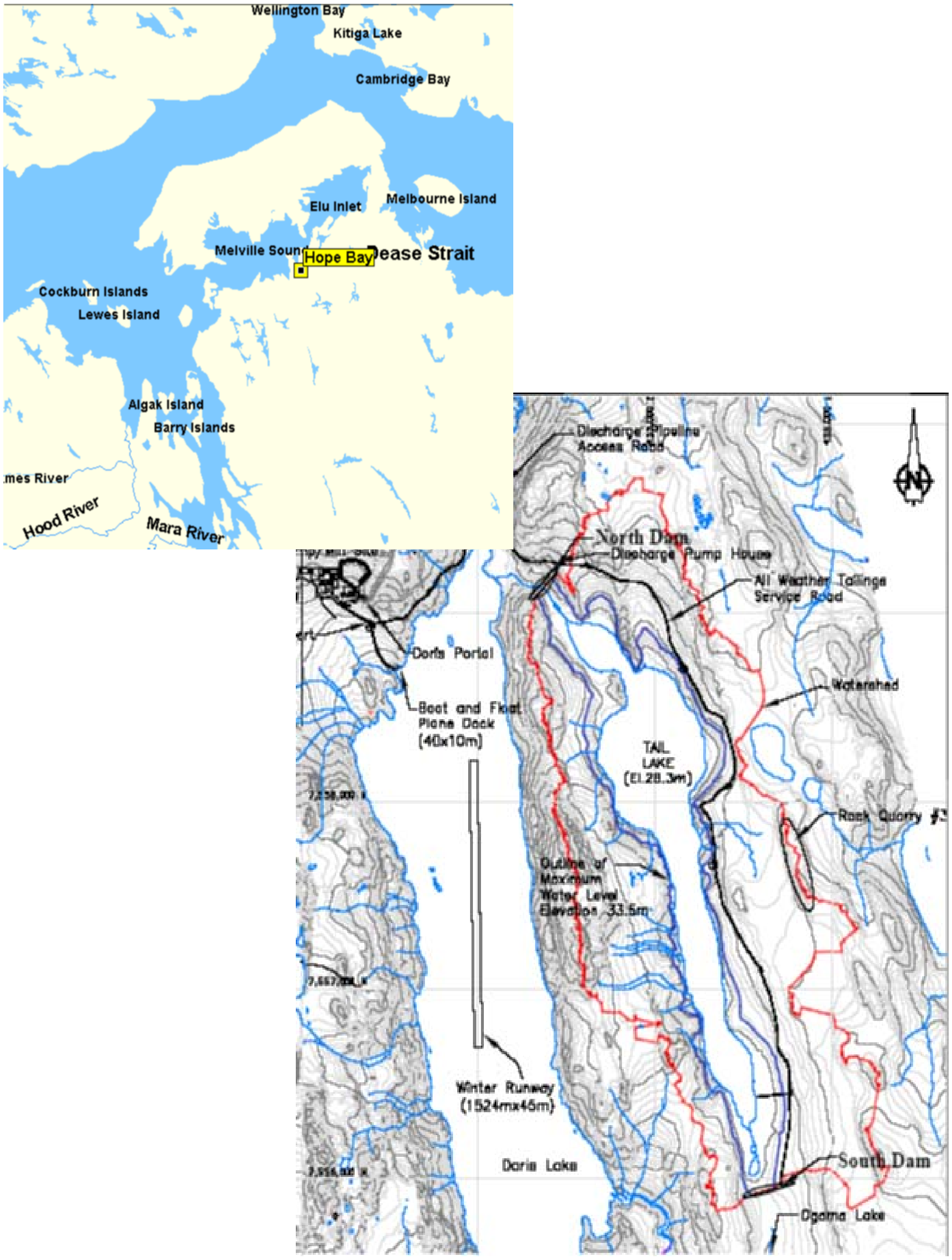


Figure 1: General Location Map

2.0 METHODS

2.1 Seismic Refraction

The seismic refraction method, commonly used to map depth to bedrock, rests on the principle that a sound wave travelling at a particular speed in one material will, on encountering a boundary with a different material, change its direction of travel *if its speed of travel changes*. The degree of the bend in direction (the refraction) is directly related to the contrast in velocity across the boundary. If a wave crosses a boundary at an angle, it will refract towards the boundary if its velocity increases across the boundary, whereas it will refract away from the boundary if its velocity decreases across the boundary.

Generally, earth materials become harder with increasing depth, and harder materials lead to faster wave speeds. Therefore downwards-travelling sound waves generated at the surface will refract upwards as they encounter harder materials at increasing depths. At some point, as shown in Figure 2, the velocity contrast across a boundary will be such that the waves will refract horizontally.

The refracted wave will travel along the layer, leaking energy back towards the surface as it travels. The waves will refract upwards into the slower-velocity material above, and they will continue to refract upwards as they encounter progressively slower-velocity material on the return towards the surface. Since these waves have travelled at velocities much higher than those waves that travelled only along the earth's surface, they will eventually overtake the surface waves, so that at some point on the surface, the first arrival of sound will come from a wave that travelled through the deeper layer. Bedrock is typically the deepest layer encountered in a refraction survey.

The technique therefore measures the travel-times of acoustic waves between a source location and a number of geophones placed at equal increments away from the source. The waves are generated by an external energy source, such as a sledgehammer or a small explosive charge. The sound waves are recorded by a series of geophones (a 'spread') connected to a seismograph. Typically, sound waves are generated at five to seven shot locations per spread, with two of those shots set off at each end of the spread in order to record bedrock information right to the edges of each spread. In this survey, however, shot locations were more tightly spaced, being located between every second geophone. The recorded travel-times are a function of the speeds of sound within the subsurface layers, and the thicknesses and depths of the layers. The depths of investigation in seismic refraction, largely a function of energy source and receiver geometry, are generally in the order of one-fifth of the geophone array dimension.

For this study, the geophones were placed at 8 m intervals, and the seismic energy source was created by striking a steel plate with a sledgehammer.

The success of the method is dependent upon the degree of contrast in velocity between the target layers. Typical acoustic velocities of common geologic materials are listed in Table 1.

The method also requires that velocity increases with depth. Velocity reversals may, at times, result in layers being "hidden" and thus undetectable by the seismic refraction method.

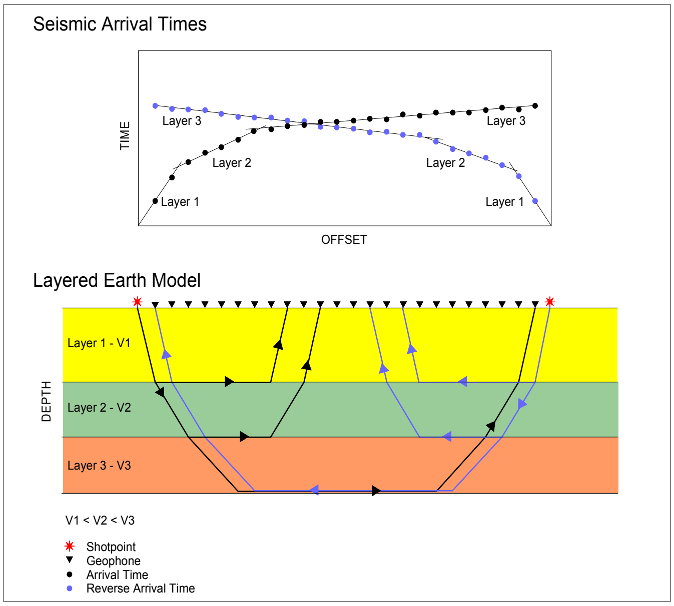


Figure 2: Illustration of seismic refraction method

Table 1: Acoustic Velocities of Common Geological Materials

Material	**v (m/s)**
Weathered Surface Material	305 - 610
Gravel, Rubble or Dry Sand	468 - 915
Wet Sand	610 - 1830
Clay	915 - 2750
Sandstone	1830 - 3970
Shale	2750 - 4270
Chalk	1830 - 3970
Limestone	2140 - 6100
Granite	4580 - 5800
Metamorphic Rock	3050 - 7020
Water (dependent on temperature and salt content)	4700 - 5500
Ice	3600 - 3700

2.2 Ground-Penetrating Radar (GPR)

The concept of radio detection and ranging (radar) is fundamentally the precise measurement of outbound and return travel times of high-frequency (10 MHz to 1000 MHz) electromagnetic waves from boundaries of contrasting electrical impedance. Two-dimensional profiles of the subsurface produced by the GPR method are referenced to signal travel times. The conversion of a timescale to one relevant to depth requires the accurate determination of the velocity of the medium traversed, often by the correlation with drill hole logs and/or the results of complementary geophysical methods.

The velocity and attenuation of radar signals within the subsurface depends on the dielectric and conductivity properties of subsurface materials. Variations in the electrical properties of soils and rocks are usually associated with changes in grain size and/or water content which, in turn, cause part of a transmitted signal to be reflected. The reflected signal is detected by the receiver where it is amplified, digitized and stored for subsequent data processing and interpretation. A schematic showing the operation of the GPR is shown in Figure 3.

Ground penetrating radar penetration depth is limited primarily by the conductivity of the subsurface. High proportions of clay and/or total dissolved solids within the groundwater can severely reduce the effective exploration depth. Although depth penetration can be increased by reducing antenna frequency, vertical resolution is compromised proportionally.

A Malå RAMAC ground penetrating radar system was used to acquire subsurface geophysical data along the proposed pipeline watercourse crossings. The system was used in a reflection, or single-fold fixed-offset, profiling mode.

Data were acquired using both 50 MHz and 100 MHz unshielded antennas during phase one and using only the 50 MHz antenna for the perimeter of the lake during phase two.

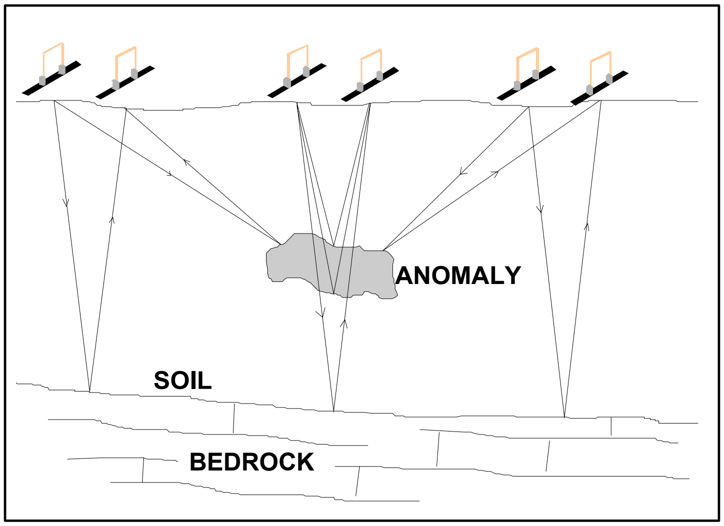


Figure 3: Illustration of the ground penetrating radar (GPR) method

2.3 Transient Electromagnetic (TDEM)

In the TDEM method, an electromagnetic field is generated by passing a square-wave electrical current through a grounded loop of wire, which in this case was laid on the ground surface. In the upper half-space (air), a magnetic field establishes itself almost instantly over the surface of the earth, while in the lower half-space (earth), the magnetic field develops much more slowly, with high-frequency components attenuated in the conductive earth. At the surface of the earth, energy refracts downward into the ground by induction. This energy induces secondary eddy currents in the earth that in turn generate their own magnetic field which travels back to the surface. This returned magnetic field is measured at a site where we wish to know the subsurface resistivity variations. As the excitation field penetrates deeper into the earth, it takes longer for the secondary field to return to the surface. At the receiver site, the returned vertical magnetic field is recorded as a function of time.

In order to measure the vertical magnetic field variations as a function of time, an induction loop receiver is placed on the surface of the ground at approximately 10 m from the side of the individual transmitter dipoles. Depending on the size of the target, sounding typical spacings of 20 to 100 m are used. At every sounding site, the vertical magnetic field changes are recorded as time-varying voltages. These curves, which start at zero time (surface) and continue from several hundred ms to over 15 sec, depending on the conductivity of the sedimentary section, are called transients. The amplitude and shape of the transients are used to determine the resistivity distribution of the sedimentary section, generally to basement, directly below the receiver loop. Bulk changes in earth resistivity with depth cause voltage changes in the recorded curves. At every sounding site, individual transients are stacked in order to improve the signal to noise ratio. Then, resistivity formulas derived from Maxwell's equations are used to calculate apparent resistivity curves as a function of time (depth) at each sounding site. Different apparent resistivity curves are calculated using different time windows. Those time windows are often referred as early-time and late-time.

After apparent resistivities have been calculated for each sounding, these curves are mathematically inverted in terms of a one-dimensional, multi-layered earth with basement the last layer. We use a ridge regression inversion program called TEMIX from Interpex Limited, Golden, Colorado. As many as six layers can be inferred. A schematic of the whole process from data acquisition to inversion is shown in Figure 3.

Using the thicknesses and resistivities derived from the inversions, elevation maps of structurally significant boundaries and structural cross-sections can be prepared. Because resistivity is a function of lithology, the lithologic architecture of the subsurface can be determined by mapping resistivity boundaries.

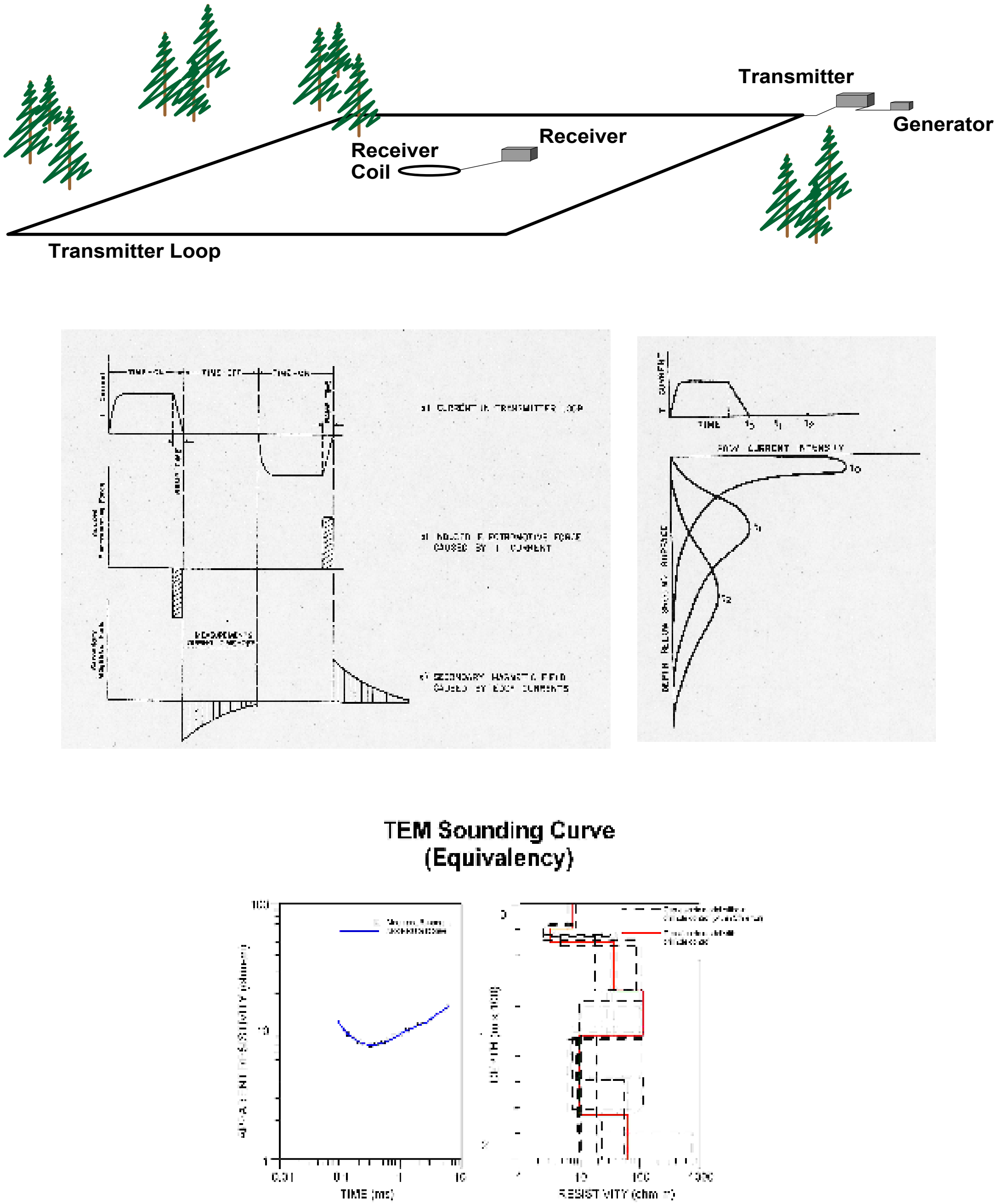


Figure 4: Principle of the TEM method from the data acquisition process (top), the geophysical properties being measured (middle), and the data inversion process (bottom)

2.4 Capacitively Coupled Resistivity Profiling (OhmMapper)

The purpose of electrical surveys is to determine the subsurface resistivity distribution by making measurements on the ground surface. The ground resistivity is related to various geological and physical parameters such as the mineral and fluid content, porosity and degree of water saturation in the rock. Presence of permafrost also has a significant influence on electrical resistivity. As the temperature of the subsurface decreases below 0°, and as the ice content increases, the resistivity of subsurface materials increases substantially as illustrated in Figures 5a and 5b.

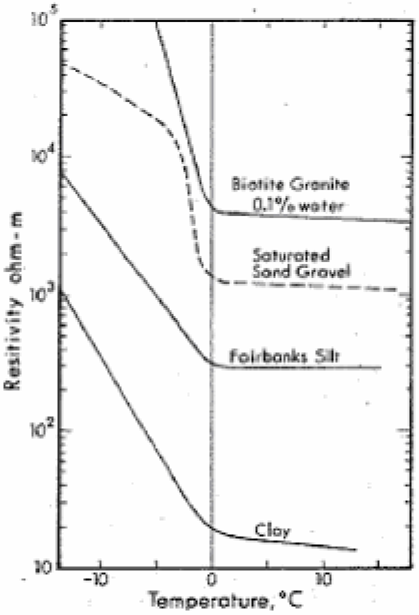


Figure 5a

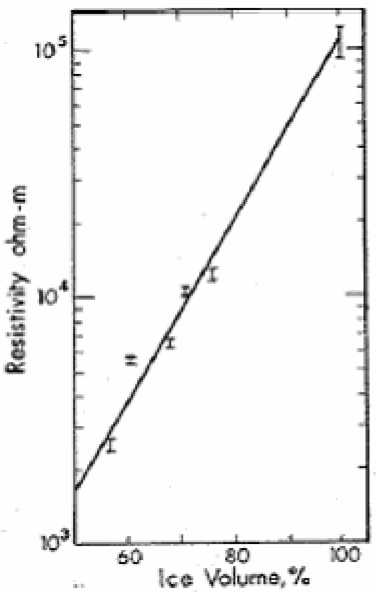


Figure 5b

The resistivity measurements are normally acquired by injecting current into the ground through two current electrodes, and measuring the resulting voltage difference at two potential electrodes. In this survey, we used a capacitively-coupled resistivity meter designed to measure sub-surface resistivity in areas where the use of traditional galvanically coupled (DC) resistivity system is impractical and slow. The OhmMapper consists of an ungrounded dipole transmitter, receiver and data logger. A transmitter electrifies two coaxial cables (transmitter dipole) with an AC current. Current is thus coupled to the earth through the capacitance of the cable. A matched receiver automatically tuned to the transmitter frequency, measures the associated voltage picked up on the receiver's dipole cables. The receiver then transmits a voltage measurement, normalized to current, to the logging console. Apparent resistivity is calculated using the appropriate geometric factor for the capacitively coupled antenna array. The OhmMapper is designed to be pulled along the ground as a streamer. The principle of its operation is shown on Figure 6.

