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Dam Safety Assessment

Mount Nansen Tailings Facility near Carmacks, YT

Project No: 0201-00-14618

May 2002

DAM SAFETY ASSESSMENT
MOUNT NANSEN TAILINGS FACILITY
NEAR CARMACKS, YUKON

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Submitted To:

Water Resources Division
Department of Indian Affairs & Northern Development – Yukon Region
Whitehorse, Yukon

Project No.: 0201-00-14618

May 2002

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EXECUTIVE SUMMARY

EBA Engineering Consultants Ltd. was retained by the Yukon Region of Indian and Northern Affairs Canada to complete a dam safety assessment of the abandoned tailings impoundment at the B.Y.G. Natural Resources Inc. mine site, west of Carmacks, Yukon. The dam safety assessment was to consider the physical condition and stability of the earthfill tailings dam, and the adequacy of both the spillway and the seepage control dyke associated with the tailings dam. The evaluation of these items was to consider both the current state of these facility elements as well as a projected condition five years in the future.

During the course of this assessment, site visits, site characterization studies, and reviews of available geotechnical data were completed. Detailed reporting of these activities is included in a Summary Data report that is presented under separate cover. Moreover, specific safety deficiencies and inadequate performance of the spillway and the seepage control dyke were identified at an early stage in the assessment process and upgrading and repair measures were enacted in the fall of 2000 and the summer of 2001 to improve these elements. Reporting on the upgrading and repair of these elements is presented in a companion report prepared to specifically document those activities.

The overall results of the dam safety assessment were that a number of significant safety deficiencies were identified for the earthfill tailings dam, the seepage control dyke, and for the spillway system. Moreover, in the five-year window considered for this assessment, the condition of the dam and spillway are not expected to improve unless significant remedial actions are undertaken. The deficiencies were determined by the comparison of observed and/or predicted geotechnical, hydrotechnical, and/or environmental performance of these impoundment elements, under both normal and extreme event conditions, in comparison to the performance guidelines of the Canadian Dam Association (1999) and environmental criteria associated with expired Water Use Licence QZ94-004.

Specific safety deficiencies identified for the dam structure in this assessment include inadequate seismic stability, a lower that acceptable static stability, excessive seepage rates, and higher than predicted permafrost degradation.

Safety deficiencies identified for the spillway system include inadequate capacity even for normal operations due to unacceptable erosion potential. In terms of predicted capacity, the spillway could suffer scour damage with flows as low as that predicted for the 20 year design storm. The spillway would not survive the 200 year storm flows and the integrity of the dam could be comprised should this occur.

The capacity of the spillway is based on its condition following upgrading and repairs completed in 2000 and in 2001. Even at flows below the 20 year return level, localized failures due to permafrost degradation are possible. The spillway is considered to be an unreliable element of the impoundment. Closure and decommissioning of the facility in the next five years would require significant upgrading of the spillway to handle floods up to the Probable Maximum Flood.

The seepage control dyke was also found to be deficient in terms of its performance. This structure was allowing excessive contaminated seepage to pass beyond the facility. Remedial measures were taken to reconstruct this structure in the fall of 2000 and evidence currently supports that it is functioning appropriately.

Over the next five years, the condition of the tailings impoundment and its elements are not expected to alter significantly, although seepage rates may continue to increase in association with greater permafrost thaw within the north abutment, and more localized failures of the spillway are possible. The greatest short term risk to the facility would be the occurrence of a significant hydrologic (flood) event. Continued monitoring of the seepage levels is critical as is operating the facility with as low a pond level as possible. It is expected that significant expenditures would be necessary in order to prepare the facility for abandonment.

1.0 INTRODUCTION

1.1 Project Description

EBA Engineering Consultants Ltd. (EBA) was retained by Yukon Region of Indian and Northern Affairs Canada (Water Resources) to complete a dam safety assessment of the tailings impoundment facility at the mine site formerly operated by B.Y.G. Natural Resources Inc. (Mount Nansen Mine Site), 60 km west of Carmacks, Yukon. The dam safety assessment was to consider the condition and physical stability of the earthfill tailings dam, and the adequacy of the dam's spillway and of the seepage control dyke downstream of the tailings dam. As requested by Water Resources the evaluation of these items was to consider both the current state of these facility elements, as well as a projected condition in 2005 (five years in the future). A longer time frame, such as would be required for a closure evaluation of the facility, was not within the scope of work of this assessment.

During the course of this assessment, site visits, site characterization studies, and reviews of available geotechnical instrumentation data were completed. Details of these activities are reported in a Summary Data Report that is presented under separate cover. Moreover, specific deficiencies in the adequacy of the spillway and of the seepage control dyke were identified at an early stage in the assessment process and upgrading measures were enacted in the fall of 2000 and the summer of 2001 to improve these elements. Reporting on the upgrading of these elements is presented in a companion report (Mount Nansen Seepage Dyke and Spillway Upgrading Construction Report, May 2002) prepared to specifically document those activities.

This dam safety assessment has been completed in accordance with generally accepted geotechnical practice and engineering judgement has been used in the development of conclusions and recommendations. This assessment encompasses most but not all of the elements of a more comprehensive "Dam Safety Review" as defined by the Canadian Dam Association in the 1999 Dam Safety Guidelines. To meet the requirements of a "Dam Safety Review", additional effort would be necessary to review and evaluate:

1. The assigned classification of the dam.
2. The operational procedures utilized at the dam site.
3. The maintenance procedures utilized at the site.

4. The emergency preparedness plans and procedures developed for the site.

For additional information regarding the use of this report, please refer to the General Conditions, included as Appendix A, that form a part of this report.

Authorization to proceed with this assessment was provided by Mr. Bud McAlpine, Water Rights Administrator, Yukon Region, on July 28th, 2000.

1.2 Scope of work

The scope of work for this assessment was outlined in EBA's proposal of July 2000, submitted to Water Resources. As outlined in that document, the proposal was prepared to address four areas of concern that had been identified during the completion of a Project Data Review report by Klohn Crippen Consultants Ltd. (Klohn) and EBA in the winter of 1999/2000, and other issues identified in subsequent discussions with Mr. Bud McAlpine of Water Resources. The four areas of concern were:

1. The spillway channel.
2. The area of subsidence on the southern edge of the downstream slope of the tailings dam.
3. The overall stability of the tailings dam in terms of the static and seismic loading cases.
4. The seepage performance of the seepage control dyke.

For Item #1, the scope of work included surveying the geometry and condition of the spillway, locating the permafrost depth underneath the spillway, and reviewing the hydrology of the impoundment and its predicted flood flows. In addition, the capacity of the spillway was to be determined and upgrading recommendations were to be provided.

For the south abutment subsidence (Item #2), the scope of work included surveying of existing monitoring pins in the area, installing additional pins for future surveys, and physically probing the area to identify the underlying fill materials and the amount of permafrost thaw.

For the overall dam stability (Item #3), the work scope included determining the seismic hazard, and completing a cone penetration test (CPT) program to characterize the embankment fill and the thawed foundation soils. Using the seismic hazard criteria and the soil characteristics determined from the CPT testing, the liquefaction potential of fill and native soils was to be determined.

A revised seepage assessment of the structure using soil parameters provided by the CPT program (if applicable), the revised dam geometry (including full pond head), and existing seepage pump back measurements would also be completed. Based on the revised model, a parametric study of soil parameters would be conducted to match measured phreatic levels. Thermistor data would be used to extrapolate the thawing underneath the dam to 2005.

Using the results of the CPT program, the liquefaction assessment, the known phreatic levels and the known and predicted thaws depths underneath the dam, static and dynamic stability analyses would be completed for the current dam state and the state anticipated in 2005.

For the final item (Item #4), EBA's scope consisted of conducting a testpitting program downstream of the seepage dyke to determine current soil, groundwater, and permafrost conditions. Where possible, seepage paths through and/or around the dyke would be identified. Based on the results of the testpitting, EBA would evaluate possible conceptual designs to improve or replace the dyke with the objective of reducing seepage losses around and through the structure.

During the course of the project, the overall scope of work was modified through the input of Water Resources, EBA, and B.K. Hydrology Service (EBA's hydrotechnical subconsultant). The modifications included limiting the spillway assessment to consideration of the 200 year flood event instead of the Probable Maximum Flood.

This change was based on the input of the hydrotechnical consultant who indicated that the spillway would require significant widening to accommodate the PMF event. Moreover, it was EBA's assumption that such an operation would require permafrost excavation. Discussions with Water Resources revealed that consideration of using the existing spillway for the closure spillway was premature at this time and it was agreed to limit the spillway assessment to the 200 year return level.

In addition, the scope of work for both the spillway and seepage dyke aspects of the project was changed to include detailed design and construction of upgrading modifications in the fall of 2000 and the summer of 2001.

1.3 Available Information

During the course of this assessment, EBA and/or its subconsultant B.K. Hydrology Services (BKH) had access to numerous documents and reports relating to the design, licensing, construction, and operation of the tailings facility. These documents included:

- Design reports prepared by Klohn Crippen Consultants Ltd. (Klohn)
- Water Use application documents prepared by B.Y.G. Natural Resources Inc.
- Water Licence QZ94-004 issued by the Yukon Territorial Water Board.
- Tailings Dam Construction report prepared by Klohn.
- Site Visit reports prepared by Geo-Engineering.
- Instrumentation Installation Report and internal memoranda prepared by EBA.
- Spillway upgrading reports prepared by Vista Engineering, and
- Project Data Review reports prepared by Klohn and EBA.

Full references to these documents as well as to technical sources consulted in the completion of this assessment are presented in the References following this report.

2.0 MOUNT NANSEN TAILINGS FACILITY

2.1 Facility Description

As shown in Figure 1, the Mount Nansen tailings facility consists of a tailings impoundment with a water reclaim/pump causeway, a tailings dam, diversion channels, a reclaim pond formed by a cross valley seepage control dyke located below the main dam, and an emergency spillway. The impoundment area is roughly 7 ha in area with rough dimensions of 250 m (north-south) by 280 m (east-west). The volume of tailings within the impoundment is estimated to be in the order of 283 500 m^3 including both subaqueous and beach tailings[1]. Surface water stored within the impoundment varies seasonally but typical ranges from 40 000 m^3 to 60 000 m^3 occupying a surface area of roughly 4.5 ha.

[1] Based on surveys completed by Yukon Engineering Services Ltd. in October 2001.

A topographic site plan of the facility including the impoundment, tailings dam, and seepage dyke (but excluding the spillway) is presented as Figure 2. Figure 2 is based on as-built topographic surveys of the tailings dam, tailings impoundment, and seepage control dyke including the changes resulting from the upgrading work completed in the autumn of 2000.

As shown on Figure 2, the Mount Nansen tailings impoundment is formed by a 270 m long dam which runs (north-south) across the Dome Creek Valley. The dam, which is located roughly 1.5 km downstream from the Mount Nansen mill site, is an earthfill structure consisting of a main embankment of about 160 m in length with a low abutment dyke extending another 110 m across a terrace feature on the north abutment. The main embankment section of the dam has a maximum height of 21.5 m, whereas the northern dyke or terrace section has a maximum height of about 6 m with a typical height of about 4 m.

To accommodate the dam and the impoundment, Dome Creek was rerouted around the impoundment into a diversion channel that runs along the north side of the valley. Runoff control ditches running along the western perimeter of the impoundment join into the diversion near the northwest corner of the impoundment. No diversion channels have been constructed on the south side of the impoundment.

The diversion channel runs at a gentle slope until it just passes the dam centreline. At that point the channel enters into a steep spillway channel that runs from an elevation of 1152 m down to 1121 m, over a distance of 315 m. At the end of the spillway the flow exits back into the original channel of Dome Creek.

To account for possible overflow from the impoundment, an emergency spillway channel runs from the northeast corner of the impoundment into the diversion spillway channel joining the spillway about 60 m downstream from the dam centre line. This emergency channel has an invert of 1150.6 m.

Control of water levels within the impoundment is accomplished primarily by diverting the flow of Dome Creek around the impoundment and by intercepting surface runoff from the western perimeter. Flow that enters the impoundment can only be removed by seepage through the dam and its abutments, by overflow through the emergency spillway, or by pumping out of impoundment up to the water treatment plant located at the mill.

To facilitate pumping from the impoundment, a causeway leading to a pump building is located on the north side of the impoundment roughly 70 m upstream of the dam.

A final element of the tailings facility is a downstream reclaim pond. The pond is formed by a seepage control dyke that is located across the former channel of Dome Creek, 40 m downstream from the toe of the main span of the tailings dam. This dyke structure was rebuilt during autumn 2000 and now consists of a 50 m long, 4 m high earthfill dyke that incorporates a PVC liner keyed roughly 2 m into the permafrost. The key trench was designed to refreeze through the use of horizontal thermosyphons. Details of the design and construction of the rebuilt seepage dyke are documented in a report entitled "Mount Nansen Seepage Dyke Design and Spillway Upgrading Construction Report" dated May 2002. Much of the seepage passing through the dam and its abutments is captured in the reclaim pond and pumped back up into the main impoundment. Some seepage is known to bypass the pond by flowing through unfrozen zones in the north terrace and into Dome Creek downstream of the seepage control pond. Monitoring of the water quality of Dome Creek is conducted by Water Resources at points downstream of the reclaim pond.

2.2 Facility Design & Construction

The tailings dam was designed by Klohn Crippen Consultants Ltd. (Klohn) in 1994 and 1995, and constructed by Ketza Construction Corporation (Ketza) for B.Y.G. Natural Resources Inc. (BYG) in 1996. The impoundment was sized to provide storage for up to 240 000 m^3 of tailings that would be produced from the mining of the Brown McDade deposit over a three year period. The maximum tailings elevation within the impoundment was to be 1149.2 m at end of the 3-year mine plan.

The impoundment was also designed to allow for the storage of water required for milling operations. The mine operator's originally submitted water balance for the site called for roughly 30 000 m^3 to 40 000 m^3 of water to be held in the impoundment at any time. At the end of the mine plan this would result in a water elevation of 1149.7 m giving on average about 0.5 m of water over the deposited tailings. As a contingency measure to handle large flood events during the 3-year mine plan, the emergency overflow spillway was to have an invert elevation of 1149.7 m, thereby spilling any water in excess of Elev. 1149.7 m.

The geometric design of the dam included a crest width of 6 m at an elevation of 1151.5 m, an upstream slope of 2.5:1 (H:V), and a downstream slope of 3.5:1 (H:V). A 4.0 m thick toe berm was included on the upstream side. The toe berm intercepted the upstream slope at an elevation of 1141.0 m and had a maximum width of 20 m. The upstream slope of the toe berm was selected to be 2:1 (H:V).

The dam was built using roughly 100 000 m^3 of fine to medium grained fluvial sand borrowed from sources within the impoundment and from the spillway excavation. The dam foundation consists of permafrost soils that, excluding an upper organic rich zone, are also a sand material. A geosynthetic clay liner was incorporated into the upstream slope to provide secondary seepage control. The primary seepage control was to be provided by the tailings deposited within the impoundment. A minimum tailings beach width of 50 m was called for in the design. Exposed slopes and the crest of the dam were protected from erosion by placing a layer of gravel and cobble sized angular rock garments produced from weathered bedrock borrowed from slopes above the north side of the dam.

Pertinent design criteria and as-built information for the dam, the seepage control dyke, and the spillway are presented in Table 1.

Construction of the facility was completed between May 1996 and September 1996. The construction process was monitored by Klohn and fully documented in a Construction Report that is included in the references to this assessment. Significant observations from the construction include that the foundation stripping was not completed to the extent proposed in the design and that compaction of sand fill was not well documented and could be suspect in some areas.

The stripping issue indicates that organic materials are likely present underneath some areas of the dam. The compaction issue suggested that zones of loose sand could be present within the dam fill. Another observation from the construction period was that seepage through the north dam abutment was observed as early as August 1996 at which time there was impoundment due to runoff during construction.