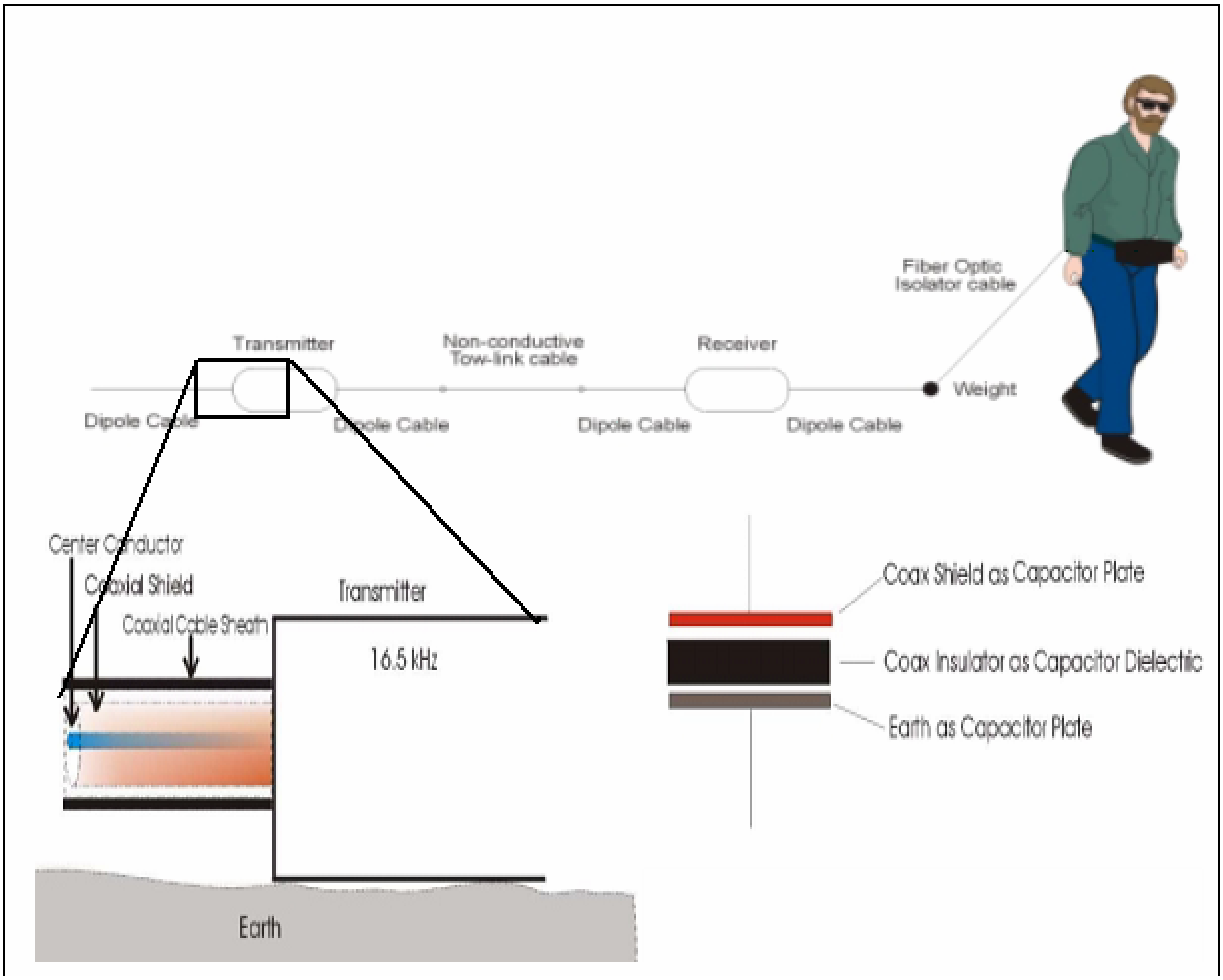


Figure 6: Principle of the OhmMapper system

3.0 RESULTS

During phase one, the OhmMapper, TEM and GPR were tested at the South Dam location and seismic refraction was tested at the North Dam. TEM was also tested at the North Dam for one day when it was too windy to test the seismic refraction method.

The OhmMapper results showed that the method did not allow good penetration into the conductive clays near the surface. The signal is limited to the first 2 or 3 m and then becomes very noisy and could not be inverted past that depth with any confidence as the correlation with available drill hole data becomes inconsistant. The test section is shown on Figure 7.

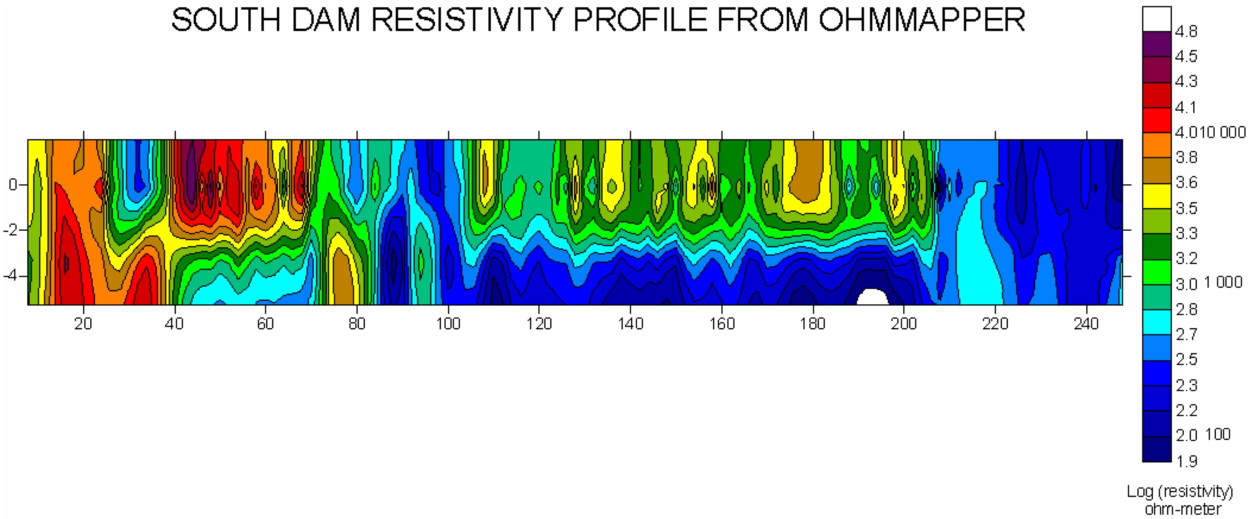


Figure 7: Resistivity section at the South Dam showing poor penetration of the signal

The TEM data were also too noisy and the data could not be inverted to provide a reasonable solution. In the field, the survey parameters were modified to minimize the noise:

- The loop size was increased from 15 x 15 m to 30 x 30 m, and then to 45 x 45 m
- Data was recorded with different sets of gain
- The integration time for taking readings (stacking) was doubled (from 15 to 30 s).

However, the inverted model could not converge to an acceptable solution and it was decided not to proceed with the TEM in phase two.

The seismic refraction data provided some interesting results and for some sections of the profile, it appears to map the boundary between the frozen overburden and the bedrock. However, drill hole data did not support the interpretation. Figure 8 shows the calculated section from the seismic data at the North Dam overlayed on the drill hole data. The correlation on the west side shows that the seismic refractor matches better with the the boundary of the frozen sand. On the east side, the seismic failed to show the subcropping bedrock. The main reason for this miscorrelation is the small contrast in velocity between frozen ground (3000 m/s) and the underlying basalt (3000 to 5000 m/s). Because of that, it is very difficult to see the change in slope (velocity) on the time-distance curves. Another limitation is the poor signal to noise ratio due to using a seismic hammer as a source (use of explosives was not an option for this survey).

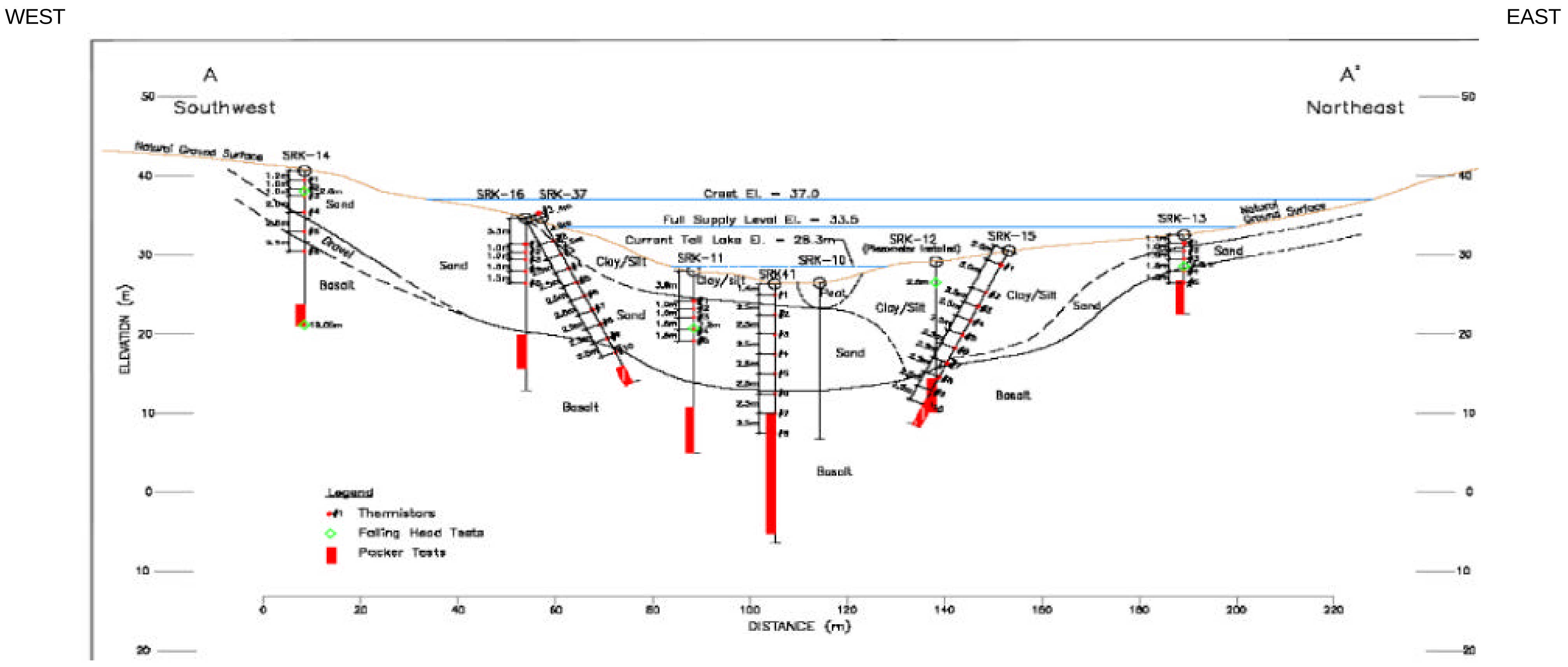
Due to logistics, while waiting for demobilization, more seismic data were acquired at the South Dam. The results are shown in Figure 9. Again, the seismic refractor fails to correlate with the bedrock. On the western portion of the profile, the correlation is even worse as only one velocity could be mapped at 3000 m/s. Similar absence of velocity contrasts on the records occur on three more profiles acquired on the east bank of the lake, and the seismic refraction failed to map bedrock.

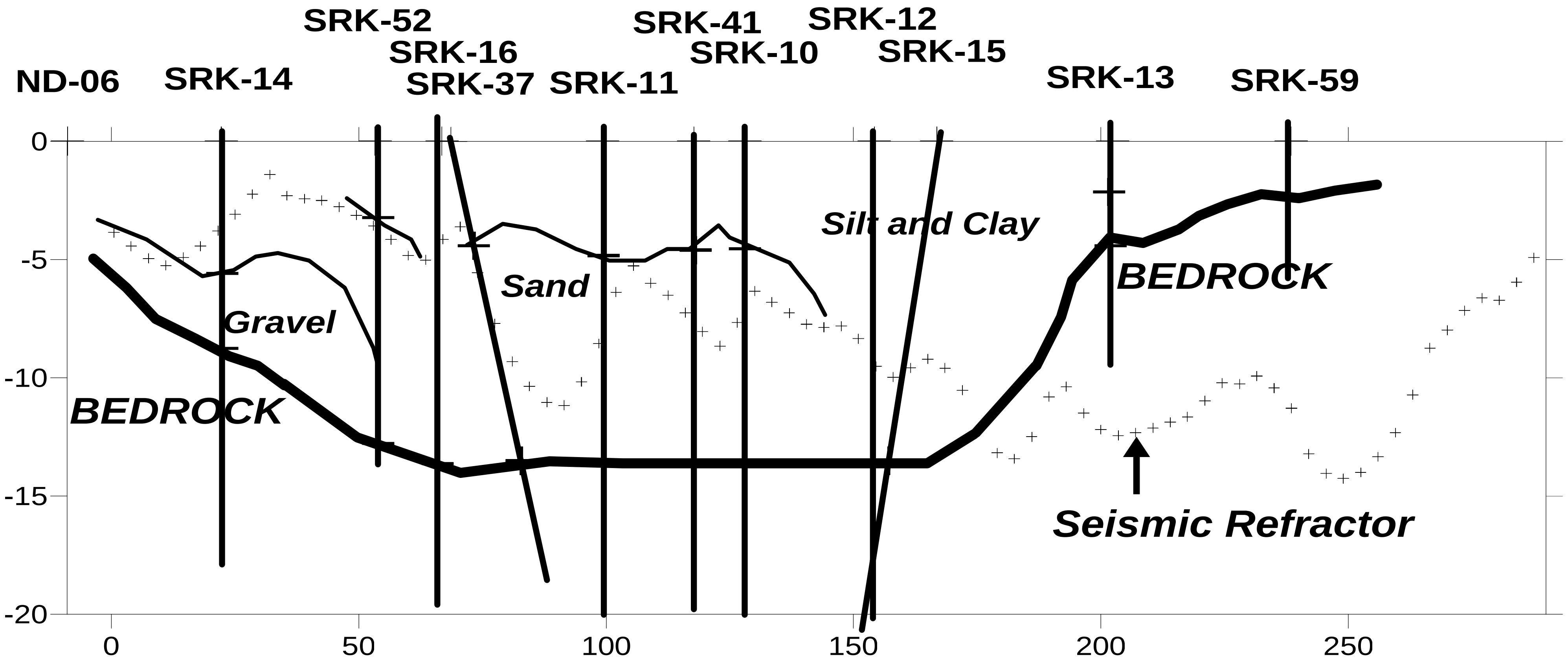
The GPR survey did yield interesting results although its depth of penetration was limited to 5 to 10 m. It succeeded in mapping the silt and clay to sand and gravel interfaces but could not profile the bedrock where it was deeper than 10 m. Figures 10 and 11 show the GPR sections across the South and North dams respectively.

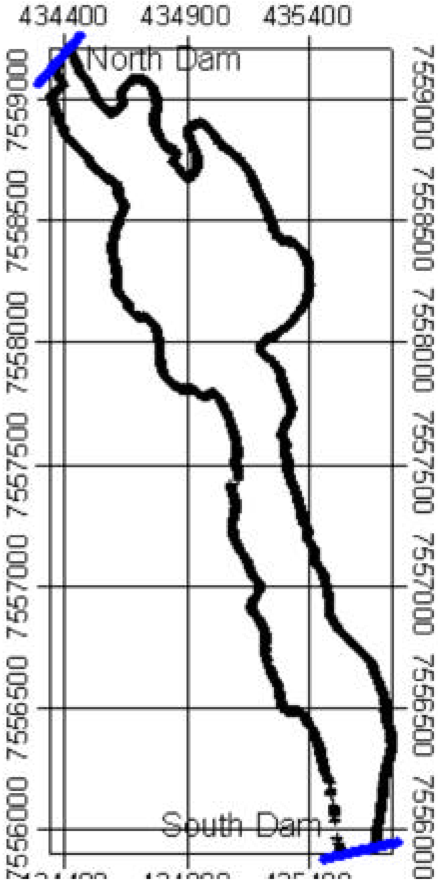
It was decided to use only GPR for the survey around the perimeter of the lake, and possibly use seismic where GPR did not have the penetration to map the bedrock or where the clay near the surface blocked the signal. The results are shown in Appendix 1, with the interpretation ovelayed on the GPR section. Correlation with the available drill hole showed that in general, it differentiated between the silt, clay and sand and could also map bedrock where it was within 5 to 10 m from the surface. On one of the profiles (P6), it showed a dipping reflector that could be associated with faulting or plunging bedrock underneath what could be a very coarse glacial deposit.

4.0 CONCLUSIONS

Of the four geophysical methods used at Tail Lake, only GPR provided valuable information for the objective of identifying regions of anomalous stratigraphy and mapping bedrock. However, its depth of penetration was limited to less than 10 m.







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Engineers and Scientists

NORTH DAM
SEISMIC CORRELATION

Figure 8
Project 06PW43
June 2006

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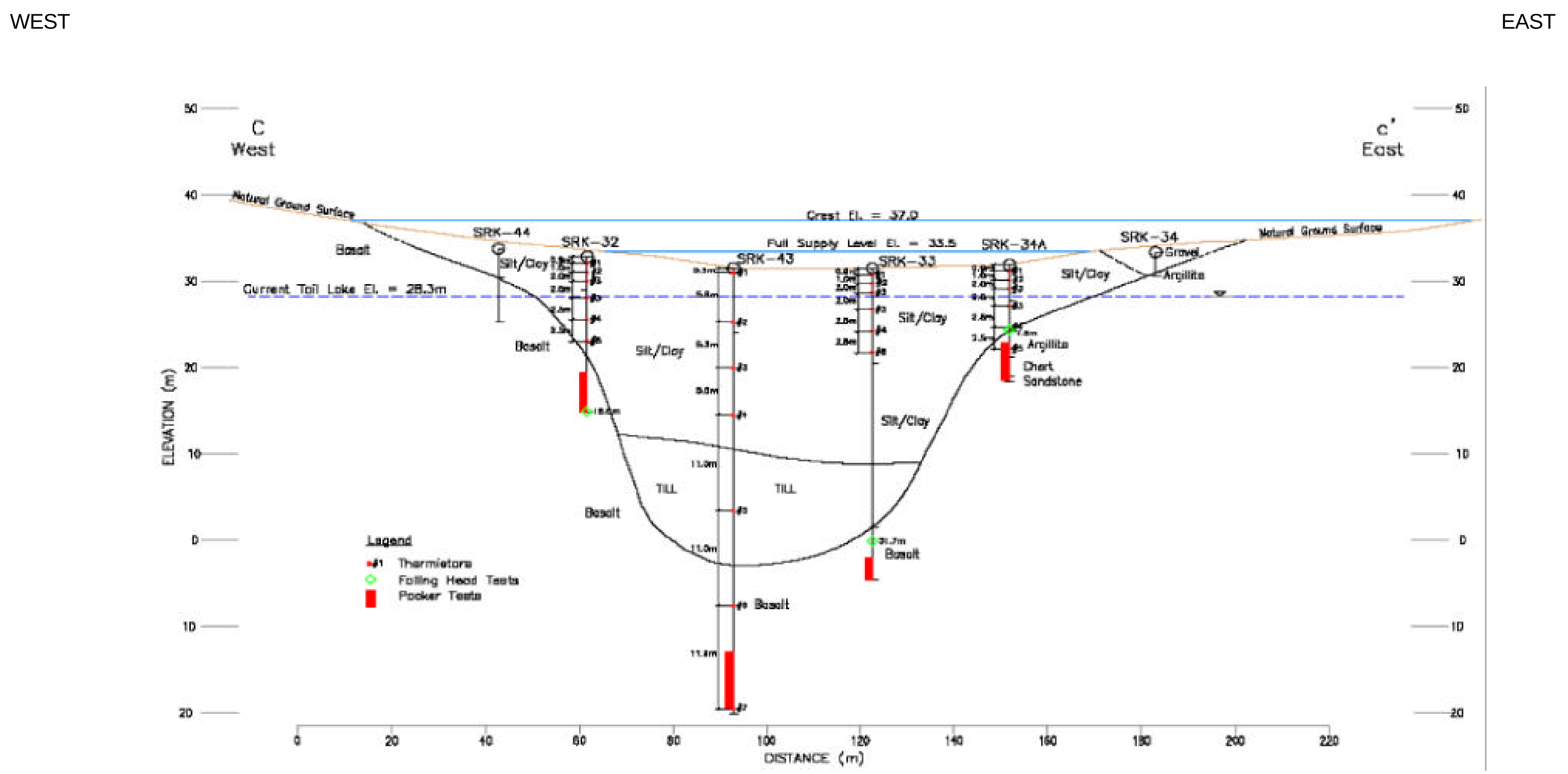