Table 1: Mount Nansen Tailings Facility Design Criteria and As-built Information[1]

Component	General	Details
Main Dam (Dam 1)	Seismic Review at Closure; 1 L/sec seepage after freezeback; can accommodate 0.6 m of crest settlement and complete thawing of foundation.	Sand fill, 2.5:1 U/S, 3.5:1 D/S; 6 m crest width, crest 1151.5 m, 20 m U/S berm at 1141 m, U/S GCL, 18.5 m maximum height, minimum 300 mm of gravel surfacing on U/S and D/S slopes and 500 mm on Crest, 50 m wide tailings beach.
Downstream Toe Berm (Dam 1)	Emergency Toe Berm constructed in July/August 1997 to control piping activity at toe of dam. Field designed by EBA and constructed in stages by mine operator without full geotechnical supervision.	4:1 slope with 7 m wide crest at 1141 m. Layered construction including: sand bedding (0.1 to 0.8 m), non-woven geotextile, clean placer gravel (0.3 m to 0.6 m), and rockfill and/or ripped residual soil shell for remainder. Cut off trench and drain installed at toe of berm.
Original Seepage Control Dam (Dam 2)	Designed to intercept seepage from Dam 1 with anchored GCL in permanently frozen ground.	Sand fill, design 3.5:1 U/S & D/S, 5 m crest width, 1130 m crest, U/S GCL; 300 mm gravel surfacing ; Actual 3.3:1 U/S, 3.1:1 D/S, 7 m crest width, 300 mm gravel surfacing U/S, 1 m average thickness gravel surfacing D/S.
Rebuilt Seepage Control Dyke (Dam 2)	Original seepage control dyke replaced with new construction in November 2000. New dyke includes PVC arctic grade liner keyed into permafrost trench. Sand bentonite seal within trench and thermosyphon system to control refreeze permafrost in key trench.	Sand fill with surfacing gravel cover; 2.5:1 U/S and D/S; crest elevation of 1131 m; seepage control using PVC liner keyed into permafrost in a sand bentonite filled trench. Refreezing of key trench promoted by horizontal thermosyphons. Maximum height 4 m.
Instrumentation for Dams 1 and 2	Instrumentation for Dam 1 and 2 was installed in 1995 and 1996. Most of this original instrumentation was destroyed and/or malfunctioned by 1997. Additional instrumentation was installed for Dam 1 in 1998 and 1999. New instrumentation for Dam 2 was installed in 2001.	Dam 1: nine thermistor suites with a total 53 measurement beads; five piezometers suites with a total of 15 pneumatic piezometers; 43 settlement/displacement points (including 9 well casings); and one standpipe. Dam 2: three thermistor suites each with 16 thermistor beads and three 50 mm sampling wells.
Permanent Diversion	20 year peak flow at 6.4 m^3/s and 1.4 m flow depth.	3 m wide invert, 1.4 m depth, 0.1% grade, 300 mm grade liner, upstream elevation 1151.5 m, horizontal distance 360 m, crosses Dam 1 centreline at 1151.0 m.
Emergency Spillway	200 year peak flow, 3.7 m^3/s for 0.1% gradient section (flow depth 0.8 m) and 6.4 m^3/s for 10% grade section (flow depth 0.4 m)	1149.7 m intake, 5 m invert, 1.1 m depth at 0.1% from -55 m to +50 m D/S of Dam 1 centreline, 0.8 m depth at 10% +50 to +340 m D/S of Dam 1 centreline; erosion protection: 300 mm of surfacing gravel within 0.1% section; filter cloth and 750 mm surfacing gravel or placer gravel within 10% section.
Reservoir	240 000 m^3 original design capacity at Elev 1149.7 m prior to reservoir enlargement by excavation for borrow, actual maximum tailings elevation 1148.7 m.	Trees and shrubs mostly removed, haul roads left inplace, access road on north side and causeway to reclaim pump, upstream borrow areas increased capacity.
Water Licence	QZ94-004, Indian and Northern Affairs Canada	Licence expired December 31, 2001.
Design Document	Tailings Impoundment Final Design Report, Klohn-Crippen Consultants Ltd., August 1995	

[1] Table updated and amended from Klohn-Crippen Construction Report , December 1996

2.3 Facility Operation

The operational history of the Mount Nansen tailings facility can be considered in two phases: first the active mining or operational phase, and then the current shutdown phase. Key events and observations from these two phases are presented in the next two sections of this report.

2.3.1 Operational Phase

During the operational phase, the impoundment was actively storing water starting in August 1996. By October 1996, the mill was in operation and tailings and process water were being discharged into the impoundment. The discharge of tailings and process water continued through to the spring of 1999 except for a brief shutdown of about 3.5 months in the winter of 1997/1998. Mill discharges ceased in the spring of 1999 due to the insolvency of the mine operator.

During the roughly 2.5 years of active mining, the tailings pond received an estimated 283 500 m^3 of tailings as well as an unknown amount of process water. Some process water was reclaimed from the impoundment during the operational period, particularly during the winter months of 1998 and 1999 when the fresh water supply line to the mill was frozen. In addition, some water was treated and released from the impoundment. The treatment and release of pond water occurred sporadically starting in the fall of 1997 and continuing until the milling operations ceased in February 1999.

During the operational phase of the mine, the water level within the tailings impoundment greatly exceeded the levels predicted based on the design water balance. The water level reached an elevation of 1149.0 m by the eighth month of milling (June 1997). This level was not predicted to occur until the 31st month of milling. Based on the predicted tailings production rate, there was at least 170 000 m^3 of water stored within the impoundment at that time.

Between June 1997 and the shutdown of the mill in February 1999, there is no record of the pond level dropping below 1149.06 m. The average level was on the order of 1149.6 m. Nevertheless, EBA is not aware of any surface water spills that occurred out of the impoundment during this phase of the mine life.

Several modifications were made to the dam, spillway, and diversion channels in 1997 and 1998. In the spring of 1997, a cutoff ditch was constructed along the western perimeter of the impoundment. This ditch joined into the existing Dome Creek diversion near the northwest corner of the impoundment.

In July 1997, seepage through the main embankment and the north abutment was leading to sand boils at the dam toe and slumping of the toe of the north abutment slope. Seepage through the dam was daylighting as high as Elev.. 1135 m on the downstream face of the dam. An emergency toe berm was placed over the lower portion of the dam (up to Elev. 1140 m) at this time to improve the downstream slope stability and to reduce the potential for a piping failure. Some of the existing dam instrumentation was destroyed during the berm construction.

Another modification to the impoundment completed in 1997 was the reconstruction of the spillway channel. The spillway portion of the Dome Creek diversion experienced significant settlement and erosion during the spring of 1997. Flow had to be diverted out of the spillway back into a temporary diversion channel used during the 1996 construction season to prevent further damage. By June 1997 further damage developed when flow from the higher elevation temporary diversion began to pipe through to the spillway section leading to more erosion. The spillway section was rebuilt in the fall of 1997 under the direction of BYG.

In the spring of 1998, new piezometers and thermistors were installed to replace the non-functional and damaged instrumentation. The Dome Creek diversion channel was also upgraded and repaired to correct for infilling with eroded sand from the western cutoff ditches.

2.3.2 Shutdown Phase

Following the shutdown of the mine in February 1999, the site entered into the current shutdown phase. During this phase, management of the tailings impoundment was initially conducted by the Mine Receiver; however, the Receiver discontinued management of the site in the summer of 1999. Due to environmental concerns, the Federal Department of Indian and Northern Affairs (DIAND), which is the Responsible Authority (RA) for this undertaking, assumed management of the facility after the Receiver suspended site operations in the summer of 1999.

During the shutdown phase (spring 1999 to present), the operation of the impoundment has consisted of the seasonal treatment and release of surface water, and the continuous collection and pump back into the impoundment of seepage collected by the seepage control dyke. The seasonal treatment and release of surface water from the main impoundment is conducted by pumping surface water from the tailings impoundment up to the water treatment plant at the former mill site.

Treated water is ultimately released into Dome Creek below the mill site and flows through the Dome Creek diversion into the spillway and beyond the facility. During the shutdown phase the pond water level has ranged from 1148.6 m to 1150.1 m, with an average recorded level of 1149.6 m.

In addition to the environmental site maintenance, the Responsible Authority (RA) has commissioned several evaluations of the impoundment and has managed two significant repair/upgrading programs associated with the seepage dyke and emergency spillway components of the facility. The evaluations commissioned by the RA, included a bathometric survey of the tailings pond, a preliminary dam safety assessment, conceptual site decommissioning studies, tailings and water quality studies, and the dam safety assessment reported herein.

The bathometric survey was completed on August 4, 1999 when the impoundment water level was at Elev. 1150.0 m. The survey showed a maximum water depth of 2.6 m and estimated the stored water volume at 53 000 m^3. Based on the reservoir storage curve provided by Klohn in 1995, this would suggest that at least 253 000 m^3 of subaqueous tailings are present within the impoundment. An important observation from the bathometry was that the deepest portion of the pond, hence the least thickness of tailings, was located upstream of the north abutment slope of the dam.

For the preliminary dam safety assessment, EBA and Klohn were jointly commissioned to complete this work in the fall of 1999. This assessment was based on a site visit in September 1999, followed by the evaluation of all available construction and instrumentation data. Klohn and EBA completed this review in January 2000. The results of the preliminary study were that there was no evidence of imminent failure of the dam but the long term performance of the structure was still unknown. The foundation soils were clearly thawing, organics and loose sand were suspected to exist within the thawed foundation soils and possibly within the dam fill, and seepage was well above design levels. The liquefaction potential of thawed foundation soil and of any poorly compacted zones of the dam was not determined.

EBA is not aware of the status or findings of the other studies (decommissioning, tailings and water quality), and the findings of the current dam safety assessment are presented within this report.

The two upgrading/repair programs directed by the RA during the shutdown phase have included the complete replacement of the seepage control dyke in autumn 2000, and upgrading and repairs to the emergency spillway in the autumn 2000 and summer 2001. As already indicated, details regarding these programs are reported separately.

2.4 Consequence Classification and Extreme Event Criteria

In order to complete the safety assessment of the Mount Nansen tailings impoundment, it was necessary for a consequence classification to be selected. The consequence classification represents the potential incremental impacts (in terms of lose of life and/or economic and environmental losses) associated with the failure of a dam or of its various components. For a given classification, extreme events, such as floods and earthquakes, that could trigger a failure are selected based the Dam Safety Guidelines of the Canadian Dam Association (1999, CDA).

A formal assessment of the hazard classification of the Mount Nansen facility was outside of the scope of work of EBA for this project. Therefore, EBA has adopted the classification as assigned by the Responsible Authority (RA) to this facility and its elements. EBA understands that the RA considers the facility and its spillway to be high to very high consequence structures due to the potential for incremental environmental damage that could result from a failure. This classification is an upgrading of the classification level considered by the Designers in 1995. In keeping with this new classification, the extreme event criterion for this structure includes the Probable Maximum Flood (PMF) and the Maximum Credible Earthquake (MCE).

3.0 FIELD EVALUATIONS

3.1 Site Visits

As part of the dam safety assessment, comprehensive site visits were completed by Mr. Cord Hamilton, P.Eng. of EBA, in August 2000 and July 2001, and by Mr. Bernie Kallenbach, P.Eng. of BKH in September 2000 and October 2001.

Mr. Hamilton was also resident at the site for extended periods during the repair and upgrading programs completed in October 2000, and August 2001.

Over the period encompassed by this study (August 2000 to October 2001) the Responsible Authority, represented by Mr. Bud McAlpine, has also completed regular site visits to observe the condition of the facility and to direct ongoing site maintenance activities.

3.2 Field Programs

Specific field programs completed over the course of this assessment have included a testpitting and probe hole drilling program in August 2000, and two Cone Penetration Test (CPT) programs (one in August 2000 and one in September 2000). In addition, topographic surveys of the tailings dam, its monitoring pins, tailings beaches, the reclaim pond and seepage control dyke, and the spillway were completed in August 2000, November 2000, and/or September 2001.

In addition to these data acquisition programs, necessary reconstruction and upgrading programs were completed in October 2000 and August 2001. The October 2000 program was undertaken to upgrade the emergency spillway and to replace the seepage control dyke. The August 2001 program was undertaken to repair a failed section of the emergency spillway, as well as to continue the general upgrading of the spillway that was only partially completed in October 2000.

Figure 3 shows the locations of all probe holes, testpits, CPT test locations, monitoring pins, and instrumentation locations on or near the tailings dam and the reconstructed seepage dyke. The locations of probe holes and testpits within the spillway are shown in Figure 4. The topographic details on Figure 4 date from the August 2000 survey that does not include any changes resulting from the autumn 2000 and the summer 2001 activities at the spillway. The Responsible Authority does not currently plan an as-built survey of the spillway, as further work is likely to be required in ensuing years.

Full reporting of all the site characterization studies is presented in EBA's Data Summary report [1]. Reporting of the remediation works for the seepage dyke and emergency spillway is presented in EBA's Seepage Dyke and Spillway Upgrading Construction Report [2].

4.0 SEEPAGE CONTROL DYKE ASSESSMENT

During the initial stages of this assessment, it was determined by the RA that the environmental performance of the seepage control dyke was not acceptable. This conclusion was based on the amount of observed seepage passing through and around the seepage dyke. During winter months, evidence of seepage included heaving of the former creek channel downstream of the dyke. The heaving was caused by massive ground ice inclusions ("pingos") fed by the escaped seepage. Testing by the RA confirmed that unacceptable levels of contaminants were present in the escaped seepage.

As a result of this conclusion, Water Resources directed EBA to develop a remediation plan to upgrade the seepage control dyke. Once reviewed, the remediation plan was fast tracked and a new seepage control dyke was designed and constructed autumn 2000. As shown in Figure 5, the remediation plan included the incorporation a remnant of the existing dyke, placement of a PVC liner downstream of this remnant, and building up a new higher structure. The liner was tied into a key trench that incorporated a sand-bentonite barrier. The key trench also incorporated Thermosyphon loops that are designed to freeze the key trench fill to prevent further permafrost thaw and to decrease seepage that could pass through the trench.

Further details on the design and construction of the new seepage dyke are provided in the Construction Report [2]. The as-built topographic survey of the seepage control dyke is shown on Figure 2. Since its construction in autumn 2000, it is understood that seepage loses from the reclaim pond have significantly reduced and that the level of contaminants present in the seepage is within acceptable limits. Based on this the current performance of the reclaim pond and seepage control dyke is believed to be acceptable.

5.0 SPILLWAY ASSESSMENT

The assessment of the hydraulic elements of the Mount Nansen Tailings Impoundment (including the diversion channel, the diversion spillway, and the emergency spillway) was undertaken by Mr. Bernie Kallenbach, P.Eng. of BKH. The assessment was completed in two phases – an initial assessment in September 2000 and a revised assessment in October 2001.

The initial assessment included a full review of the Designer's hydrology for the impoundment complete with recommendations regarding design flows up to the 200 year return storm. Also included in the initial assessment was a condition survey that included recommendations to upgrade the spillway system in order to safely pass the predicted 200 year storm flows.

The revised assessment of the spillway was completed in October 2001 following implementations of the recommendations of the initial assessment in the fall of 2000 and the summer of 2001.

As noted in Section 2.0, these assessments were based upon a revised work scope that limited the assessment to include the 200-year storm event but not the PMF event. This modification was based on the assumed timeline of 5 years in which a decision would be made to either upgrade the spillway to handle the PMF event or to decommission the impoundment.

The initial and revised spillway assessment reports completed by Mr. Kallenbach are included in Appendix B of this report. The hydrology review, channel capacity and condition, proposed spillway modifications, and final spillway capacity and conditions are summarized in Sections 5.1, 5.2, 5.3, and 5.4 of this report.

5.1 Hydrology Review

A review of the original flows forecast by Klohn was completed. BKH concluded that the flood flows predicted by Klohn were excessive. The diversion channel flows forecast by BKH included a 20 year storm event leading to an instantaneous peak flow of 1.2 m^3/s and a 200 year storm event leading to an instantaneous peak flow of 3.0 m^3/s. These values are considerably less than those predicted by Klohn for the diversion.

Another significant difference between the Klohn forecasts and those completed by BKH was that the Klohn forecast suggested that the diversion channel would breach during the 200 year event thereby routing the 200 year event through the reservoir. The lower 200 year flow predicted by BKH might be handled by the diversion channel without breaching.

5.2 Channel Flow Capacity and Condition

Based on the flows forecast by BKH and on the observed condition of the diversion channel, diversion spillway, and the emergency spillway in September 2000, it was concluded that the diversion channel and emergency spillway would be adequate to handle the 200 year storm event, but the diversion spillway section would be inadequate to handle even the 20 year storm event without significant damage. Details regarding the individual channel sections are presented in Sections 5.2.1 to 5.2.3.