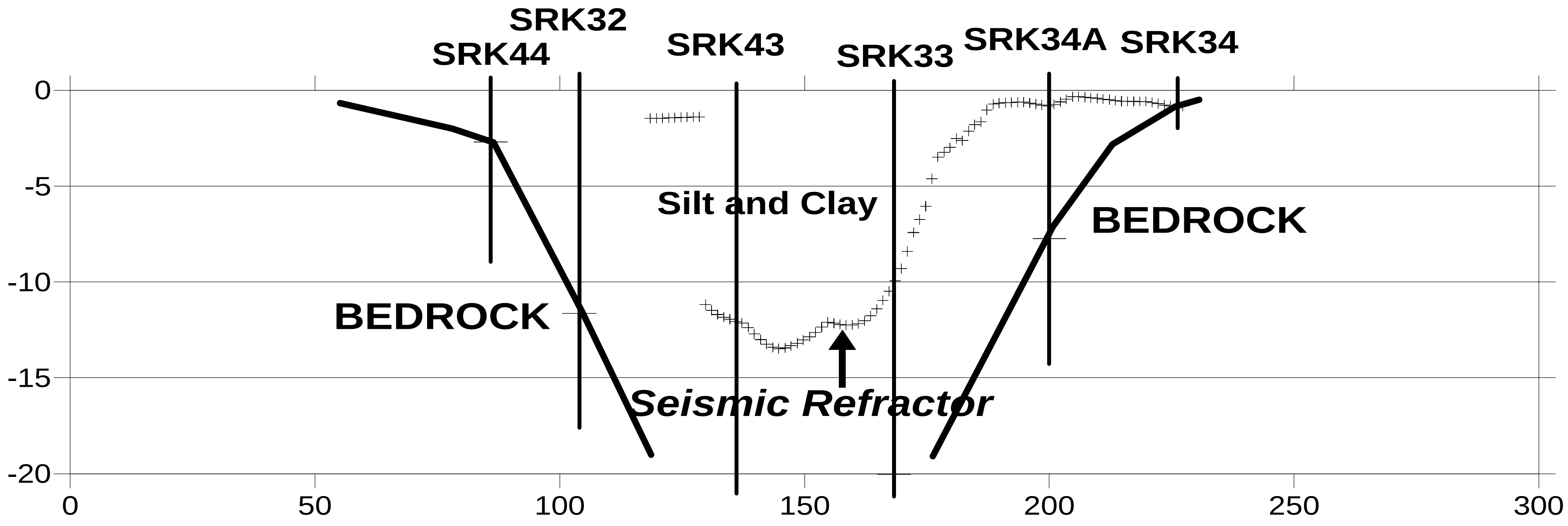

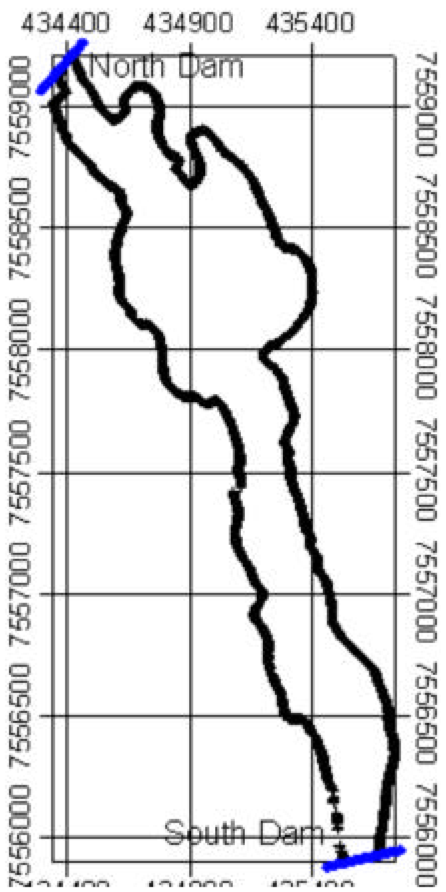

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SOUTH DAM
SEISMIC CORRELATION

Figure 9
Project 06PW43
June 2006

IMC
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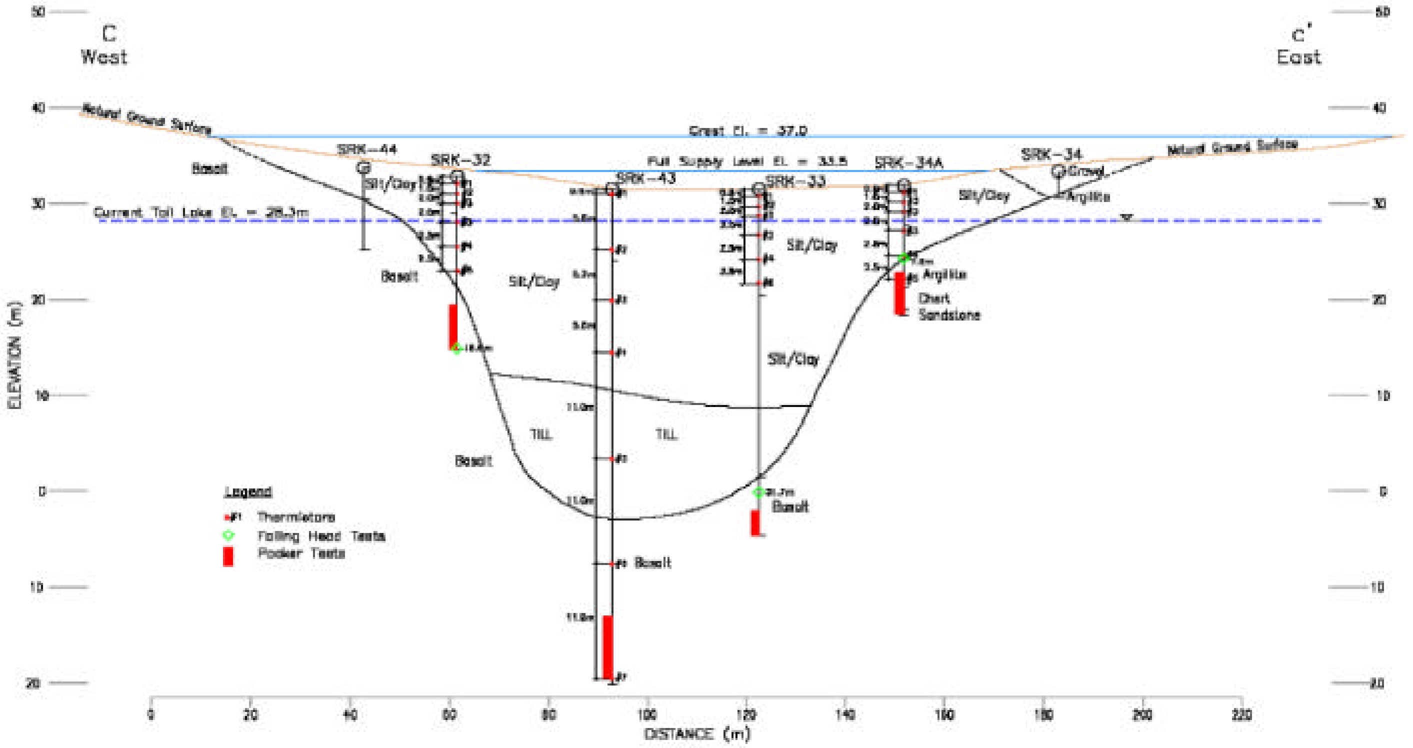
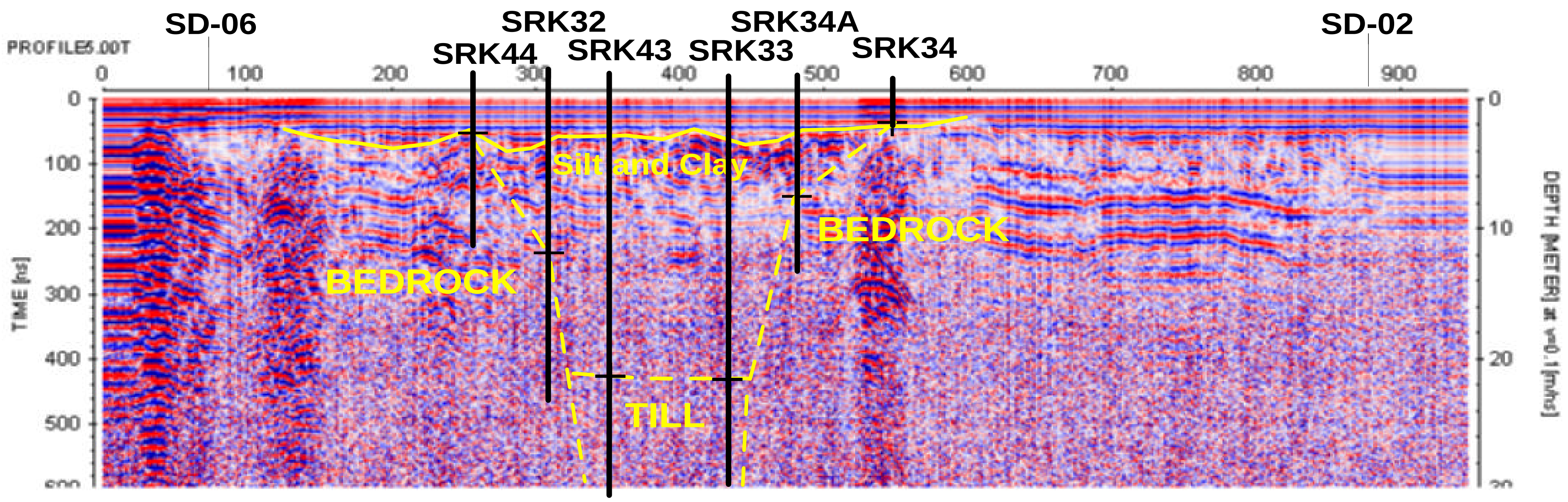
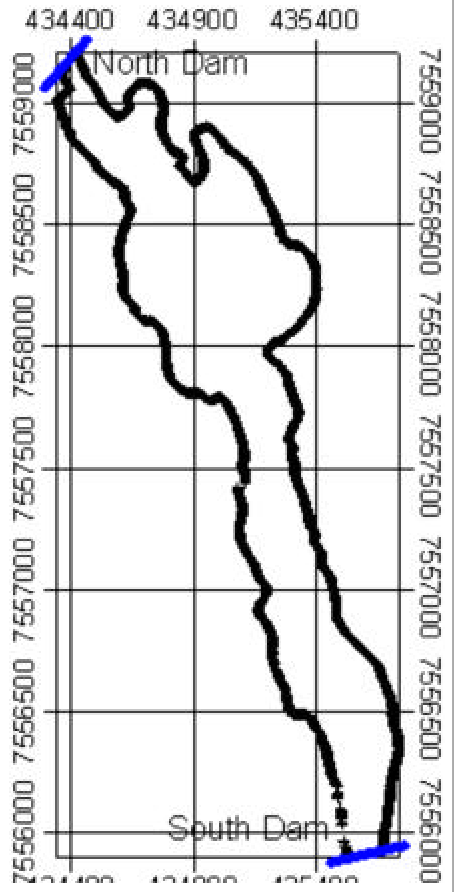
WEST

EAST







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SOUTH DAM
GPR CORRELATION

Figure 10
Project 06PW43
June 2006

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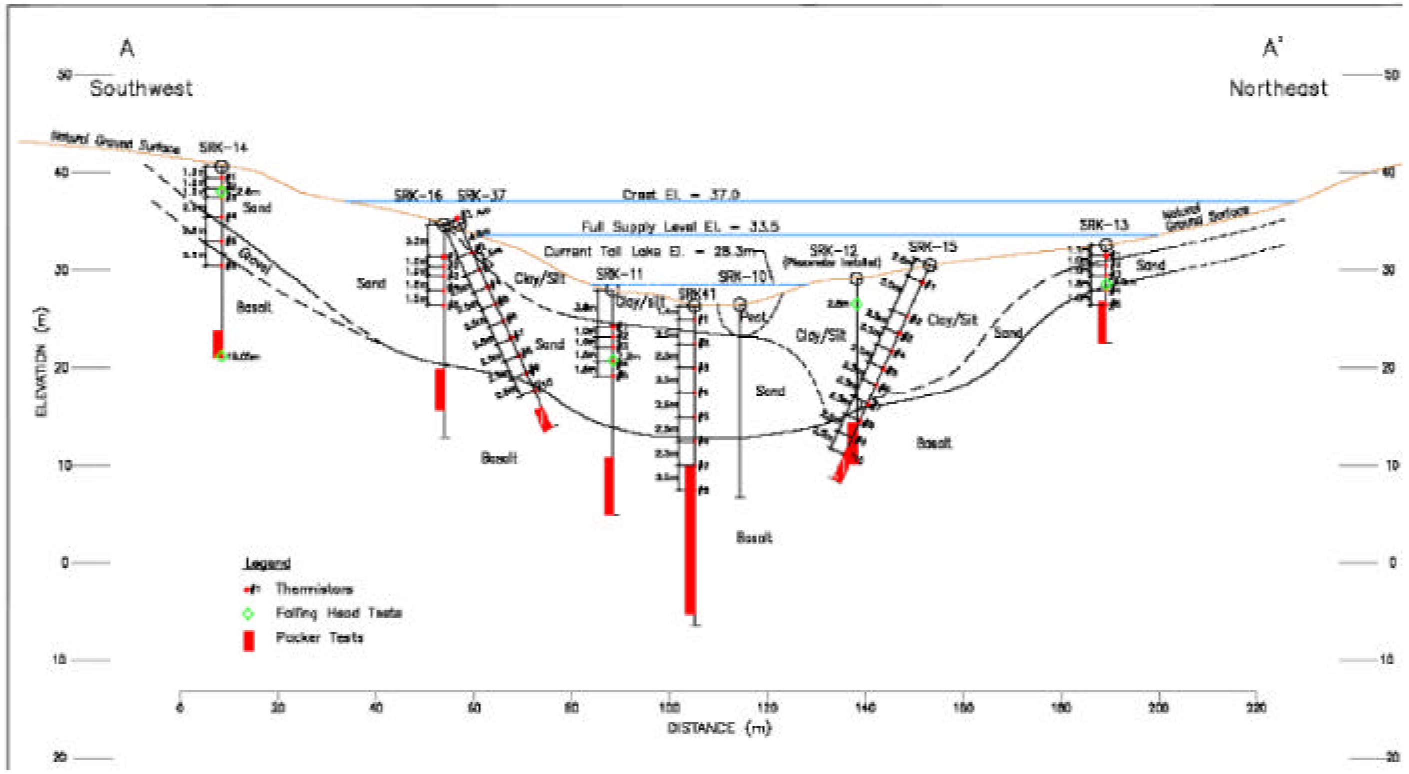
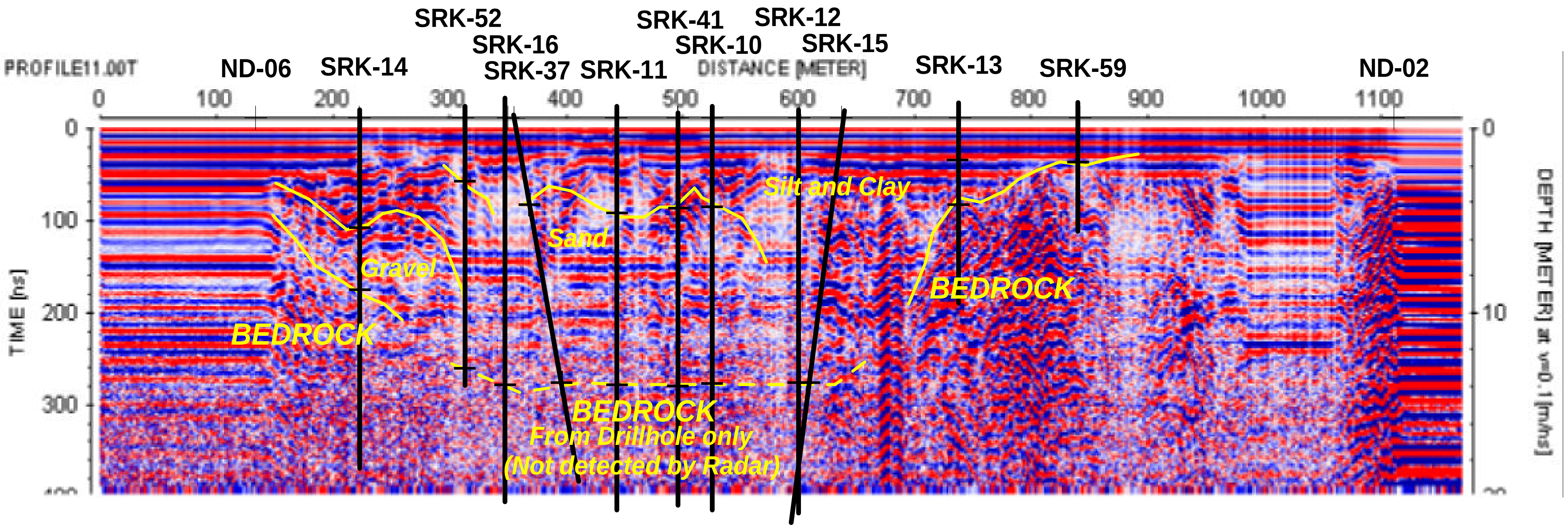
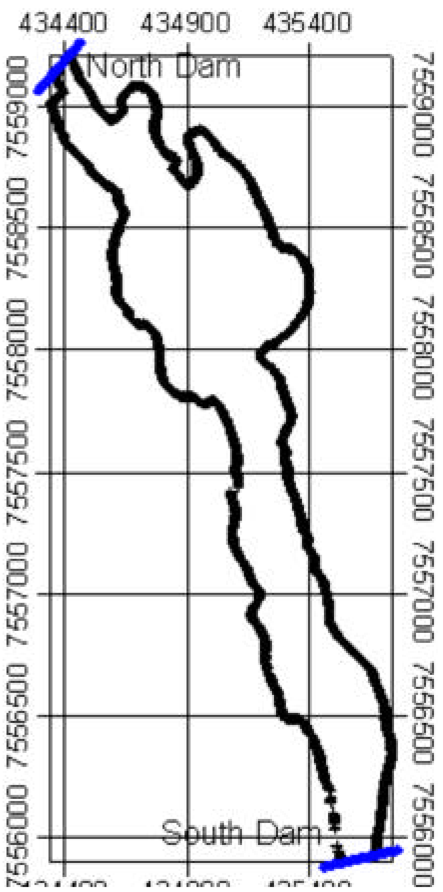
WEST