5.2.1 Diversion Channel

The conclusion that the diversion channel could pass the 200 year event was based on the observed channel geometry and the assumption that a 300 mm thick gravel lining was still in place within the channel. The presence of the original gravel lining was not confirmed due to the presence of silt and sand sediment within the channel. The silt and sand within the diversion channel originates within the cutoff ditches along the western perimeter of the impoundment. Removal of this sediment load has been completed on at least four occasions and is an ongoing concern requiring at least annual attention.

In 1998, the mine operator raised the elevation of the downstream containment berm for the diversion channel in order to provide the greater freeboard level required to account for the annual sediment load being deposited within the channel. There is a reasonable concern that continued excavation of sediment from the diversion channel may remove some or all of the original gravel lining material. Therefore, it is possible that the diversion channel could fail during the 200 year event. Should this occur, the 200 year flow would then be routed into the reservoir and would lead to overflow of the reservoir through the emergency spillway channel that leads from the impoundment into the main spillway.

5.2.2 Emergency Spillway

The emergency spillway section of the impoundment was intend to serve as an outlet for flood flows that resulted from breaches of the diversion channel into the tailings impoundment. BKH considered the scenario of the 200 year flows being routed through the reservoir and out through the existing emergency spillway. It was concluded that the emergency spillway geometry and construction would be adequate to handle the resulting tranquil flows that would pass through the emergency spillway section and out into the diversion spillway.

BKH noted that the emergency spillway flows resulting from a breach of the diversion channel during the 200 year storm event would be greatly attenuated by passing through the reservoir. Therefore, the actual flow would be less than the 3.0 m^3/s flow predicted in the diversion channel if it does not fail by breaching into the impoundment.

5.2.3 Diversion Spillway

The conclusion that the spillway section of the diversion channel was inadequate was based on the scour potential as opposed to the geometric capacity of the spillway channel. In fact the geometry of the channel was projected to lead to fairly shallow but relatively high velocity flows (supercritical flow state) that could be readily contained by the channel size.

The original channel scour protection, which was designed for deeper and more energetic flows than predicted by BKH, had failed during the spring of 1997. The failure was understood to be the result of permafrost subsidence that locally displaced armouring materials and exposed the highly mobile native sands upon which the spillway was constructed. Large scour holes developed in the spillway and subsequent piping and deposition of sand from slopes above the channel ultimately rendered the channel unusable.

The Mine Operator reconstructed the channel in 1998 using a design provided by Vista Engineering Ltd.; however, details of the actual reconstruction process are not available. Therefore, only the finished product surface could be viewed and evaluated during this assessment. Observation of this surface by BKH and EBA confirmed that it consisted of a heterogeneous mixture of residual soil and mine waste rock varying in size from fine sand to 600 mm rock fragments. The placement technique used to distribute this material over the spillway resulted in the channel bed being covered with a predominately fine sand and gravel fraction of the material and the channel sides being covered with a coarser "riprap" fraction of the material.

It was the opinion of BKH that the gravel and sand material lining the channel bed would become unstable at flows as low as 0.6 m^3/s. This stable flow limit was less than the 1.2 m^3/s projected for the 20 year storm flow and far less than the 3.0 m^3/s projected for the 200 year storm flow.

5.3 Diversion Spillway Upgrading

5.3.1 Upgrading Design

As described in Section 5.2, BKH concluded that the spillway section of the Dome Creek diversion was not adequate to pass even the 20 year storm event; therefore, upgrading of the spillway was necessary. Given that this spillway is considered temporary (not the closure spillway), the upgrading level was designed to handle the 200 year event.

To pass the 200 year event, a 300 mm thick gravel lining and a 600 mm thick riprap blanket were specified. In addition, riprap drop structures were also required to create stilling ponds at two metre vertical intervals down the length of the spillway. Full details of these modifications are described in initial spillway assessment report presented in Appendix B and within the Construction Report [2].

5.3.2 Upgrading Activities

Modification of the spillway, according to the recommendations of BKH was initiated in October 2000 during the reconstruction of the seepage control dyke. The intent of the modifications was to utilize existing material stockpiles and to complete the activities prior to the onset of winter conditions. Unfortunately, the proposed modifications could not be completed in October 2000 due to ice development within the channel. Activities completed in October 2000 consisted of placement of the gravel bedding over the channel base and placement of some riprap cover over this material. The drop structures were started but not completed.

In July 2001, construction activities resumed in the spillway. Construction was preceded by a scour and piping failure that developed near the end of the spillway along the south side of the channel. The cause of the failure was not determined; however, permafrost thaw due to disturbance that could have resulted from seepage dyke reconstruction traffic in the fall of 2000 is a possible cause. During July and August 2001, the failed section was rebuilt, additional riprap was placed in the channel, and the drop structures were completed, excluding Drop Structure #1 that was not installed due concerns about glaciation. An important observation of the completed modifications was that the riprap coverage and thickness desired by BKH was not achieved because available riprap stockpiles were nearly depleted.

5.4 Final Spillway Condition

In October 2001, Mr. Bernie Kallenbach, P.Eng. of BKH returned to the site in order to observe the spillway modifications completed in 2000 and 2001. Mr. Kallenbach's report based on this site visit is presented in Appendix B.

The findings of this final assessment were that the channel improvements have increased the stable flow capacity that could be handled by the spillway from 0.6 m^3/s to nearly 1.2 m^3/s. As such the channel is still only adequate to handle flows up to the 20 year return storm. In order to achieve the desired 200 year flow capacity, the riprap coverage, which is currently only half of the desired thickness and covers only 50% to 80% of the channel base, would have to be increased. In addition, many of the drop structures were found to be substandard and would require additional work in order to achieve the 200 year flow capacity.

The assessment by BKH is further supported by the concerns of EBA in regard to the construction of the spillway channel. EBA's observations during the recent reconstruction and failures suggest that there is little certainty as to whether consistent construction is in place underneath all sections of the spillway. It is considered likely that geotextile coverage, filter gravel thicknesses, as well as riprap thickness vary considerable along the spillway length. This likelihood, combined with the two past failures (that are considered to related to permafrost thawing) suggests that the spillway reliability will always be suspect unless full reconstruction is enacted.

6.0 TAILINGS DAM ASSESSMENT

EBA's mandate to assess the tailings dam included two aspects; firstly the dam had experienced some deformation and subsidence along the south abutment and this was to be evaluated; and secondly to evaluate the overall static and dynamic stability of the structure. The stability was to be considered for both the dam condition as of August 2000 and for the dam condition as it is expected in August 2005.

In assessing these two aspects of the dam it was necessary to also review the performance of the dam, specifically the performance in terms of permafrost thaw, pond levels, phreatic levels within the dam, seepage, and settlement. Each of these performance factors provides input into the stability and/or subsidence evaluations that were to be completed.

6.1 Dam Performance

6.1.1 Permafrost Thaw

Thawing of permafrost underneath the dam is considered to be one of the most significant performance parameters for this dam. Thawing of the permafrost could lead to relatively large total and differential settlements. It would also increase the amount of seepage underneath the structure. Excess pore pressures that could develop within thawing permafrost could also impact the stability of the dam. Finally, thawing of frozen sand zones could represent a potential stability hazard should they be loose enough to liquefy during a seismic event.

In EBA's Data Summary Report [1], five sources of data were reviewed to evaluate the depth of thaw into the native permafrost soil that underlies the tailings dam. These sources consisted of:

1. Baseline permafrost data from exploration of the site in 1994 and 1995, and from construction of the dam in 1996.
2. Data obtained from thermistors between 1995 and 2001.
3. CPT testing data from August and September 2000.
4. Probe holes and testpits advanced by EBA in August 2000.
5. Observations of large scale excavations during the reconstruction of the Seepage Dyke in October and November 2000.

It should be noted that all of these information sources, excluding the original baseline data, are limited to providing information on permafrost thaw downstream of the dam centreline. No data is currently available to predict or to confirm permafrost thaw depths upstream of the dam centre-line and within the impoundment.

Based on a review of the data from these sources EBA concluded that the depth of permafrost thaw underneath the dam is highly variable and appears to be a function of the original aspect of the native ground, the concentration of seepage over or through the ground, and the amount of stripping completed during construction. Three different thaw zones were identified underneath the dam. These consist of the south abutment and valley base, the north abutment, and the north terrace.

The depth of thaw into the original permafrost underneath the south abutment and valley base ranges from nil to as much as 4.5 m. However, the typical range based on the thermistors and the CPT test data is 1.5 m to 2.5 m, except near the edges of the dam (dam toe and south abutment interface) where a thaw depth of less than 1 m is estimated.

The north abutment slope comprises the former south facing side of the Dome Creek valley. Below an elevation of about 1146 m (below the gently sloping north terrace) it has experienced much more significant thawing than other locations. Given that this area had an original active layer thickness of between 2.0 m and 2.5 m, the depth of thaw ranges between 5 m and 6 m below the former active layer.

Observation of the excavation completed for the reconstructed Seepage Dyke in autumn 2000 revealed that thawing had proceeded 18 m laterally into the north slope (measured from the toe of the slope) as well as 5.4 m vertically below the toe of the slope. The result is that a thick thawed zone is expected to extend from within the impoundment to past the dam toe along the north abutment.

The final thaw zone that was identified was the north terrace. Reviewing the data from Klohn's remaining thermistor and EBA's probe holes in this area suggests thaw depths of 2 m to 3 m below the original ground. The actual retreat of the permafrost table is dependent on the active layer thickness that was originally present. Based on the original exploration holes, the active layer ranged from 0.15 m to 2.5 m depending on the type of ground cover. Therefore the amount of actual retreat varies from over 2 m to essentially nil.

In order to predict the depth of thaw five years into the future, EBA has reviewed the thermistor data to identify thawing trends and has completed a one dimensional thermal analysis. The trend for the south abutment and valley bottom is that very little additional thawing is anticipated. For this location underneath the dam, it is felt that the organic materials that were not fully removed are acting to effect some insulation of the foundation soils.

For the north abutment, a prediction of the expected permafrost retreat could not be prepared. This is because there are no deep thermistor installations within this zone. Moreover, this zone is expected to be lacking in organics as well as having high seepage flow rates. EBA can only state that further thawing is likely and that it could be substantial.

For the north terrace section of the dam, the Klohn instrumentation data from installation (DH 95-04) can be reviewed to crudely predict the future thawing rate. The response of this thermistor string between 1998 and 2001 seems to suggest that the permafrost is relatively stable. EBA would suggest that thawing would be minimal except where concentrated seepage flows may lead to localized zones with significant thaw.

These current and predicted thawing depths can be compared to the design assumptions presented in the designer's Final Design Report [13]. Thawing predictions for the first 3 years of operation were presented for the maximum dam height cross section (main embankment) and these indicated thawing would be limited to underneath the tailings pond and underneath the toe of the upstream toe berm.

At the upstream toe, the depth of thaw was predicted to be about 2 m. From the upstream toe the permafrost table would rise upward until it had aggregated into the dam fill by about 3 m at the dam centreline (roughly 1.5 m above the original ground elevation before stripping). The permafrost table would then drop back down to the fill/native soil interface level (1.5 m below original ground) by about 15 m from the downstream toe of the dam. The predictions for the period after closure of the facility indicated that the permafrost table would move upward significantly into the upstream and downstream portions of the dam.

The thermistor data has shown that the actual thaw has exceeded the original predictions underneath the toe berm crest. This suggests that significant refreezing of the dam fill has not occurred at this time.

6.1.2 Pond Levels, Phreatic Levels, and Seepage Volume

The control of seepage and phreatic levels within a tailings dam is a critical aspect of the dam's overall performance. High phreatic levels and/or high seepage flows ultimately increase the potential for slope failures and/or piping. Moreover, seepage of contaminants can also be an environmental liability if it is not properly collected and treated.

Long term control of the phreatic levels and seepage for the Mount Nansen tailings dam was to be based on gradually raising the pond level over the 3 year mine life, having a 50 m wide tailings beach comprised of relatively fine grained tailings, and maintaining relatively shallow depths of water within the impoundment.

In the Designer's seepage evaluation [13] it was also shown that the upstream toe berm would function as a drain and would draw down seepage before it enters the main embankment. The resulting phreatic levels and seepage volumes would be low.

In EBA's Data Summary Report [1] the phreatic levels (indicated by piezometers underneath the dam crest and toe berm crest), pond levels, and seepage volumes (indicated by pump back volumes) were reviewed. The distribution of tailings within the impoundment as indicated by the bathometry survey and the beach survey, and grain size testing of the tailings soils was also reviewed. Overall, the review of these data sources has shown poor correlation to design predictions and assumptions likely due in part to the operation of the pond. Details of the completed reviews are contained in Sections 6.1.2.1 to 6.1.2.3.

6.1.1.1 Pond Level

The design had assumed the pond level would show a gradual increase over the life of the impoundment with the maximum level being established at Elev. 1149.7 m at the end of the three year mine plan. In addition the amount of water stored behind the impoundment was to be limited to 30 000 m^3 to 40 000 m^3. As noted in Section 2.3, the pond level in June 1997 (month 8) had already reached Elev. 1149.0 m level, which was predicted for June 1999 (month 31). Since that time, the average pond level has been roughly Elev. 1149.6 m, which is close to the end of mining predicted level of 1149.7 m.

The high pond level recorded early in the life of impoundment also meant that the volume of water stored within the impoundment greatly exceeded the design assumptions. The excess of stored water was in the range of 130 000 m^3 during the first year of operation. This extra water was present prior to significant tailings deposition. The tailings were required to act as a seepage barrier and insulating layer for thermal protection.

Since the shutdown of mining operations, the pond level has been maintained at close to the design elevation; however, the depth of water and the distribution of tailings within the impoundment are significantly different than the design assumptions. The design had assumed that the beach width would be 50 m at the end of the third year of operation.

Moreover, the tailings slope would be quite shallow both on the beach and below the water surface. This would have resulted in shallow water depths near the dam.

The bathometry and beach surveys completed since the shutdown of the mine site in February 1999 has shown that the assumed beach and tailings configurations have not been achieved. Depending on the pond level, the average beach width approaches 35 m to 40 m, with a minimum level over at the north end of the main span of the dam of less than 20 m.

The bathometry survey also shows that the slope of the submerged tailings is relatively steep resulting in water depths of over 2.5 m near the north abutment of the dam. This area of relatively deep water also happens to be adjacent to the minimum width of the tailings beach and the area of greatest permafrost thaw.

6.1.1.2 Phreatic Levels

In terms of phreatic levels within the dam, the seepage analysis presented in the Final Design predicted a "typical" phreatic surface as a function of foundation thawing. The typical phreatic level underneath the dam crest was shown as less than Elev. 1137 m. This prediction was based on the incorrect assumption of Elev. 1135 m for a tail water level (the actual dam toe is Elev. 1130 m). EBA reproduced the seepage analysis using the parameters and methodology of the designer's, but altered the tailwater level and dam toe elevation to match the actual as constructed design. This revised analysis predicted a phreatic level under the dam crest of Elev. 1136.8 m when the foundation thaw depth was 8 m. The same analysis based on a tailwater and dam toe elevation of 1135 m predicted a phreatic level of Elev. 1138.7 m.

The actual phreatic surface across the dam crest varies considerable but is generally in excess of Elev. 1140 m across the main embankment. Across the north terrace section, the phreatic level is not known; however, testpitting and probe hole drilling along the downstream toe of the north terrace section suggests that the phreatic level is in the order of 1 m ±0.5 m above the permafrost table. This would suggest that it ranges from Elev.

1149.4 m at the emergency spillway channel to Elev. 1143 m near where the main embankment begins. The greatest difference from the predicted level within the main embankment is at the south abutment where phreatic levels typically exceed Elev. 1144 m versus the predicted level of less than Elev. 1137.

The difference in the phreatic level across the main embankment (Elev. 1144 m to Elev. 1141 m) may be related to the distribution of tailings within the impoundment and the depth of thaw of the foundation soils underneath the dam. At the north abutment of the main embankment, the least thickness of tailings cover is present over the upstream toe berm and the natural sand slopes. In addition this area also has the greatest amount of thawed sandy foundation soil. The result of this seems to be that the thawed zone of the north abutment is acting in concert with the upstream toe berm to form a drain that is drawing down the phreatic level in that area – similar to the predicted action of the upstream toe berm in the designer's seepage analysis. At the south abutment, the lack of significant thaw and the presence of a thick tailings cover in front of the dam and over the upstream slope may be impeding drainage and leading to the recorded high phreatic levels at the dam crest.