EAST







SRK Consulting
Engineers and Scientists

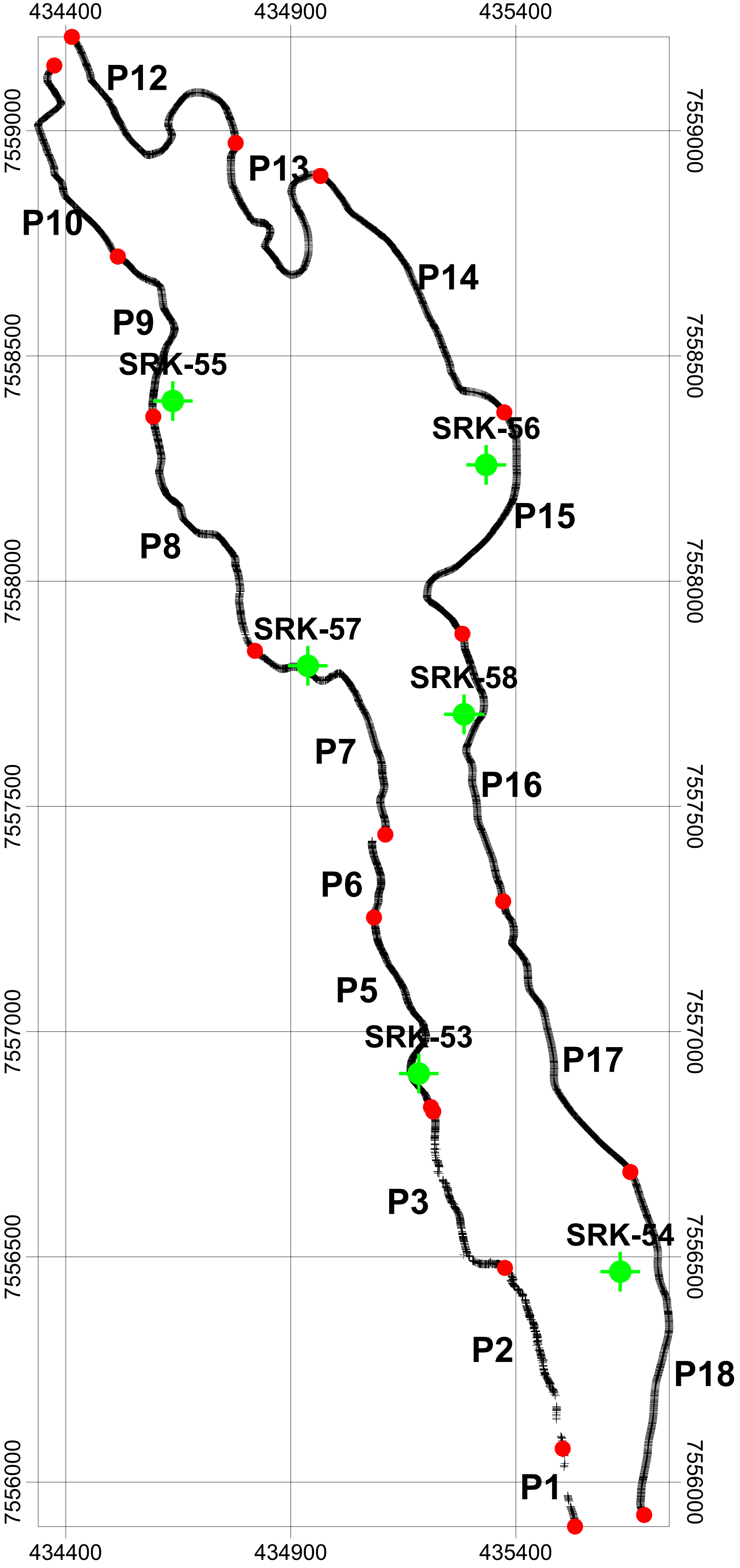
NORTH DAM
GPR CORRELATION

Figure 11
Project 06PW43
June 2006

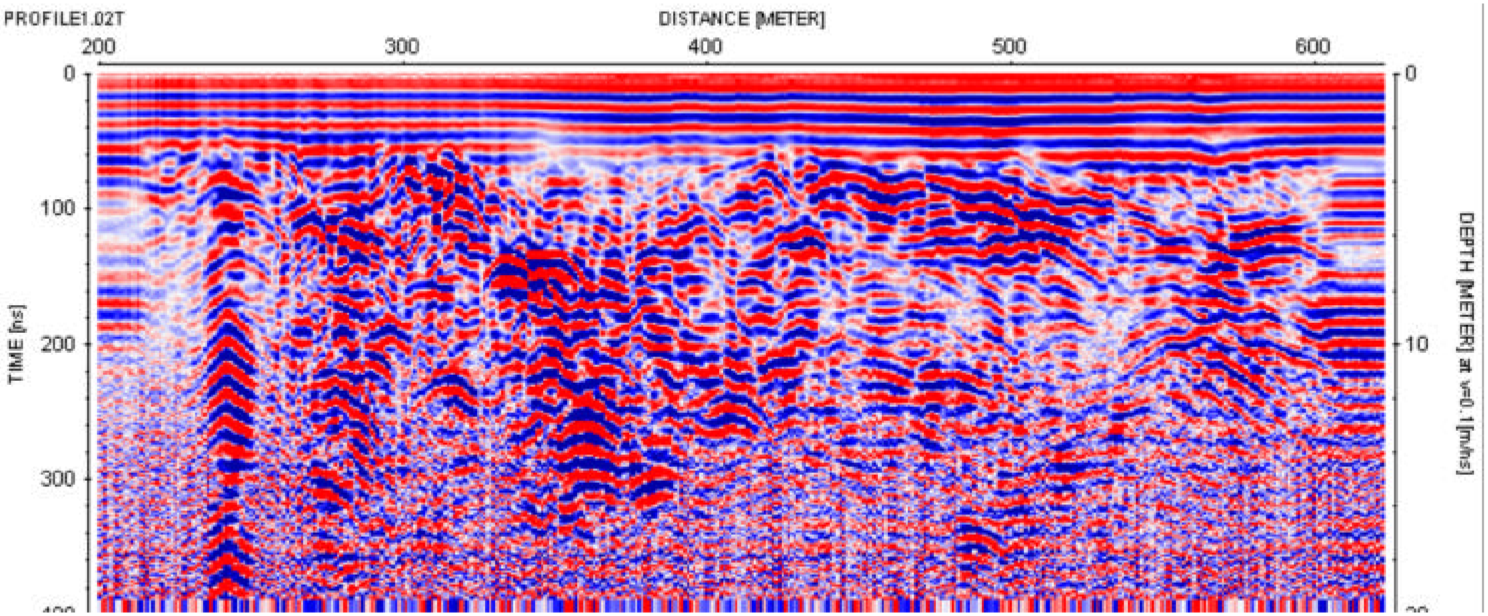
IMC
Associated Mining Consultants Ltd.

APPENDIX 1

RESULTS



LOCATION OF GPR PROFILE AROUND THE LAKE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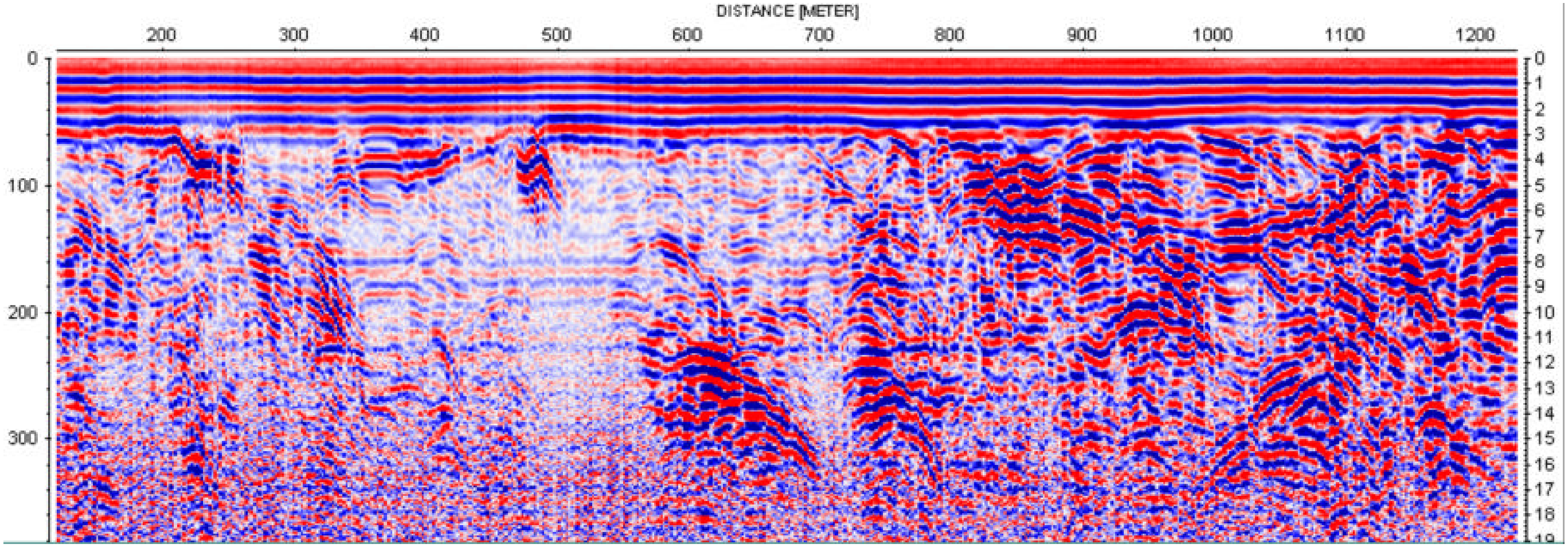
20

P1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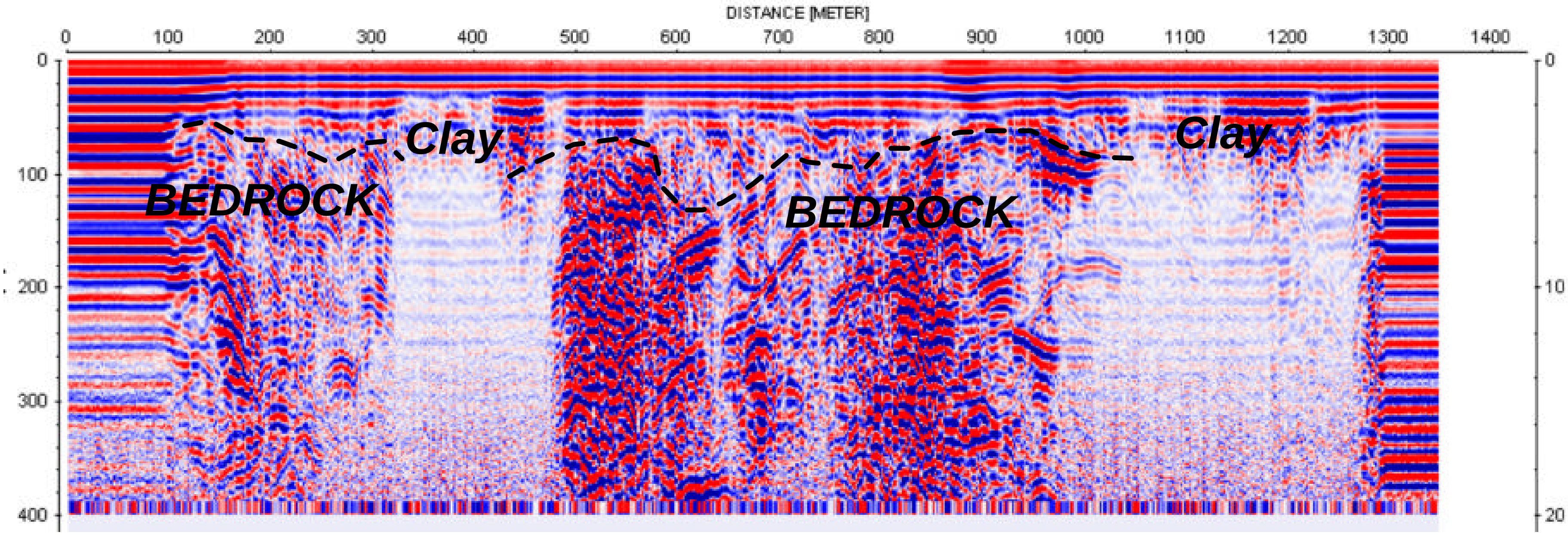
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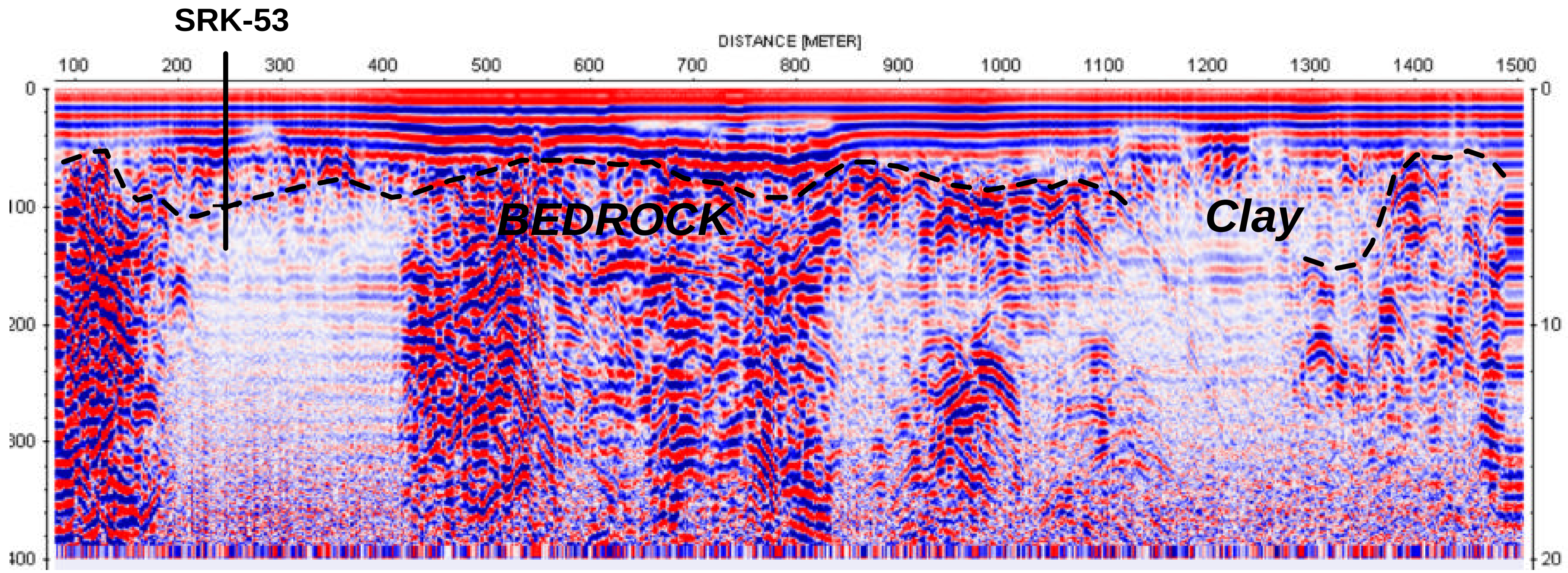
20
Clay
Clay
BEDROCK
BEDROCK

P3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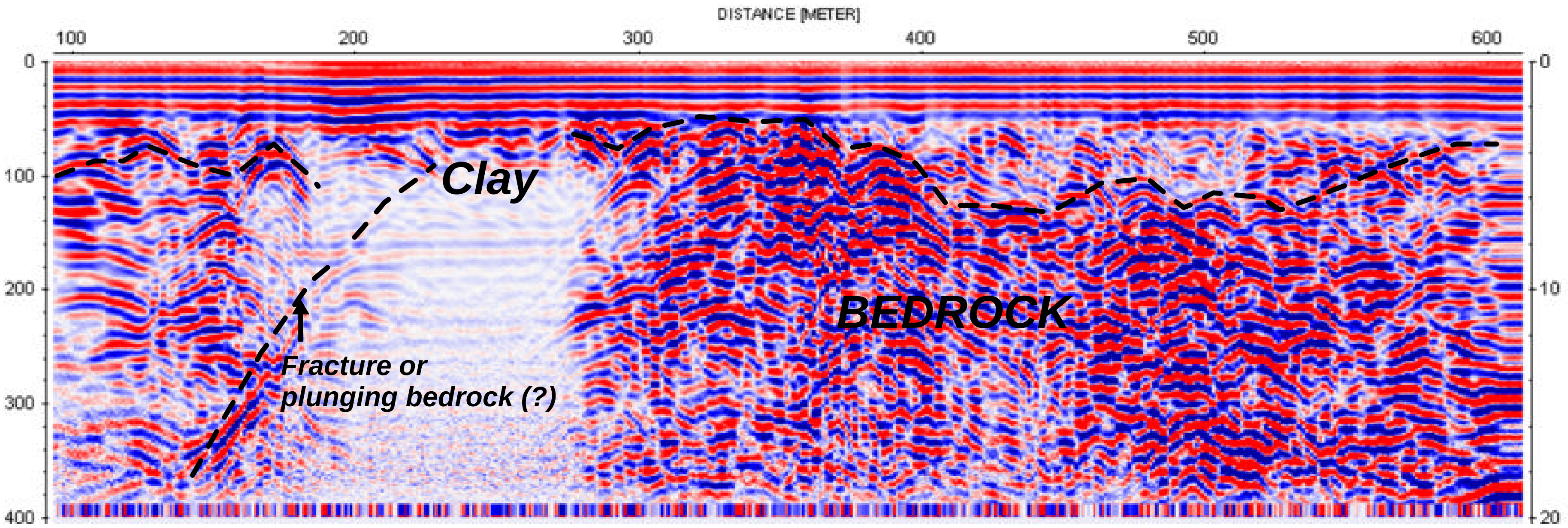
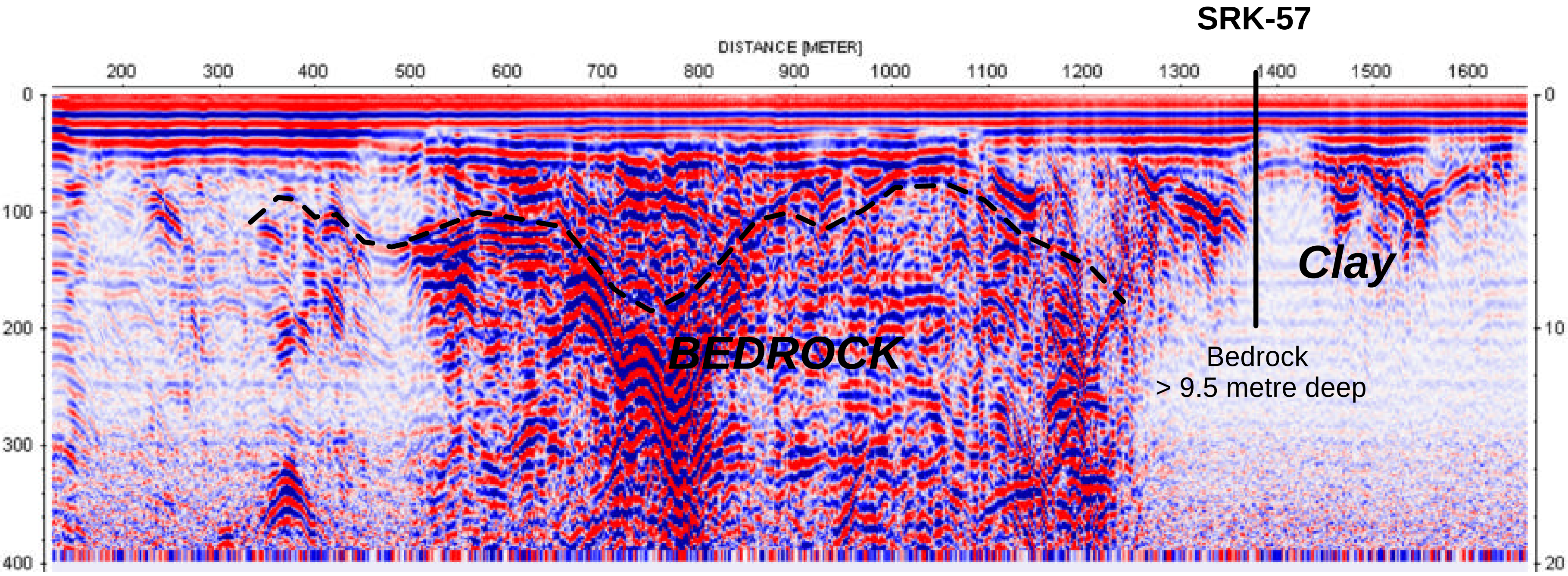
20
BEDROCK
Clay

P5



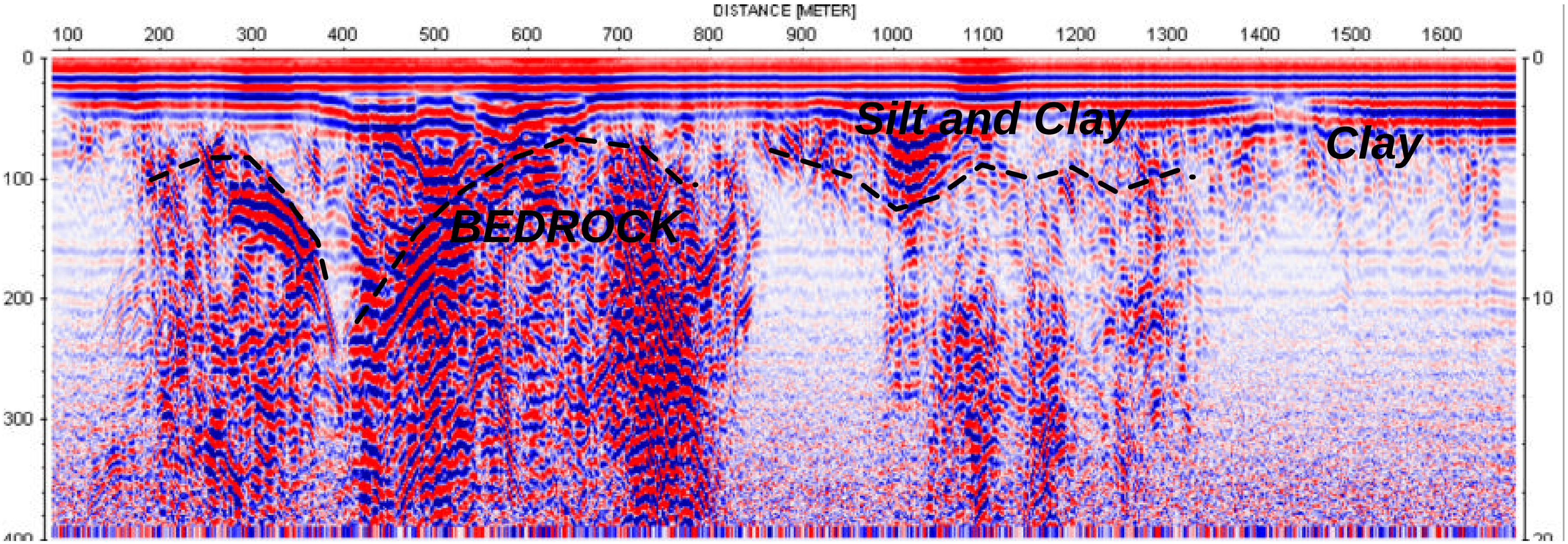
Fracture or plunging bedrock (?)

P6



Bedrock
> 9.5 metre deep

P7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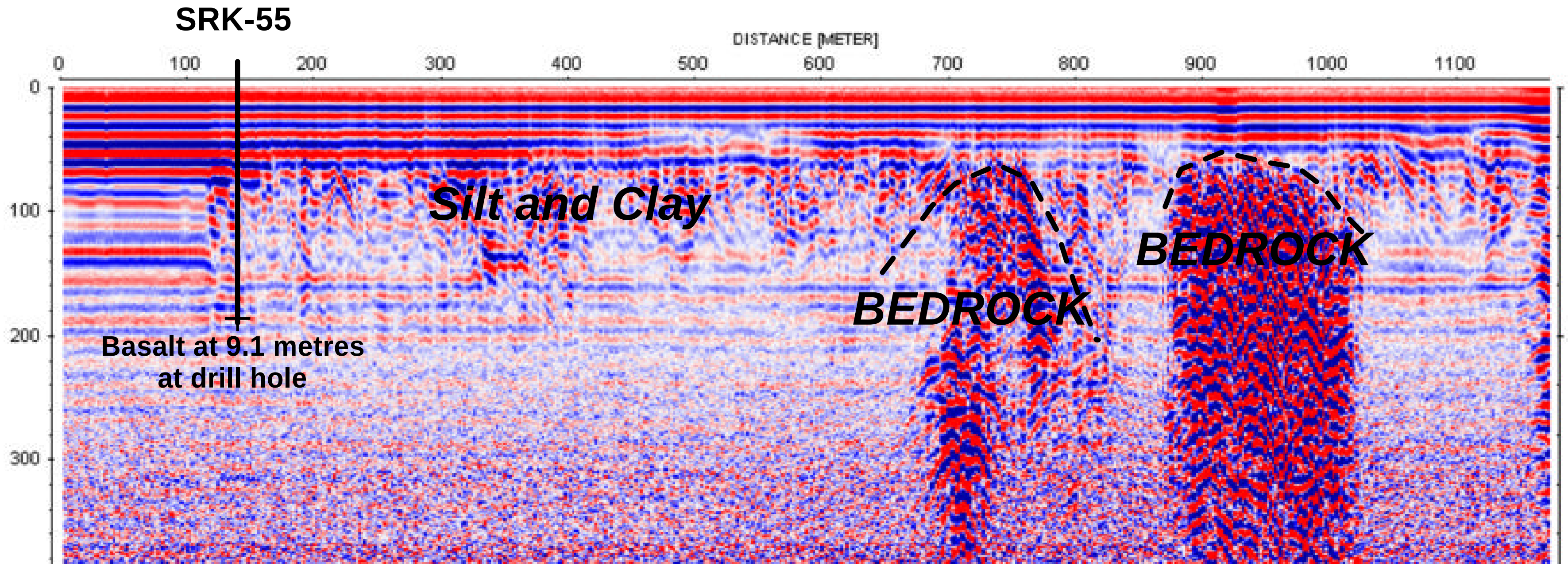
20
Silt and Clay
Clay
BEDROCK

P8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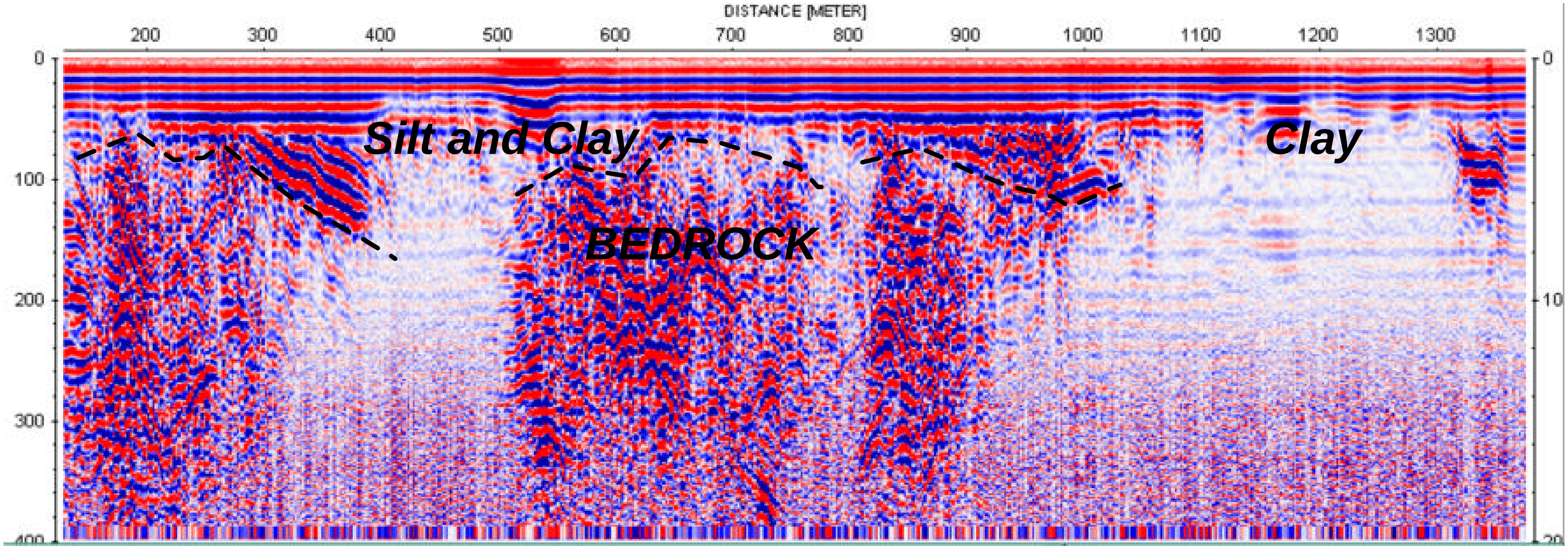
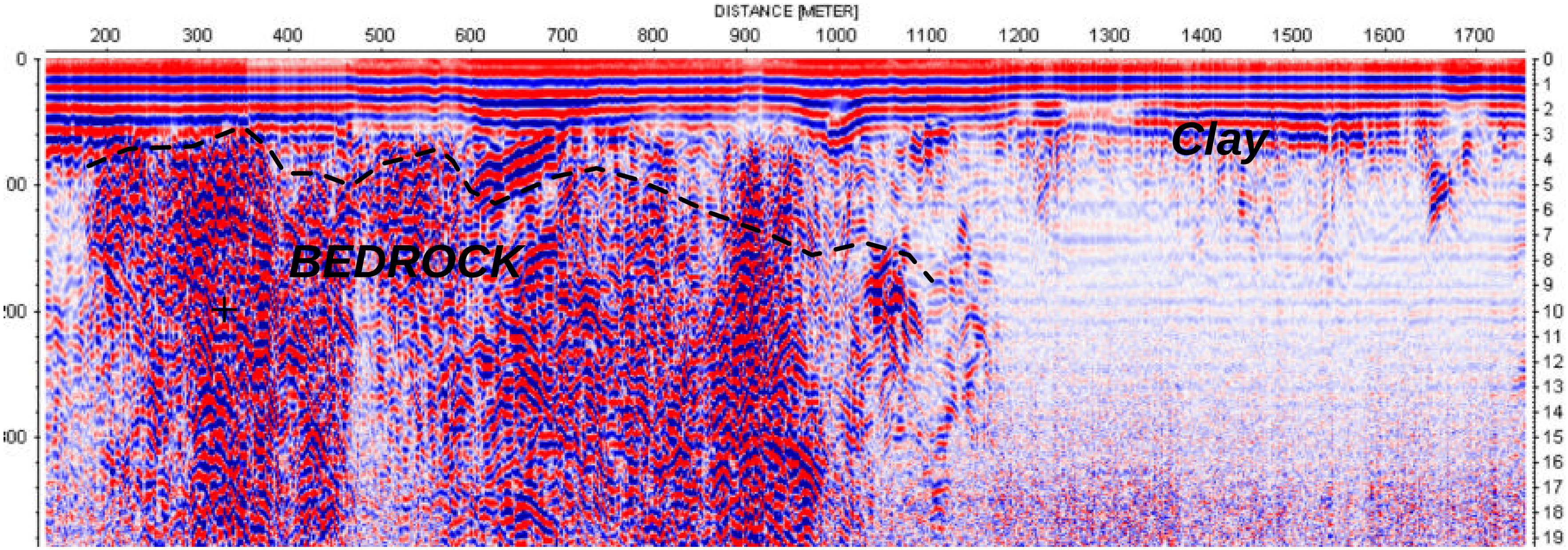
10
Silt and Clay
BEDROCK
BEDROCK
Basalt at 9.1 metres
at drill hole

P9



P10

End of West Bank (above) - Beginning of East Bank (Below)



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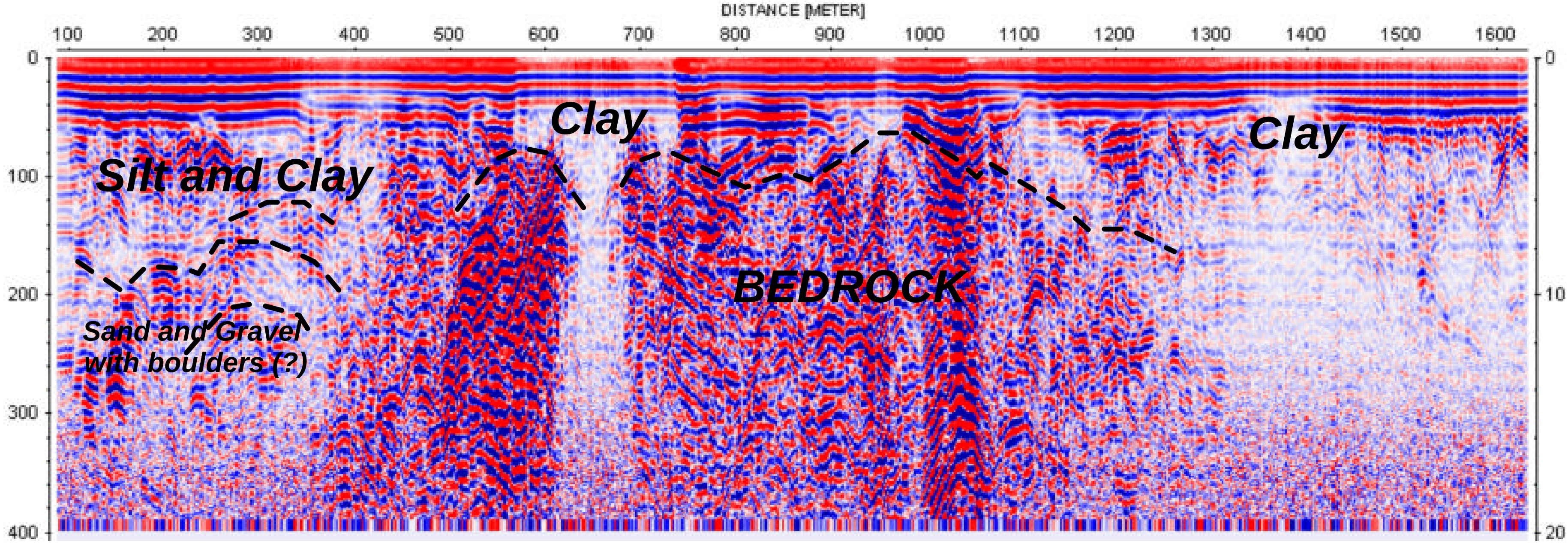
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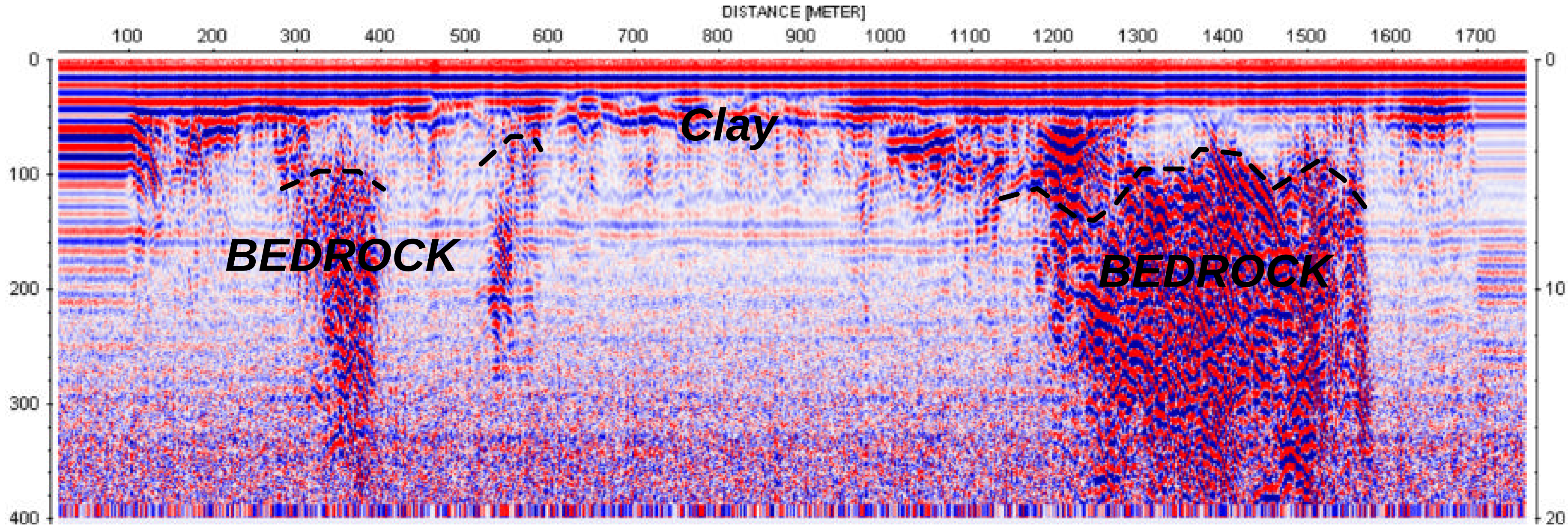
20
Silt and Clay
Clay
Clay
Sand and Gravel
with boulders (?)
BEDROCK

P13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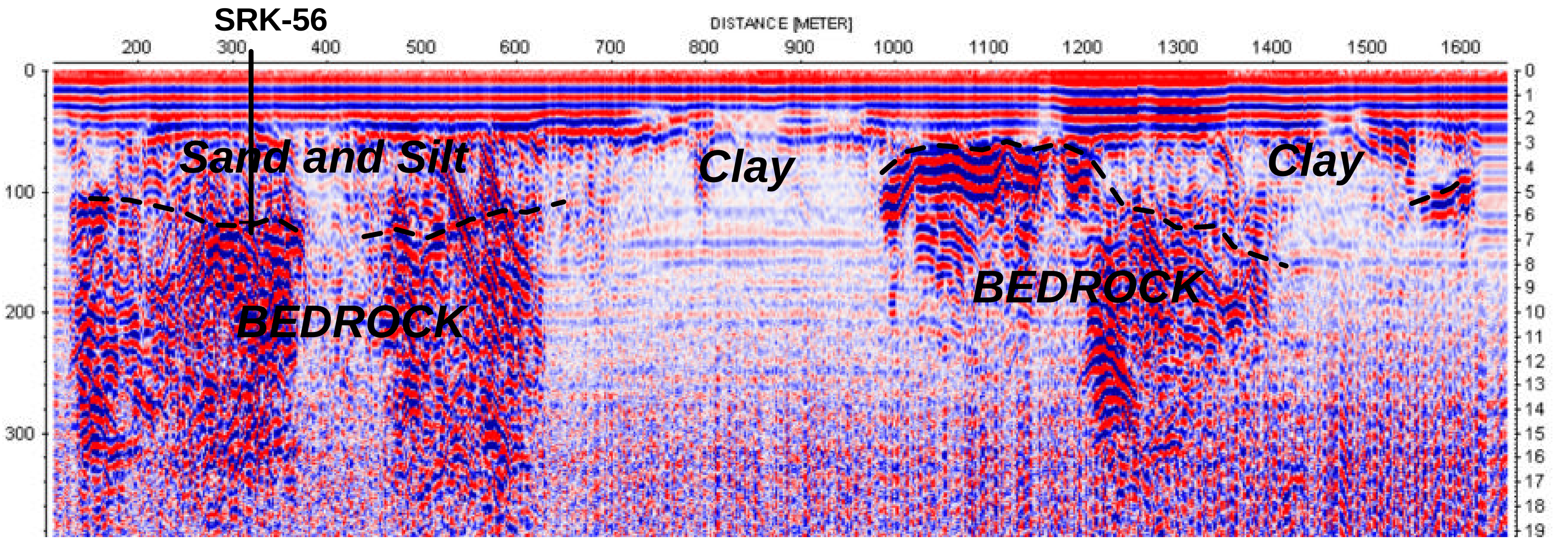
20
Clay
BEDROCK
BEDROCK