At the downstream toe berm crest, the phreatic level seems to level out to a typical elevation of about 1137 m across the main embankment, although there is still a small drop from south to north (about 0.5 m). EBA's revised seepage analysis, using the Klohn parameters applied to the correct tailwater and toe elevation, predicted a phreatic level of around Elev. 1133 m underneath the toe berm crest – but this was for a foundation thaw depth of 8 m.

From the toe berm crest to the toe of the dam no piezometer data is available to confirm the current phreatic levels. Prior to the construction of the emergency toe berm in July 1997, seepage was daylighting at an estimated elevation of 1135 m on the downstream face of the dam. Since the construction of the toe berm, seepage is no longer daylighting on the dam face during the summer months. During some winter months, seepage has caused the formation of ice, which has been observed glaciating out from the south side of the downstream face of the dam some distance below the crest of toe berm but no survey has been completed to identify the elevation where this occurs. Based on these observations, it is inferred that the phreatic level underneath the downstream face of the toe berm is high and likely within 1.0 m to 1.5 m of the face of the slope.

Overall it is clear that higher than anticipated phreatic levels have been and are still being experienced by the tailings dam.

6.1.1.3 Seepage

The performance of the Mount Nansen Dam in terms of seepage has been poor. High seepage pressures associated with high pond levels led to sand boils at the toe of the dam and slope failures along the north (natural) abutment slope above the seepage pond in July 1997. Continued piping development at the dam toe could have lead to a slope failure developing and the potential breaching of the structure. This action may have been prevented by the placement of an emergency toe berm and armouring the natural abutment slope in July/August 1997.

The volume of seepage that has occurred through the dam has been estimated based on the amount of seepage captured in the seepage pond and pumped back to the impoundment. The percentage of total seepage that has been intercepted by the seepage pond and returned to the pond has not been determined.

Seepage through and bypassing the seepage control dyke was observed during the reconstruction of the dyke in autumn 2000. Moreover, seepage was also observed to be flowing through the active zone on the top of the north terrace and the emergency spillway. Neither of these seepage paths are being intercepted by the reconstructed dyke.

Between the spring of 1997 and November 1998, the volume of water pumped from the seepage pond was estimated by the mine operator based on occasional records of pumping rates and times. After November 1999, a flow meter was installed and near continuous pump back volumes have been reported.

Review of the pump back data indicates that pump back rates have varied from 2.3 L/s to 4.1 L/s, although it was noted that these values could vary significantly on a daily basis due to snowmelt and runoff from precipitation entering into the seepage pond. In terms of seasonal variations, typically the winter rates have ranged from 2 L/s to 3 L/s where as summer rates have ranged from 3 L/s to 4 L/s.

The winter pump back rate is expected to more closely estimate the seepage rate, as there is no snowmelt or precipitation inflow into the seepage pond. Nevertheless, the increased pump back rate during the summer months could at least partially be due to higher seepage resulting from increased pond levels typical in the summer months. Comparison of pump back rates from winter 2000 and winter 2001 indicated an increase of about 0.3 L/s to 0.4 L/s. This could represent either increased seepage through the dam and/or the capture of more seepage after the reconstruction of the seepage dyke in the fall of 2000.

The predicted seepage rate from the dam design would suggest that seepage in the order of 0.5 L/s would be expected based on the level of thawing that has been observed. This is four to eight times less than the seepage that has been estimated based on pump back records.

As part of EBA's work scope a seepage analysis was completed in an attempt to match the recorded phreatic surface and translate this into an estimate of seepage. This 2-D analysis was completed for the maximum dam height cross section. It was found that the hydraulic conductivities necessary to match the existing phreatic levels were roughly twice the values used by the designer's for the embankment sand and thawed foundation soils and 5 times the value assumed for the tailings.

Extrapolation of the seepage flux from the modelled section using a width of 200 m (as per the Final Design estimate) would predict a seepage rate of 3.7 L/s. Given the analysis was based on the maximum dam section it would be expected to produce a conservative estimate and this seems to be the case as 3.7 L/s is on the high range of captured seepage (which includes snow melt and runoff).

Moreover, it is noted that such an extrapolation is not theoretically correct as seepage in this situation is clearly impacted by 3-D effects such as the variable beach width, embankment height, and foundation thaw depths.

6.1.2 Settlement & Deformation

Settlement of the dam crest was identified in the design process as a likely occurrence due to the thawing of permafrost foundation soils. The designer's settlement estimate for the first 3 years of the dam life was conservatively set at 0.6 m with this resulting from up to 4 m of permafrost thaw.

Monitoring of the predicted settlement was the responsibility of the mine operator and was to be based on surveying of monitoring pins across the crest of the dam. Unfortunately the mine operator did not complete these surveys as frequently as required, did not complete them accurately, and/or did not record the results in a proper manner. Therefore, there is a lack of data on the settlement of the dam crest and this has made the comparison of predicted and actual crest settlements impossible. The maximum crest settlement that could be inferred during the period of September 1996 to September 1999, based on the available data was 0.27 m as reported by Klohn in January 2000 [7]. This settlement amount was based on the simple assumption that the as-built crest elevation was uniformly at Elev. 1151.50 m.

Since the period ending September 1999, Water Resources has had three settlement/deformation surveys completed on the dam. The first survey, conducted in September 1999, was to install proper monitoring pins and to obtain baseline data for future settlement/deformation surveys. In total twenty-seven monitoring pins were established on the crest (11 pins) and downstream face of the dam (16 pins). In addition to these installed pins, nine borehole/instrumentation casings were also surveyed in 1999 for use in settlement/deformation monitoring. Three of the boreholes casings were located on the dam crest with the remainder being on the downstream face of the dam.

The established pins and borehole casings were resurveyed in August 2000; however, damage to the control monuments which had been established in 1999 resulted in some uncertainty in the validity of horizontal movements that were determined from the comparison of 1999 and 2000 data. It was the conclusion of the surveying consultant that any horizontal movement identified by the comparison of the two surveys was either within the limits of precision of the survey or was the result of disturbance to the horizontal control utilized at the site. Therefore, it was deemed that no significant horizontal movement had occurred during this period, where significant is defined as greater than the precision of the survey. Vertical settlements determined during this period ranged up to 20 mm; however, the precision of the survey was considered to be ±20 mm. Therefore, the vertical movement was also not considered to be significant.

During the August 2000 survey, some additional monitoring pins were added to the monitoring system in order to provide a higher density of coverage in the area of the south abutment subsidence. In total seven additional pins were installed in this area.

All of the monitoring pins and borehole casings were resurveyed in October 2001. The results of that survey showed both vertical and horizontal movement had occurred during the period between August 2000 and October 2001. The data indicated cumulative (1999 – 2001) dam crest settlements ranging from 8 mm to 61 mm. The most significant crest settlements were located over the south abutment side of the crest. Corresponding horizontal movements of the dam crest were not significant, as considered for the period of August 2000 to October 2001.

Cumulative settlements (1999 – 2001) on the downstream face of the dam were typically in the range of 10 mm to 40 mm. Horizontal movements during August 2000 to October 2001 on the downstream face varied from 6 mm to 64 mm with a typical range of 15 mm to 35 mm. The direction of horizontal displacement on the downstream face of the dam was primarily in a downslope direction (Mine Grid East), although some monitoring pins located near the north abutment showed movement in a southeast direction. Figure 6 presents the settlement/displacement data obtained from the completed surveys. The figure includes the cumulative settlement of each monitoring point (1999 – 2001), as well as the horizontal displacement and direction from the period of August 2000 to October 2001.

In addition to the obtained survey data, visual evidence of deformation and cracking of the dam has also been observed. Subsidence and cracking of dam fill materials has been observed on the downstream face of the dam near the edge of the south abutment. The deformation consists of depressions ranging up to 1.2 m in depth and extensive cracking around and near the depressions. This area of deformation was not observed until after the construction of the emergency toe berm and crest access road in the summer and fall of 1997.

Prior to July 1997, an erosion channel had developed along the interface between the south abutment and the dam fill materials. This channel varied in height and width; however, a steep face of embankment sand up to 1.2 m in height developed along some sections of the channel. Seepage was observed daylighting into this channel at elevations above 1135 m and this was creating sloughing and erosion of the exposed sand face.