P14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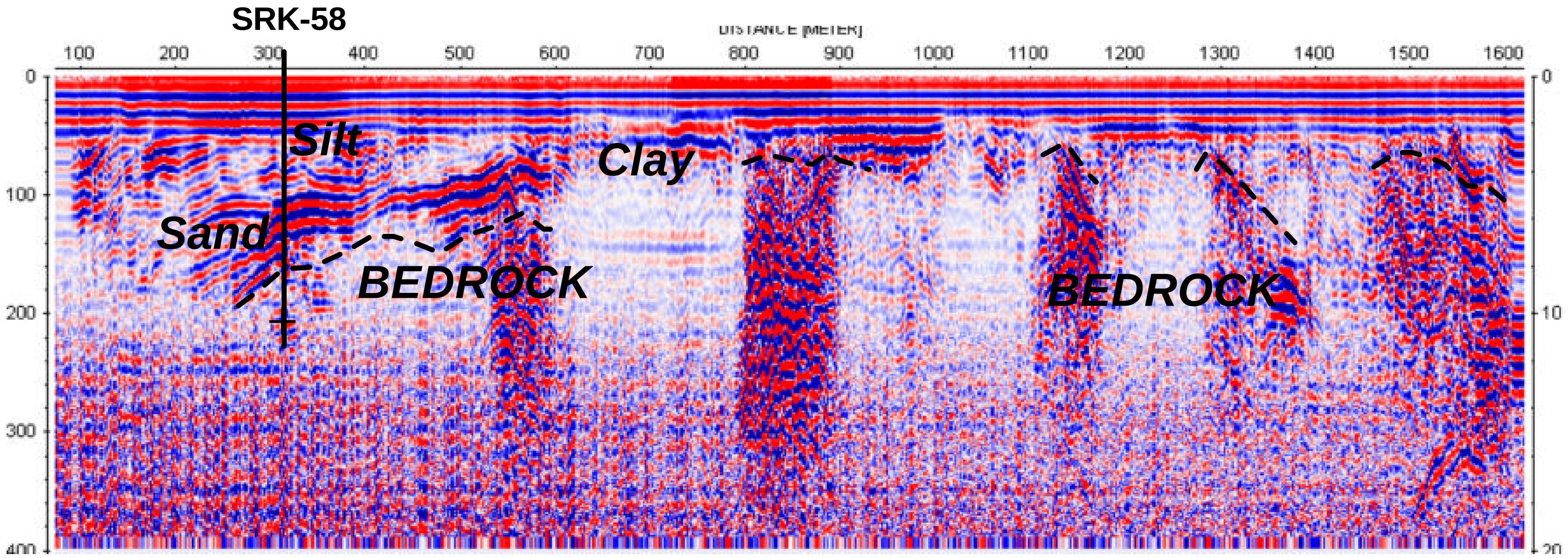
Sand and Silt
Clay
Clay
BEDROCK
BEDROCK

P15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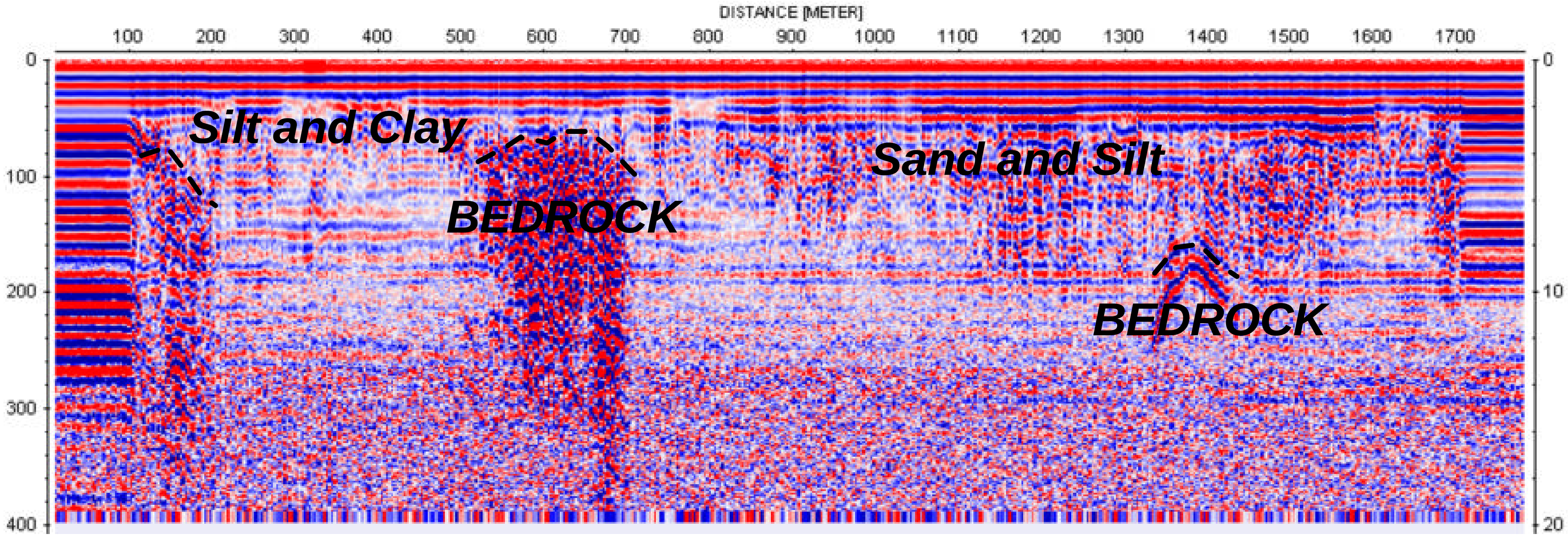
Silt
Clay
Sand
BEDROCK
BEDROCK

P16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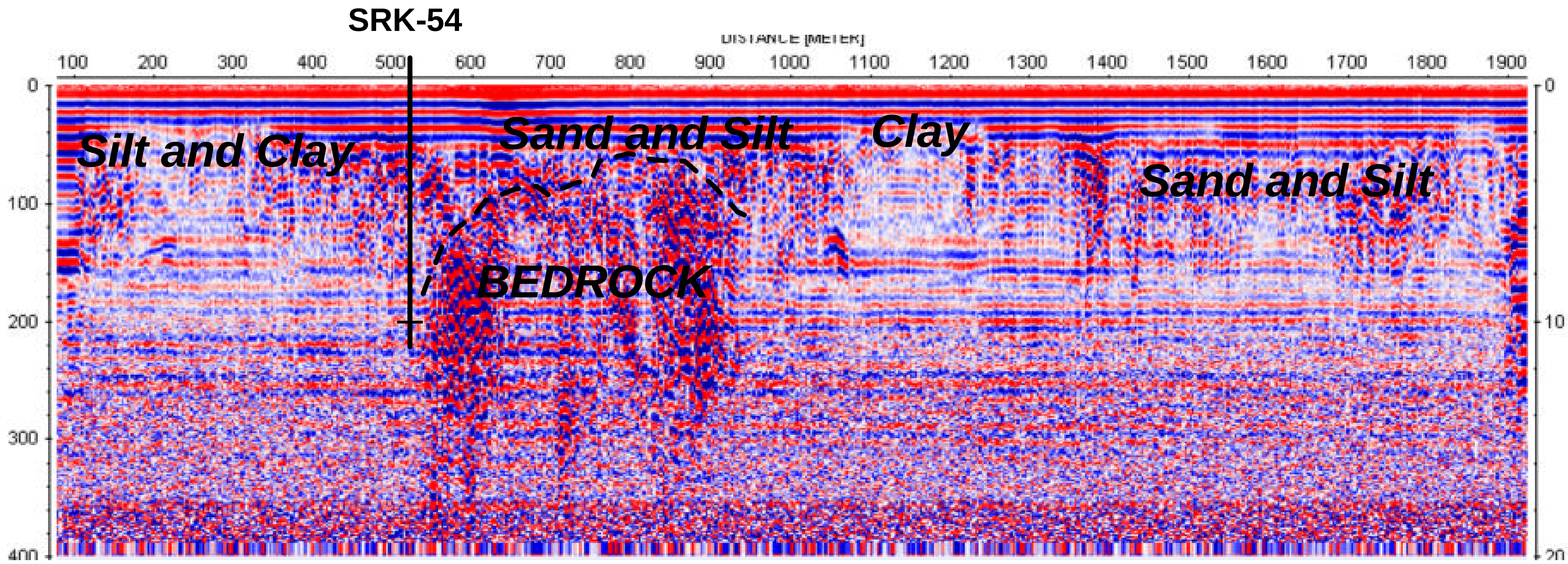
Silt and Clay
BEDROCK
Sand and Silt
BEDROCK

P17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1700
1800
1900
0
100
200
300
400
0
10
20
Silt and Clay
Sand and Silt
Clay
Sand and Silt
BEDROCK

P18

Appendix E

SRK Technical Memorandum re: Wave Run-up Calculations

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Technical Memorandum

To:	Brian Labadie	**Date:**	September 6, 2005
cc:	Project File	**From:**	Lowell Wade, Maritz Rykaart
Subject:	Wave Run-up Calculation to Determine Hydraulic Freeboard for Doris North Project Tailings Dam	**Project #:**	1CM014.006

1 Introduction

This technical memorandum documents the wave run-up calculations that were used to determine the appropriate wave run-up portion of the hydraulic freeboard design height for the North Dam of the Doris North Project.

2 Previous Wave Run-up Calculation

A preliminary calculation of the wave run-up was documented in SRK (2005), and used empirical tables correlating wind speed, fetch and wave height (USDI 1987). Based on an arbitrary maximum wind speed of 160 km/hr, with a maximum fetch distance of 3,200 m (equal to the maximum distance between the North and South Dams); the resultant wave height would be 1.13 m, requiring a 1.7 m vertical freeboard height. Since this value is obviously too conservative, a more rigorous assessment of the wave run-up height was carried out as described in the following sections.

3 Wind Speed Determination

A long term database of site specific wind data is not available for the Doris North Project site. The data that is available includes two years of data from a weather station at Doris Lake, as well as approximately 5 years of data at the Boston site 60 km south of the Project (Golder 2005a, b; AMEC 2003). Golder (2005b) carried out a correlation between wind data at Roberts bay (4 km north of the site) with wind data from Cambridge Bay (160 km north of the site); however, this involved only a few days of data. Neither of these data sets is sufficient to estimate wind speeds required for wave run-up calculations.

SRK contracted Mr. Pat Bryan, P.Eng., an associate hydrologist to determine design wind speeds for any given recurrence interval that could be used at the Doris North site.

The estimation of extreme winds entailed a three step process. The first step was to extract an annual series of annual maximum hourly wind speed from the climate record of the Cambridge Bay Airport (see Table 1). This station has been measuring wind speed since 1953 and its record now spans 52 years. The largest event on record occurred on October 3, 1974 and attained an hourly average wind speed of 101 km/h.

The second step involved fitting the annual series of maximum wind speeds to three different frequency distributions to estimate extreme wind speeds for a variety of return periods. The results of the analysis are presented in Table 2. All three distributions provided reasonably similar estimates for all return periods from 2 to 500 years. The largest estimate of the 500-year event was generated by the Log-Pearson Type III distribution and is only 11% greater than the smallest estimate, which was generated by the Generalized Extreme Value distribution.

Table 1: Observed Annual Maximum Hourly Wind Speeds at Cambridge Bay.

Calendar Year	Annual Maximum Hourly Wind Speed (km/h)
1953	71
1954	80
1955	68
1956	71
1957	76
1958	80
1959	89
1960	69
1961	76
1962	71
1963	89
1964	69
1965	72
1966	68
1967	69
1968	72
1969	85
1970	84
1971	80
1972	89
1973	82
1974	101
1975	87
1976	97
1977	80
1978	93
1979	83
1980	89
1981	65
1982	67
1983	70
1984	69
1985	80
1986	80
1987	70
1988	69
1989	74
1990	83
1991	74
1992	74
1993	67
1994	70
1995	82
1996	76
1997	74
1998	83
1999	65
2000	74
2001	80
2002	65
2003	70
2004	65
Minimum	**65**
Average	**76.7**
Maximum	**101**

Table 2: Estimated Annual Maximum Hourly Wind Speeds (km/h) at Cambridge Bay for Various Return Periods.

Return Period (years)	Frequency Distribution Used to Predict Extreme Hourly Wind Speeds:		
	Generalized Extreme Value	3-Parameter Lognormal	Log-Pearson Type III
2	75	75	74
5	84	83	83
10	89	89	89
20	95	95	96
50	102	104	105
100	107	111	112
200	112	118	120
500	118	128	131

The third step entailed determining how representative the Cambridge Bay wind data are of the mine site. This was done by comparing annual extremes at the two mine site weather stations with the corresponding annual extremes at Cambridge Bay. Table 3 tabulates the annual maximum wind speeds at the three stations over the period 2000 to 2004. The annual peak hourly wind speeds at the Boston station tended to be about 17% smaller than the annual peaks measured at Cambridge Bay. The peaks at Doris North, on the other hand, were nearly identical to the peaks observed at Cambridge Bay. Accordingly, these data suggest the Cambridge Bay data are reasonably representative of the mine site conditions. The Boston site provided four annual peaks for the comparison while the Doris Site provided two.

Table 3: Comparison of Maximum Wind Speeds at Cambridge Bay and Mine site Meteorological Stations.

Year	Meteorological Station					
	Cambridge Bay Airport		Boston		Doris North	
	Completeness of Annual Record (%)	Annual Maximum Hourly Wind Speed (km/h)	Completeness of Annual Record (%)	Annual Maximum Hourly Wind Speed (km/h)	Completeness of Annual Record (%)	Annual Maximum Hourly Wind Speed (km/h)
2000	99.8	74	99.7	57		n/a
2001	99.9	80	81.5	65		n/a
2002	100	65	99.2	57		n/a
2003	100	70	70.0	60	49.3	71
2004	100	65		n/a	90.1	64

4 Fetch Length

The maximum fetch length for any waves that may connect with the North Dam, for any wind direction, when the full supply level of 33.5 m has been reached in Tail Lake, is 1,326 m (northwest direction). Similarly, the maximum fetch length impacting the South Dam is 3,012 m (north-northwest direction).

5 Wave Run-up Calculations

For the wave run-up calculations the extreme wind speeds calculated according to the Log Pearson Type III method (Table 2) was used. Calculations were carried out for a 1:100 year and 1:500 year recurrence interval, at fetch lengths of 1,326 and 3,102 m.

Table 4 list the results of the wave run-up calculations according to the method described in Sorenson (1997, Section 2.9).

Table 4: Summary of Wave Run-up Calculation Results.

Parameter	North Dam		South Dam	
Fetch	1,326 m		3,012 m	
Recurrence Interval	1:100	1:500	1:100	1:500
Wind Speed (km/hr)	112	131	112	131
Wind Speed (m/sec)	31.1	36.4	31.1	36.4
Significant Wave Height (m)	0.58	0.68	0.87	1.02
Peak Spectral Period (sec)	2.2	2.3	2.8	3.0
Wavelength (m)	7.2	8.1	12.5	13.9
Wave Run-up, Smooth Surface (m)	0.33	0.38	0.50	0.59
Wave Run-up, Rip-rap Surface (m)	0.16	0.19	0.25	0.29

6 Conclusion

Based on the wave run-up calculations documented in this technical memorandum, the maximum hydraulic freeboard required to prevent overtopping of the dams due to wave run-up is 0.29 m.

7 References

AMEC Earth & Environmental Ltd. 2003. *Meteorology and Hydrology Baseline, Doris North Project*. November.

Golder Associates Ltd. 2005a. *Doris North Project Air Quality Assessment Methods*. Report No. 05-1373-008, May.

Golder Associates Ltd. 2005b. *Potential Impacts on Shorelines Due to Construction of a Jetty at Roberts Bay, Miramar Doris North Project*. Report No. 04-1373-009.4100, May.

Sorenson, R.M., 1997. *Basic Coastal Engineering*. Chapman & Hall. ISBN: 041212341X. pp. 288.

SRK Consulting (Canada) Inc. 2005. *Revised Dam Design, Preliminary Engineering, Hope Bay Doris North Project, Nunavut, Canada*. Project No. 1CM014.04. May.

United States Department of the Interior, Bureau of Reclamation. 1987. *Design of Small Dams*. A Water Resources Technical Publication, Third Edition, 66. 860.

Appendix F

SRK Technical Memorandum re: Water Balance Calculation

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Tel: 604.681.4196
Fax: 604.687.5532

Technical Memorandum

To:	Larry Connell, MHBL	**Date:**	March 30, 2007
cc:	Project File	**From:**	Maritz Rykaart
Subject:	Doris North Project TCA Water Balance – Revised and updated	**Project #:**	1CM014.008.165

1 Introduction

This Technical Memorandum describes the Doris North Project tailings containment area (TCA) water balance. This water balance has been used in conjunction with other relevant technical information to determine the design height of the TCA containment dams and water management strategy. This water balance also forms the basis for the water quality predictions in the Water Quality Model for the Project (SRK 2007a).

After initial completion of the water balance, the mill processing was changed, which also affects the water balance. Complete details of these changes are documented in SRK 2007a, and will therefore not be repeated her. This memo however does repeat the initial water balance calculations, but demonstrates the net effect of the changes by means of additional sensitivity runs.

2 Original Water Balance

2.1 Methodology

The water balance has been calculated using a custom, and site specific model developed for the Doris North TCA. This water balance model is build using an EXCELL spreadsheet calculation, and the resolution is based on monthly time steps, from the start of operations through to final closure and abandonment.

2.2 Primary Assumptions

Conservative, but reasonable engineering assumptions have been made to develop a realistic water balance for the TCA. These assumptions include;

- Tail Lake will be completely isolated with respect to surface and groundwater from the adjoining Doris Lake and Ogama Lake catchments by two water retaining dams.
- Tailings deposition will be sub-aqueous and will be managed such that the final tailings surface will be relatively horizontal.
- Tail Lake will not be pumped out prior to constructing the dams or starting deposition.
- The volume of Tail Lake at its normal full supply elevation of 28.3 m is ~ 2,196,000 m^3.
- Annual decant release from the TCA is planned; however, the TCA is designed as a zero discharge facility for the two-year mine life (at a constant production rate), plus an additional period of natural runoff after mining ceases.

- The impact of varying climate and hydrology on the water balance is illustrated with a sensitivity analysis.

- The water balance is calculated in monthly time steps. The water balance calculations use a year that starts in March and ends in February.

- All values in this water balance are expressed in terms of the dam FSL. It should be noted that at any time there will be a minimum 4 m freeboard height above the FSL, which serves as thermal and wave run-up protection.

2.3 Water Balance Calculation

The TCA has a total surface catchment area of 450 ha. A bathymetric survey of Tail Lake (Rescan 2001) and a topographical survey of the catchment were used to develop a stage curve for the TCA, as illustrated in Figure 1.

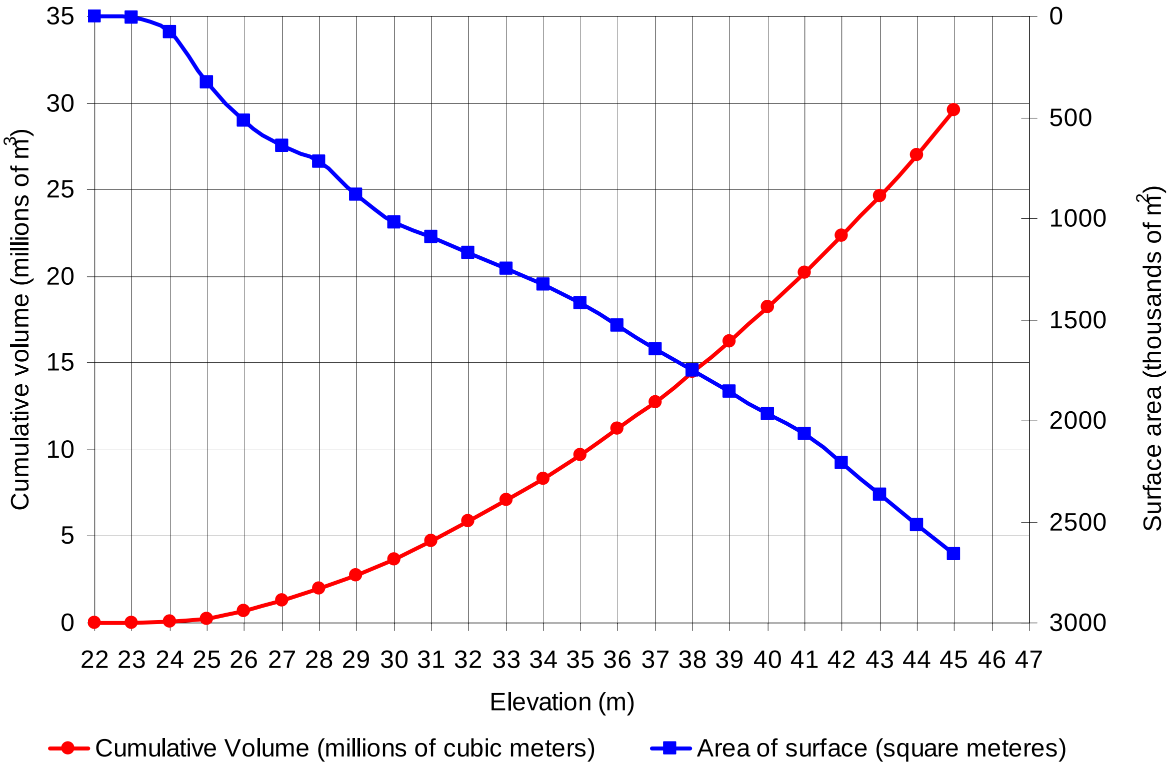

Figure 1: TCA Stage Curves.

The best-fit mathematical expressions for the TCA surface area and storage volume, expressed in terms of the lake water elevation are respectively given as:

$$V = -0.00020 \cdot h^3 + 0.07354 \cdot h^2 - 2.94811 \cdot h + 31.26314 \qquad \ldots(1)$$

$$A = 0.32656 \cdot h^6 - 67.95719 \cdot h^5 + 5829.42586 \cdot h^4 - 263467.30709 \cdot h^3 + 6607466.03356 \cdot h^2 - 86950808.77770 \cdot h + 467883520.90250 \qquad \ldots(2)$$

In addition an expression for the TCA water elevation, expressed in terms of the total storage volume is given as:

$$\begin{aligned} h = &-1.18740\times10^{-42}\cdot V^{6}+1.13280\times10^{-34}\cdot V^{5}-4.17315\times10^{-27}\cdot V^{4} \\ &+7.47625\times10^{-20}\cdot V^{3}-6.83698\times10^{-13}\cdot V^{2}+3.72593\times10^{-6}\cdot V+2.28955\times10^{1} \end{aligned} \qquad \ldots(3)$$

Where V = TCA pond storage volume (m^3),
A = TCA pond surface area (m^2), and
h = TCA pond elevation (m).

The TCA water balance is schematically illustrated in Figure 2. The water balance is calculated in monthly time steps using the following expression:

$$\Delta S = Q_1 - Q_2 + Q_3 + Q_4 - Q_5 - Q_6 \pm Q_7 + Q_8 + Q_9 \qquad \ldots(4)$$

Where ΔS = change in TCA storage volume (m^3),
Q_1 = volume of direct precipitation falling onto TCA (m^3),
Q_2 = volume of potential evaporation from TCA (m^3),
Q_3 = volume runoff entering TCA (m^3),
Q_4 = volume of tailings feed pumped to TCA (m^3),
Q_5 = volume of reclaim water pumped back to mill (m^3),
Q_6 = volume of decant allowed from TCA (m^3),
Q_7 = volume of dam(s) seepage pumped back to TCA (m^3),
Q_8 = volume of sewage water sludge pumped to TCA (m^3), and
Q_9 = volume of underground mine water pumped to TCA (m^3).

The individual components Q_1, Q_2, and Q_3 are calculated as follows:

$$Q_1 = (A_{lake})P \qquad \ldots(5)$$

$$Q_2 = (A_{lake})PE \qquad \ldots(6)$$

$$Q_3 = (A_{catchment} - A_{lake})Y \qquad \ldots(7)$$

Where A_{lake} = surface area of TCA (m^2),
$A_{catchment}$ = total area of TCA catchment (m^2),
P = total precipitation (m),
PE = potential evaporation (m), and
Y = water yield (m^3).

The total precipitation (P) is calculated as follows:

$$P = R + S \qquad \ldots(8)$$

Where R = total rainfall (m), and
S = total snowfall water equivalent (m).

The tailings feed volume, Q_4, is calculated as follows:

$$Q_4 = Q_{10} + Q_{11} \qquad \ldots(9)$$

$$Q_{10} = \frac{F}{\rho_d} \qquad \ldots(2.10)$$

$$\rho_d = \frac{G_s}{(1+e)} \qquad \ldots(2.11)$$

$$Q_{11} = \frac{F}{\omega} \cdot (100 - \omega) \qquad \ldots(2.12)$$

Where Q_{10} = the volume of tailings solids (m^3),
Q_{11} = the volume of water entrained in the tailings slurry (m^3),
F = the tailings slurry feed (tonnes/day),
ρ_d = the tailings solids dry density (tonnes/m^3),
G_s = the tailings solids specific gravity,
e = the tailings void ratio for hydraulically placed tailings under water (-), and
ω = the tailings slurry feed solids (by weight) content (%).

The impoundment seepage Q_7 is calculated as follows:

$$Q_7 = Q_{12} + Q_{13} - Q_{14} \qquad \ldots(2.13)$$

Where Q_{12} = volume of seepage pumped back from the North Dam (m^3),
Q_{13} = volume of seepage pumped back from the South Dam (m^3), and
Q_{14} = volume of seepage lost to deep deep recharge (m^3).

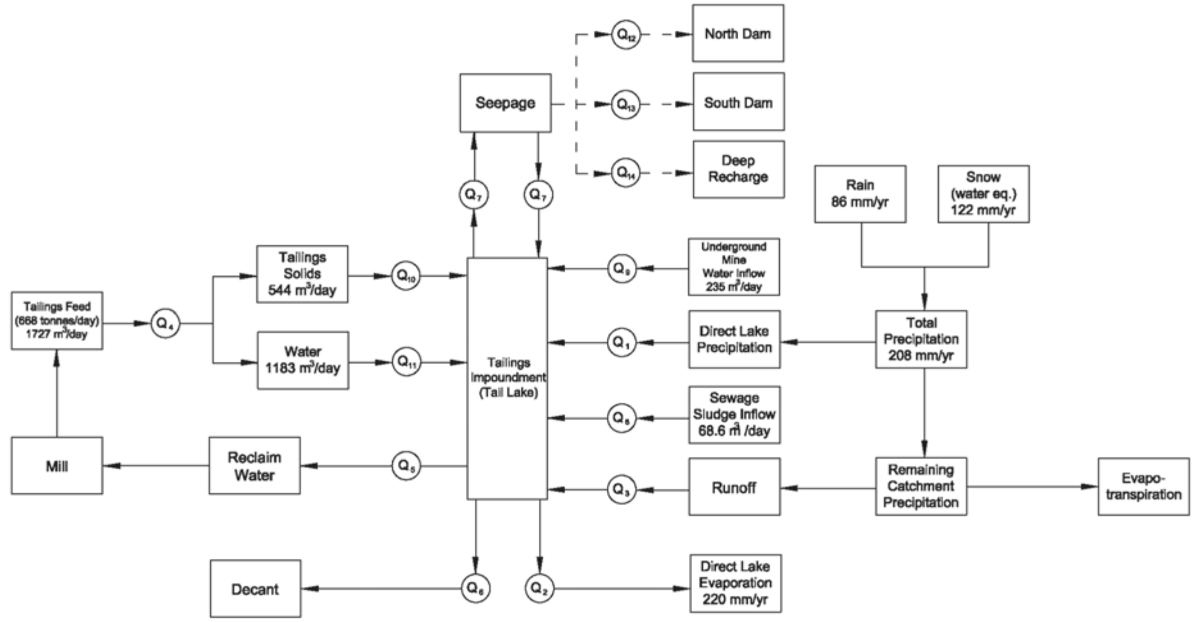

Figure 2: TCA Water Balance.

2.4 Water Balance Data Set

2.4.1 Total Precipitation

AMEC (2003) developed a detailed metrological and hydrological baseline for the Doris North Project, specifically targeting the Tail Lake and Doris Lake catchments. This document was used for the primary climatic data in the water balance. The mean monthly rainfall, snowfall (water equivalent) and total precipitation data extracted from AMEC (2003) is reproduced as Table 1.

Table 1: Mean monthly precipitation data

Month	Rainfall (mm)	Snowfall – Water Equivalent (mm)	Total Precipitation (mm)
January	0.0	8.0	8.0
February	0.0	8.4	8.4
March	0.0	10.8	10.8
April	0.1	11.8	11.9
May	3.2	12.5	15.7
June	13.7	3.6	17.3
July	24.2	0.2	24.4
August	29.0	1.9	30.9
September	14.4	12.5	26.9
October	1.2	25.8	27.0
November	0.0	14.4	14.4
December	0.0	11.6	11.6
Annual	85.8	121.5	207.3

Golder (2006) presents a detailed discussion of the meteorological and hydrology data with respect to possible variances from the baseline. That information will not be reproduced in this report; however, the impact of those variations had been thoroughly tested in a series of water balance sensitivity analysis.

With respect to mean annual precipitation (MAP), an increased value of 225 mm was suggested, and this value was subsequently used in the sensitivity analysis for the water balance.

In general the water balance is conducted using average climatic year data; however, it is recognized that extreme events can affect the outcome. The water balance sensitivity analysis therefore includes an evaluation of extreme wet and dry years. These extreme events are documented in Golder (2006).

2.4.2 Potential Lake Evaporation

AMEC (2003) did an extensive evaluation of the limited lake evaporation data for Doris North and developed the data set reproduced in Table 2, which was used in the water balance calculation to establish Q_3 in Equation 7. MHBL (2005) documents, that for sensitivity analysis, it would be appropriate to alter the evaporation to ±20% of this value, i.e. varying the evaporation between 176 and 264 mm.

Table 2: Monthly and annual lake evaporation data

Period	Days with Evaporation	Average Evaporation (mm)
June	15	35
July	31	95
August	31	77
September	30	13
Annual	105	220

2.4.3 Runoff/Water Yield

AMEC (2003) documents the annual outflow from Doris and Tail Lake. Based on this data they determined the annual water yield (runoff from the catchment) for Tail Lake to be 111 mm, and 134 mm for Doris Lake. Golder (2006) presents a detailed discussion on water yield, and concludes that the water yield could be as much as 180 mm/yr. For the purpose of the water balance presented in this report the base case water yield was conservatively assumed to be 180 mm, and the effect of the lower water yield was evaluated through sensitivity analysis.

2.4.4 Seepage

Seepage from the TCA can be via three primary routes; North Dam, South Dam and deep recharge through the lake basin. In reality, the North and South Dams will be frozen core dams, which should not have any seepage. Furthermore, in the event that seepage was to occur, MHBL would intercept this seepage and return it to Tail Lake.

It was however considered appropriate to estimate what this seepage may be, such that an evaluation could be made as the potential effect that it may have on the water balance. Since the seepage rates at any of these points cannot be physically measured at this time, first order seepage calculations were made using the D'arcy equation (Holtz and Kovacs 1981):

$$q = k \cdot i_{north/south} \cdot A = k \cdot \frac{\Delta h_{north/south}}{L_{north/south}} \cdot A \qquad \text{…(14)}$$

$$A = d_{unfrozen} \times w_{unfrozen} \qquad \text{…(15)}$$

$$\Delta h_{north/south} = h_{\max} - h_{north/south} \qquad \text{…(16)}$$

$$L_{north/south} = d_{crest} \cdot \left(\Delta h_{north/south} \times S_u\right) \cdot \left(\Delta h_{north/south} \times S_d\right) \qquad \text{…(17)}$$

Where q = seepage flow rate (m^3/s),
k = tailings permeability (m/s),
$i_{north/south}$ = hydraulic gradient for North or South dam respectively (-),
A = seepage area (m^2),
$\Delta h_{north/south}$ = change in headloss for North and South dam respectively (m),
$L_{north/south}$ = Length of seepage zone for North and South dam respectively (m).

Table 3 lists the assumed constants for the theoretical seepage calculations. The resultant seepage rates for each potential seep are listed in Table 4. The permeability used for these calculations are an average, maximum or minimum value to provide a range of seepage results. These permeability values are based on typical gold tailings data.

Table3: Constants used in the seepage calculations

Parameter	Symbol	Value
Maximum tailings impoundment water level	h_{max}	33.5m above sea level
North dam foundation elevation	h_{north}	26.5m above sea level
South dam foundation elevation	h_{south}	32.0m above sea level
Assumed thickness of "unfrozen" zone	$d_{unfrozen}$	15 m
Assumed width of "unfrozen" zone	$w_{unfrozen}$	35 m
Dam crest width	d_{crest}	10 m
Upstream Dam slope	$1:S_u$	6 (i.e. 1 vertical:6 horizontal)
Downstream Dam slope	$1:S_d$	4 (i.e. 1 vertical:4 horizontal)
Average tailings permeability	$k_{s(avg)}$	1 x 10^{-5} m/sec
Minimum tailings permeability	$k_{s(min)}$	1 x 10^{-6} m/sec
Maximum tailings permeability	$k_{s(max)}$	1 x 10^{-4} m/sec

Table 4: Estimated seepage rates through North and South Dams

Condition	North Dam (m^3/day)	South Dam (m^3/day)	Total (m^3/day)
Minimum	6	1	7
Average	61	13	74
Maximum	607	137	744

The deep recharge component refers to the potential for deep recharge via the foundation materials of the TCA, i.e. Tail Lake. The intact permeability of these foundation materials was measured by SRK using packer tests (SRK 2003). The average hydraulic conductivity was 9.9 x 10^{-12} m/sec. The reason for this low permeability was twofold; the ground was frozen, and the bedrock was not fractured.

In calculating the deep seepage an assumption has been made that the pool size is constant at 131 ha (the surface area covered by Tail Lake at FSL), the hydraulic gradient is constant at 1, and foundation permeabilities are as follows; average = 9.9 x 10^{-12} m/s; minimum = 9.9 x 10^{-13} m/s; maximum = 9.9 x 10^{-11} m/s. The resultant deep recharge rates are listed in Table 5.

Table 5: Estimated deep recharge rates for Tail Lake

Condition	Deep Recharge (m^3/day)
Minimum	0.1
Average	1.1
Maximum	11.1

The average condition theoretical seepage calculations described above have been used in the TCA water balance. It was however assumed that all seepage from the North and South Dams would be intercepted and pumped back to the TCA. The average deep seepage rate is so low that it has been omitted from any water balance calculations. Since the TCA is designed for full containment, this was deemed to be conservative.

2.4.5 Tailings Slurry Feed

The average constant tailings production rate will be 668 tonnes/day at a solids moisture content of 36.1%, a solids specific gravity of 2.7 and a submerged in-place tailings void ratio of 1.2. This will result in a daily slurry feed of 1,727 m^3 (544 m^3/day solids and 1,183 m^3/day water).

2.4.6 Reclaim Water

Tail Lake is relatively shallow, and it is possible that during the winter months increased turbidity may be experienced as a result of a reduced water volume in the lake created by the freezing conditions (lake-ice). Consequently, 100% recirculation water (1,183 m^3/day) is assumed for four months of the year only (June through September). During the remainder of the year fresh water make-up will be from Doris Lake.

2.4.7 Sewage Sludge Volume

The sewage treatment plant outflow and sludge will be pumped to the TCA as part of the tailings feed stream. This volume of sludge is dependant of the size of the camp. For the purpose of this water balance calculation we have assumed a 175-man camp, for a total sewage treatment plant load of ~ 68.6 m^3/day (value supplied by MHBL).

2.4.8 Underground Volume

Mining at the Doris North Project will exploit the Doris Hinge reserves, which are located north of Doris Lake. There is known permafrost in this region, and therefore it would be reasonable to assume that water make-up in the mine would be negligible.

SRK (2005) documents the results of a scoping level geohydrological model for a mining scenario where the Doris Connector and Doris Central sections are exploited. Under this scenario, mining will move underneath Doris Lake and it is conceivable that there could be an inflow of water into the mine. This inflow has been estimated to average 235 m^3/day.

Although the two-year Doris North project would in all likelihood not experience any mine water inflow (SRK 2005), a conservative assumption has been made that a mine inflow of 235m^3/day would occur for the life of the project. This water would be captured in the mine and pumped to Tail Lake.

2.4.9 Decant Rate

Baseline hydrology prepared by AMEC (2003) confirmed that flow from Tail and Doris Lakes only occurs during the period June through October. Any outflow from Tail Lake will flow into Doris Lake, immediately upstream from Doris Creek.

A tailings management strategy is proposed that would allow annual discharge from Tail Lake directly into Doris Creek, at a location immediately upstream of a 4.3 m high waterfall (SRK 2007b).

2.5 Water Balance Results

The primary purpose of the water balance was to determine an appropriate height for the containment dams, such that there would be sufficient storage capacity in Tail Lake. Therefore, the water balance was initially applied for an arbitrary period of 25 years. During this time the facility was kept as a full containment facility, and the height of containment dams to retain this volume was calculated. This 25 year period, includes two years of mining. The results of this analysis are presented in Table 6.

Table 6: Required FSL for full containment over 25 years

Scenario	MAP (mm)	Evap. (mm)	Water Yield (mm)	Extreme Events	Recycle (months)	Mine Water (m^3/day)	Required FSL (m)
Base Case (#1)	207	220	180	None	4	235	39.4
#2	225	220	180	None	4	235	39.7
#3	207	220	111	None	4	235	36.4
#4	207	176	180	None	4	235	40.1
#5	207	264	180	None	4	235	38.7
#6	207	220	180	Yr 1 = 1:500 Wet	4	235	39.7
#7	207	220	180	Yr 1 = 1:500 Wet; Yr 2 = 1:100 Wet	4	235	39.8
#8	207	220	180	Yr 1 = 1:100 Dry	4	235	39.3
#9	207	220	180	Yr 1 = 1:100 Dry; Yr 2 = 1:100 Dry	4	235	39.1
#10	207	220	180	None	0	235	39.5
#11	207	220	180	None	12	235	39.2
#12	207	220	180	None	4	0	39.3
#13	300	220	180	None	4	235	40.8
#14	207	233	111	None	4	235	36.1
#15	115	228	42	None	4	235	30.8
#16	158	228	84	None	4	235	33.8
#17	132	306	53	None	4	235	30.4
#18	207	220	131	None	4	235	37.5

Table 7 list the required FSL in Tail Lake at different times, as calculated using the Base Case (Scenario #1) input parameters. Through an iterative procedure, and in consultation with MHBL, it was determined that an optimal design FSL in Tail Lake would be 33.5 m. Based on that water level, a sensitivity analysis was completed to illustrate the time to overflow. These results are presented in Table 8.

Table 7: Base Case (Scenario #1) required FSL at different time intervals

Time Interval	Required FSL (m)
25 years	39.4
20 years	38.1
15 years	36.4
10 years	34.1
5 years	32.5

Table 8: Calculated decant time based on a design FSL of 33.5 m (start date is March 2008)

Scenario	Decant Time (Date)	Decant Time (Years/Months from Start
Base Case (#1)	August 2015	7 years, 5 months
#2	July 2015	7 years, 4 months
#3	July 2020	12 years, 4 months
#4	December 2014	6 years, 9 months
#5	July 2016	8 years, 4 months
#6	July 2014	6 years, 4 months
#7	August 2013	5 years, 5 months
#8	July 2016	8 years, 4 months
#9	October 2016	8 years, 6 months
#10	May 2015	7 years, 2 months
#11	July 2016	8 years, 4 months
#12	January 2016	7 years, 10 months
#13	July 2014	6 years, 4 months
#14	July 2016	8 years, 4 months
#15	Never	Never
#16	May 2030	22 years, 2 months
#17	Never	Never
#18	Aug 2018	10 years, 5 months

The effect of decanting water from Tail Lake on the water level in Tail Lake is illustrated in Table 9. As the volume of annual decant is increased the time until the FSL is reached is increased. At some point the discharge is greater than the annual inflow and the FSL will not be reached. If the annual discharge is greater than the annual inflow, the water level in Tail Lake would decrease over time.

Table 9: Effect of decant on the Base Case (Scenario #1)

Decant Scenario	Decant Time (Date)	Comment
100,000 m^3/year	June 2017	This is more than 9 years before FSL is reached
250,000 m^3/year	July 2021	This is more than 13 years before FSL is reached
500,000 m^3/year	Likely Never	Max. water level of 33.0m reached after 42 years
750,000 m^3/year	Likely Never	Max. water level of 29.9m reached June 2010; Water level drops to 28.3 by August 2019
1,000,000 m^3/year	Likely Never	Max. water level of 29.4m reached June 2010; Water level drops to 28.3 by September 2010

The water balance illustrated that an operating FSL of 33.5 m for Tail Lake would be appropriate. Under the most conservative water balance assumptions, Tail Lake can operate as a zero discharge facility for just under 5½ years before reaching FSL. Using more realistic water balance assumptions Tail Lake can operate as a zero discharge facility for at least 7½ years.

The water balance also clearly illustrates that, by allowing an annual discharge the time to reach FSL in Tail Lake is dramatically increased. Allowing as little as 100,000 m^3/year of discharge increases the time to FSL under the base case (Scenario #1) to just under 9½ years, which is a 27% increase in time. If the annual discharge is 500,000 m^3/year, FSL in Tail Lake will likely not be reached, since the decant rate will exceed the annual inflow.

3 Additional Sensitivity Runs

As is illustrated in SRK 2007 the mill processing has changed, and also a portion of the tailings stream will be used as backfill. The details of these changes are completely documented in SRK 2007a and will not be repeated here. To demonstrate that these changes has no material affect on the TCA water balance and therefore the primary impoundment design assumptions are still valid, the first 11 scenarios were rerun using the new input assumptions. The subsequent results are listed in Table 10, which is directly comparable to Table 8. Clearly there is only a few months difference, with the average increase in time to reach FSL being between 1 and 3 months.

Table 10: Calculated decant time based on a design FSL of 33.5 m (start date is March 2008) (modified mill processing numbers)

Scenario	Decant Time (Date)	Decant Time (Years/Months from Start
Base Case (#1)	November 2015	7 years, 8 months
#2	August 2015	7 years, 5 months
#3	December 2020	12 years, 9 months
#4	April 2015	7 years, 1 month
#5	August 2016	8 years, 5 months
#6	August 2014	6 years, 5 months
#7	September 2013	5 years, 6 months
#8	November 2016	8 years, 8 months
#9	January 2017	8 years, 9 months
#10	August 2015	7 years, 5 months
#11	August 2016	8 years, 5 months

4 References

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Appendix G

SRK Technical Memorandum re: Shoreline Erosion Protection Adaptive Management Plan

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Technical Memorandum

To:	Larry Connell	**Date:**	February 2, 2007
cc:	Project File	**From:**	Maritz Rykaart
Subject:	Shoreline Erosion Protection Adaptive Management Plan	**Project #:**	1CM014.008.165

1 Motivation for Adaptive Management

The proposed water management strategy for the TCA will result in a maximum operational water level of 29.2 m, with a total flooded footprint of about 13 ha. The TCA has however been designed to operate as a zero discharge facility for a period of time, to account for unforeseen upset conditions, under which case the maximum FSL of 33.5 m may be reached in Tail Lake, which will result in an increased flooded footprint.

Irrespective of how large the flooded footprint may be, this surface area consists of ice-rich permafrost with predominantly silts and clays, which is expected to thaw when subject to submergence, and result in shoreline erosion which will in turn create turbidity concerns within Tail Lake. A comprehensive evaluation of the shoreline erosion processes, risks and mitigation measures are documented in SRK (2005), which is also included as an appendix to SRK (2007b).

Since it is neither practical nor reasonable to implement full preventative mitigation measures against shoreline erosion at the outset of the project, MHBL has committed to the development of an Adaptive Management Plan to address shoreline erosion. This Technical Memorandum provides the basis of this plan which will be developed as a stand-alone MHBL document prior to deposition of any tailings in Tail Lake.

2 Pro-Active Shoreline Erosion Implementation Measures

MHBL has committed that prior to tailings deposition about 20% of the flooded footprint up to elevation 29.2 m will be pro-actively mitigated. Areas to be pro-actively mitigated are indicated on Dwg. T-14 (SRK 2007c) and are areas that are subject to the greatest fetch distances. These are the areas that according to the best available understanding of shoreline erosion processes at the site would likely experience shoreline erosion first.

Pro-active erosion protection will consist of laying a geotextile directly onto the tundra and covering it with a 0.5 m thick layer of run of quarry material as illustrated in Dwg. T-14.

3 Shoreline Erosion Monitoring & Implementation Triggers

Adaptive management of the shoreline erosion mitigation measures will be contingent on monitoring. This monitoring will consist primarily of observational techniques. Appropriately trained mine operations staff will carry out daily visual inspections along the entire TCA shoreline taking note of, and recording any signs of shoreline erosion. Visual signs which would indicate that shoreline erosion is starting, or may soon start and that would trigger implementation of mitigation measures include:

- Surface abnormalities along the exposed shoreline, such as cracks, slumping and/or surficial erosion.
- Signs of turbidity along the shoreline, not directly attributable to tailings deposition.
- Signs of vegetation dieback as a result of being flooded.
- Signs of high-walls forming as a result of wave-action.

In addition, the following permanent instrumentation (as illustrated in Dwg. T-14) will be installed around the TCA perimeter (note that some of this instrumentation has already been put in place):

- *Thermistors:* 6 vertical thermistors have been installed along 6 important transects of the TCA perimeter, between elevations 28.3 m and 33.5 m. These thermistors will monitor the temperature profile in the overburden soils as the water level rises, giving advance warning of thaw. This data must be collected manually at least once a month. Should divergences from the normal data trends be observed, it will be a sign that mitigation measures will have to be implemented.
- *Survey transects:* 6 detailed strip surveys along the same six transects where the thermistors are installed will be monitored such that the extent of shoreline erosion can be determined. Each strip will be 50 m wide. Strip surveys should be redone annually, including a bathymetry survey of the underwater section of the strip down to the original TCA water level of 28.3 m (this should continue for five years after the FSL is reached or till there is no discernable profile difference in any two consecutive years). This will provide information of the slope morphology as time progresses, as well as allow for calculation of a sediment balance.

In addition to the observational and instrumentation monitoring, MHBL must include and inspection of the shoreline conditions, as well as a review of all monitoring data as part of the Annual Dam Safety Inspection Report by a qualified registered Professional Engineer in Nunavut. During this inspection, appropriate training of new staff with respect to monitoring may also be considered. The report must include a detailed section that discusses and addresses the status of any shoreline erosion processes, and must recommend if necessary any remedial action that must be carried out.

4 Mitigation Measures

If based on the monitoring, either during operation, or prior to final closure, there is any physical evidence of shoreline erosion, repairs should be carried out as per the details presented in Dwg. T-14. Modifications to this technique may be considered if reviewed and approved by a qualified registered Professional Engineer in Nunavut.

At final closure, i.e. as soon as the water level is lowered to 28.3 m, the entire flooded perimeter of the TCA shoreline should be inspected and in areas where visible erosion is seen to occur, shoreline erosion protection must be put in place.

5 Construction Materials

Construction fill material for shoreline erosion protection consists of run of quarry material, which will be produced on site from one of four local quarries. Complete geological, mineralogical and geochemical details on these quarry sites are documented in MHBL (2003), AMEC (2003), and SRK (2006). The grain size distribution envelopes for all the construction fill is presented in Dwg. G-05.

Other materials that will be used include geotextile. Complete details of this material are provided in the Technical Specifications (SRK 2007a); however, for completeness a brief summary is presented below:

- *Run of quarry material:* This material will be used to construct the outer shell of the dams, as well as armouring for the spillway and for the shoreline erosion protection. It will consist of run of quarry rock and have a maximum size of 500 mm. Fabrication of this material will be dependent on the condition of the rock and the blasting procedure at the quarry.
- *Geotextile:* Non-woven geotextile is used as a filter layer beneath the run of quarry fill for shoreline erosion protection. The geotextile acts as a filter layer should erosion occur.

6 Construction Methodology

Placement of the shoreline erosion protection should be done as follows:

- Stake out the areas that require shoreline erosion protection and clear the snow from those areas.
- Place the geotextile directly onto the tundra, taking care to allow at least a 0.5 m overlap between layers.
- End dump and spread the run of quarry material, such that the finished surface is about 0.5 m thick, and has a smooth top surface that mimics the natural ground topography. The upslope perimeter of the erosion protection material must be shaped such that no natural runoff would be ponded behind the erosion protection layer.

7 Routine Maintenance & Operational Procedures

Maintenance tasks associated with the shoreline erosion protection works are as follows:

- Every year immediately before freeze-up the entire TCA shoreline must be inspected and areas where shoreline erosion is detected must be marked such that those areas can be clad when the ground is frozen.
- At any time during the summer months, if areas are detected along the TCA shoreline that are undergoing active shoreline erosion, silt curtains must be deployed such that any suspended matter will be contained. Physical erosion protection works will be carried out in those areas as soon possible.
- The six monitoring transects must be inspected, surveyed and routine maintenance must be carried out on the instrumentation to ensure that the instruments are in working order.

- If at any point during the life of the TCA, the suspended sediment concentration in the TCA exceeds the MMER value of 15 mg/L at the point of discharge, a silt curtain will be installed around the discharge uptake. At such time, water quality monitoring will be done both upstream and downstream of the silt curtain to ensure the success and integrity of the curtain.

- If the TSS concentration downstream of the silt curtain (if required) exceeds the MMER value before the TCA reaches FSL, or would not meet CCME Guideline values at the designated monitoring point downstream of the waterfall in Doris Creek during discharge, no water will be discharged.

8 References

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Appendix H

SRK Technical Memorandum re: Addressing NWB Technical Review Comments

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Technical Memorandum

To:	Larry Connell	**Date:**	March 20, 2007
cc:	Project File	**From:**	Maritz Rykaart
Subject:	Additional Technical Responses to NWB Review Comments Regarding the October 2006 Tailings Containment Area Design Report (Supporting Document 1 of the MHBL Water Licence Application)	**Project #:**	1CM014.008.165

The NWB submitted a list of review comments and information requests to MHBL on December 27, 2006. SRK has attempted to address most of these comments in the March 2007 Revision to the Supporting Documents; however, some of the comments cannot be logically addressed in the reports, and therefore those comments are dealt with in this Technical Memorandum. In each case, the original NWB comment/question is repeated, followed by the formal SRK response.

17. Engineering Drawings **T-03**, **T-04**, and **T-06** – The NWB understands that the cut-off trench, as shown, is 2 m in depth. In the North dam drawings (**T-03** and **T-04**) a typical additional trench excavation is shown to remove peat materials. The EBA letter report dated February 28, 2005, **Re: Review of Alternative Dam Design, Hope Bay Doris North Project**, indicated that the depth of the cut-off trench for the North Dam should be increased from 2 to 3.5 m. This letter also suggested that an additional drilling program, consisting of at least 3 holes, be carried out to supplement the geotechnical data and reduce uncertainties in the stratigraphy of the North Dam. Does MHBL believe there is adequate data and analysis to confidently support the implementation of a 2 m cut-off trench under the North Dam? If trench depth is altered in the field during construction, what impact would this have on the installation and performance of the thermosyphons?

SRK Response: *The EBA letter specified "… increase the expected depth of the cutoff trench for the North Dam from 2 m to 3.5 m while retaining flexibility to adjust the depth locally...". EBA confirmed that this statement made specific reference to deepening the cutoff trench depth where the thick peat deposit was present, and not throughout the dam alignment.*

Although additional holes were not drilled at the North Dam, it is SRK's Professional opinion, supported by those of EBA, that the stratigraphic profile used in thermal/creep analyses was conservative. The intent of the deepened cutoff trench was to remove the peat below the core, not for thermal/creep deformation reasons.

Variable trench depth would not impact thermosyphon installation and performance, as a levelling course will be placed to ensure that there is a smooth surface on which to lay the thermosyhons.

20. The NWB understands that the creep/deformation analyses provide estimates of vertical and lateral deformation along the dam foundation. The NWB also understands that MHBL indicates that settlements due to consolidation of the marine clay and silt foundation soils are less of a concern than those predicted to result from creep and thaw. The NWB understands that the North Dam is expected to have larger differential settlement than the South Dam; however, the South Dam will have large

settlements. The NWB also understands that creep and thaw settlements are expected to be high along the upstream and downstream foundation sections of the dam, but low beneath the core. **Figure 24** of the TCA Design report provides predicted settlement over time under the crest of the core.

a. The NWB understands that **Figure 24** may be based only on creep deformation. The NWB requests additional information on the predicted total settlement including contributions from other mechanisms (such as thaw and consolidation)? Has this total settlement been estimated?

> ***SRK Response:*** *As demonstrated in the EBA (2006) report, beneath the dam crest, additional settlement due to thaw or consolidation is not anticipated. Thaw settlement is expected to occur beneath the dam shell upstream of the core; this is anticipated to be local, and because of the flat dam slope, it is not expected to induce additional deformation of the core. Settlement due to consolidation is anticipated to be negligible over the 25-year design life as the frozen saline silt and clay (especially beneath the dam core) are anticipated to have very low permeability, and the length of the drainage path is relatively long.*

b. The NWB understands that there may be variation in settlement across the dam, including where the thermosyphon's evaporator pipes will be located within the foundation of the dam. How will the estimated settlements affect the performance of the thermosyphon? Will the evaporator pipes be able to tolerate settlement/ deformation along the foundation without rupture or damage?

> ***SRK Response:*** *Settlements are predicted to be gradual and progressive. Settlement predictions have been based on the assumption that no measures will be taken after dam completion to repair or remediate the dam slopes. In reality, it is expected that the dams will be continually monitored and dam performance reviewed, with the expectation that remedial measures, such as repairing or flattening dam slopes, be adopted, should dam deformation be considered a problem.*

c. **Figure 24** shows only settlement under the crest of the core. How large are the settlements along the upstream and downstream sections of the dam?

> ***SRK Response:*** *From a creep-only perspective, the upstream and downstream toes of the dam are actually pushed upwards as the permafrost foundation beneath the dam core is being squeezed laterally (refer to Figure 25 in EBA (2006)). Minor thaw settlement is anticipated beneath the downstream toe, although more is anticipated beneath the upstream toe. At the North Dam, up to 6 m of permafrost thaw is predicted beneath the upstream toe over the 25-year design life; assuming 50% thaw strain, this translates to up to 3.0 m of thaw settlement at the toe. Given the flat design dam slopes, this isn't expected to initiate deep-seated slope failures that could damage the core, and if such settlements are observed, the slope should be repaired during routine maintenance. At the South Dam, up to 5 m of permafrost thaw is predicted beneath the upstream toe; again, assuming 50% thaw strain, this translates to 2.5 m thaw settlement; again, no deep-seated slope failure or damage to the core is anticipated.*

21. Does MHBL believe that the latest thermal modeling generally confirms that the thermosyphon will be required to keep the dam foundation frozen? If so,

a. Is there precedent for installation of thermosyphon in dam foundations with complex stratigraphy and expected foundation deformation/settlement on the same order as that for the North and South Dams?

> ***SRK Response:*** *There is no precedent for installation of thermosyphons with these foundation conditions and expected deformations.*

b. What is the expected behavior of the evaporator pipes subjected to high total and differential settlements?

<u>SRK Response:</u> *The expected behaviour is that the evaporator pipes (steel A53B Sch 40, 3/4" diameter) are believed to be sufficiently flexible to be able to support the settlements without rupturing. Differential settlements are anticipated to be gradual. This will have to be structurally verified by the specialist thermosyphon contractor.*

c. Does MHBL agree that the thermal modeling report indicates that the contribution of the thermosyphons in the first ten years of operation is critical to keeping the dam foundation frozen? From the Technical Specifications the NWB understands that that the thermosyphon will have a five year manufacturer's warranty against loss of heat transfer. How is this discrepancy resolved?

<u>SRK Response:</u> *Thermosyphons are NOT required to keep the permafrost foundation frozen; it is intended only to provide additional COOLING to the foundation, to minimize long-term creep deformations. As with any mechanical instrument, it is reasonable for the manufacturer to provide some time limit over which he can guarantee his product, especially one exposed to extreme arctic conditions as are these thermosyphons. Like many consumer items, thermosyphons have most often performed well beyond its warranty period; the most common problem has required adjustment of the charge pressure of the refrigerant fluid (CO_2), which can be easily done without replacing buried pipes.*

d. What kind of installation measures, if any, to accommodate total and differential settlements will need to be implemented to ensure that the thermosyphon will function as required, in view of the complex dam foundation at the site?

<u>SRK Response:</u> *There are no other installation measures that can accommodate the anticipated settlements, should they be sufficiently large to damage the thermosyphons. The recourse is to install sloped thermosyphons, if necessary, and/or to flatten the dam slope.*

e. Will the thermosyphons function in at least the first ten years? Does MHBL believe this is critical to keep the dam frozen?

<u>SRK Response:</u> *Thermosyphons are not required to keep the dam foundation frozen, but are included in dam design to minimize long-term creep-induced deformations. The actual thermal performance is more likely to be influenced by impoundment levels during the dam's design life. Analyses were carried out assuming that the water level reaches full supply level (33.5m) the second freshet following dam completion and remains at that level throughout the 25-year design life. As SRK's calculations show (Design Report, Section 6.3), it may take between 5½ and 9½ years following dam completion before the water level reaches full supply level; therefore, the thermal and creep deformation calculations beneath the upstream dam shell are considered very conservative.*

f. In the event that the installed thermosyphon stops functioning as required, what alternative options can be implemented? Does MHBL plan to install additional thermosyphons after the dam is constructed?

<u>SRK Response:</u> *Should the thermosyphons fail, the contingency is to install sloped thermosyphons and/or to flatten the dam slope, if deemed required from dam performance monitoring.*

g. Have thermal analyses been carried out to include short term application of the thermosyphon (i.e. thermosyphon that will only function in the first few years of the dam operation)? Does MHBL believe that this type of analysis may provide information on the critical time period at which the thermosyphon will no longer be required? Does MHBL believe this type of analysis should be completed and provided to Regulators? If not, why not?

SRK Response: *SRK does not believe that this type of analysis is warranted because SRK believes that should the thermosyphons fail, especially over the first ten years of the dam's design life, they should be repaired or sloped thermosyphons added. Secondly, this kind of analysis emphasizes a theoretical "what if" scenario, as if the need for ongoing thermosyphon cooling could be judged independently of performance monitoring. Performance monitoring will indicate whether or not the dam is behaving as predicted, and whether or not the assumptions used in dam design were reasonable. Should dam performance monitoring suggest dam distress, other remedial measures, such as flattening dam slopes, can be adopted to alleviate the problem. Finally, the thermal regime is more likely to be governed by the actual water level history (for which the thermal analyses incorporated conservative assumptions) than thermosyphon operation.*

22. The NWB understands that the finite element thermal modeling analyses that were carried out for the North and South Dams were completed using two different modeling softwares. The initial analyses completed for the FEIS utilized commercially available SVHEAT software, while those carried out for the Water License Application utilized a proprietary EBA's GEOTHERM modeling software. With the GEOTHERM modeling, has MHBL's latest data been used? Are the model results from the two different software packages comparable and complementary to the design of the frozen dam foundation? The NWB requests a detailed discussion outlining this comparison and detailed discussion outlining any new finding discovered through GEOTHERM modeling.

SRK Response: *Thermal modeling with EBA's GEOTHERM software considered measured ground temperature data up to September 2005. Although more recent temperature measurements were available (to June 2006), climatic data for 2006 were not available at the time of modeling. The borehole logs from the two 2006 geotechnical boreholes at the South Dam were consistent with the stratigraphic profile used in the thermal model.*

The latest results from EBA's GEOTHERM modeling were not compared to SRK's earlier modeling using the SVHEAT software, the reason being that there were several different assumptions used in modeling, including:

- *Modeled permafrost foundation stratigraphic profile;*
- *Dam geometry;*
- *Temperature boundary to simulate heating from water impoundment;*
- *Climatic data used in thermal analyses; and*
- *Different thermosyphon configuration and assumptions used to calculate thermosyphon heat extraction.*

The main difference between EBA's GEOTHERM model and the commercial SVHEAT software is in the application of the ground-surface energy balance. The GEOTHERM model (see Hwang, 1976) calculates the surface energy balance based on climatic data (air temperature, snow cover and thermal properties, wind speed, and solar radiation) as well as ground surface properties (surface albedo, emissivity, and evapotranspiration). SRK's 2005 Preliminary Design Report used SVHEAT (v. 3.09), which is based on an N-factor approach, in which the ground surface properties are lumped into a number by which the air temperature is multiplied to compute the ground surface temperature. The N-factor approach, while still commonly used in thermal design calculations, is recognized as more empirical and less rigorous than the surface energy balance approach that GEOTHERM uses. It is understood that the latest version of SVHEAT (v. 4.0) uses a climate-data boundary, although this has not been examined in detail.