In the fall of 1997, the mine operator covered this trench and built an elevated access road over it leading from the south end of the toe berm up to the dam crest. The road over the channel area was built using a mixture of waste rock and excavated residual soils (gravel, sand, and cobbles with a trace of silt). The quality of construction, in terms of placement and compaction, during this operation is not known.

During the summer of 1998, the area downslope of the road and toe berm was observed to contain the depressions and cracks as already described. As far as can be determined by EBA, all of these features are located in areas that were subject to material placement in the fall and summer of 1997 and all are downslope of the downstream edge of the toe berm crest and the crest access road. Observation of the highest extent of tension cracks has not shown any observable change in the period of 1998 to 2001 even though surveying of monitoring pins in this area has indicated both vertical and downslope horizontal movement is occurring in this area.

EBA completed two testpits in this deformation area -- one was located in a small depression near the edge of the dam (south downslope face) and the other was located immediately downslope of the crest of the toe berm at its southern end. The testpit at the edge of the dam revealed that the thawing underneath the fill soils near the edge was minimal (0.5 m). The other testpit, located further in from the edge of the dam, could not be safely excavated to permafrost without greatly disturbing the downstream slope of the dam.

Both testpits revealed residual soil fill that contained boulders and cobbles. This residual soil/waste rock fill varied in thickness (thinner at the dam edge) and did contain observable voids associated with the cobbles and boulders in the matrix of the fill. The testpit at the edge had these materials overlying a non-woven geotextile. Underneath the geotextile, native organic rich sand was observed.

The testpit located further in from the dam edge had 1.0 m to 1.2 m of fill sand underlying the residual sand, gravel and cobbles. The embankment sand was saturated and was found to easily liquefy when disturbed. Underneath the embankment sand, organic rich native sand was observed.

Overall, the magnitude of the settlement of the dam to date cannot be conclusively determined; however, it seems that the settlement may be less than was anticipated by the Designer. In addition, deformation of the downstream slope of the dam is occurring particularly on the south abutment area that is known to have high phreatic levels.

6.2 Liquefaction and Stability Assessment

As part of EBA's work scope the static and seismic stability of the downstream slope of the main embankment has been evaluated. This evaluation has incorporated the dam performance data described in EBA's Data Summary Report [1] and summarized in Section 6.1, above. The seismic stability evaluation also included the evaluation of the liquefaction potential of the dam fill and of thawed native soils, and the assignment of residual strength values for any materials that may be shown to be potentially liquefiable. The scope of this liquefaction and stability assessment included the following tasks

- Conduct additional field characterization studies.
- Identify the design earthquake event through a seismic hazard study.
- Assess the soil liquefaction potential for both the thawed foundation soils and the dam fills.
- Assess the stability of the dam structure taking into account potential liquefaction of dam fill materials and thawed foundation materials.

To evaluate the stability of the Mount Nansen tailings dam it was necessary to consider all of the factors that would affect the stability both in the present and in the 5 year window that was being considered for this assessment. These factors included the dam geometry, the amount of thaw underneath the foundation of the dam, the condition of the thawed soils, the pore pressures developed by those thawed soils, the phreatic level within the dam, the seismic hazard or seismic loading that could be anticipated, the liquefaction potential, the residual strength of the liquefied soils, and the strength parameters for the fill and native soil materials.

The results of the stability evaluation, including the seismic hazard, liquefaction potential, and the resultant stability analyses, are presented in the following sections of this report, as well as in the Appendices where indicated.

6.2.1 Seismic Hazard

The initial step in completing the seismic evaluation of the Mount Nansen tailings dam was to determine the appropriate Maximum Design Earthquake (MDE) for the site. Since the consequence associated with the failure of the Mt. Nansen Tailings dam is high to very high, the Canadian Dam Association Guidelines (1999) suggest that the MDE be taken as equal to that associated with a deterministically evaluated Maximum Credible Earthquake (MCE), or the earthquake having a return period of 10,000 years as determined using probabilistic procedures.

EBA contracted the Pacific Geo-Science Centre to complete the seismic hazard assessment of the mine site in August 2000. The completed assessment is presented in Appendix E. As shown in that assessment, the tectonic setting of the region is defined by three major faults that could pose a significant seismic threat to the mine site: the Fairweather fault (254 km away in the southwest), the Denali fault (125 km away in the southwest), and the Tintina fault (132 km away in the northwest). Using this tectonic setting both probabilistic analysis and deterministic analysis were completed for the seismic hazard assessment of the mine site. The results of the probabilistic assessment are summarized in Table 2. The results of the deterministic analysis are shown in Table 3.

Table 2: Results of Probabilistic Seismic Hazard Analysis

RETURN PERIOD (YEARS)	475	1,000	10,000
Peak Horizontal Ground Acceleration (g)	0.096	0.119	0.27
Peak Horizontal Ground Velocity (m/s)	0.218	0.265	0.61

Table 3: Results of Deterministic Seismic Hazard Analysis

FAULT	DISTANCE TO SITE (km)	MAXIMUM MAGNITUDE	PGA
Fairweather	254	8.7	0.131 g
Denali	125	7.3	0.125 g
Tintina	132	7.3	0.117 g

Based on these results and the recommended procedures of the Canadian Dam Association, the MDE for the Mt. Nansen dam site, would have a probabilistically determined Peak Horizontal Ground Acceleration (PGA) of 0.27g. This level of PGA was associated with a local magnitude 7.5 seismic event.

6.2.2 Liquefaction Assessment

The determined strong ground motion parameters (PGA and Local Magnitude) from the seismic hazard assessment and the cone penetration resistance data obtained from the two CPT programs completed at the site can be used in Seed's Simplified approach to determine whether soil liquefaction is likely to occur at the CPT test locations. This procedure is described in Section 6.2.2.1 and the results are presented in Section 6.2.2.2. The use of more complex evaluation procedures such as a dynamic analysis were not deemed to be warranted based on the results of the simplified procedure.

6.2.2.1 Liquefaction Assessment Procedure

The liquefaction potential was assessed by comparing the cyclic stress ratio (CSR) caused by the earthquake with the cyclic resistance ratio (CRR) of the soil. The procedures used to assess liquefaction potential of the granular soil follow the guidelines of the America National Centre for Earthquake Engineering Research (NCEER) using the liquefaction resistance chart as a function of the normalized field Standard Penetration Test (SPT) blow counts, $(N_1)_{60}$ (Youd and Idriss, 1997). It is noted that the liquefaction resistance chart was based on the original work of Seed et al. (1985) and modified by NCEER to reflect more recent field experience. The soil liquefaction resistance is also a function of the earthquake magnitude and the in situ soil stress condition prior to earthquake shaking.

The procedure used in the liquefaction assessment is outlined below.

- The earthquake induced cyclic stress ratios (CSR) can be calculated using the following simplified relationship proposed by Seed et al. (1985) for level ground condition:

$$CSR = \tau_h / \sigma'_o = 0.65\, A_{max}\, r_d\, \sigma_o / \sigma'_o$$

WHERE,

A_{max} : the peak ground surface acceleration in gravity units;

σ_o : total vertical stress;

σ'_o : effective vertical stress;

r_d : reduction factor for soil with depth;

τ_h : equivalent shear stress caused by the design earthquake.

An A_{max} of 0.35g was used in the analysis based on the peak ground acceleration of 0.27g and an amplification factor of 1.3 through the thawed foundation soil and the compacted dam fill.

- With the above computed CSR and for a factor of safety (FOS) of 1.0, the required field cyclic shear resistance ratios (CRR_{field}) can be computed as:

$$CRR_{field} = FOS \bullet CSR = CSR$$

- The cyclic resistance ratio, CRR, is derived from the required field cyclic resistance ratio using the following equation

$$CRR = CRR_{field} / (K_\sigma K_M K_\alpha)$$

Appropriate corrections for earthquake magnitude (K_M) and in-situ effective vertical stresses (K_σ) recommended by NCEER (Youd and Idriss, 1997) were applied in the calculations. Earthquake magnitude scaling factors of 1.0 were used for the maximum design earthquake (MDE)(M=7.5). A correction factor for static shear stress of K_α =1 was used in the calculation.

- The required $(N_1)_{60}$ values for the design earthquake can be obtained from CRR using the liquefaction resistance chart recommended by NCEER (Youd and Idriss, 1997).

- The required $(N_1)_{60}$ were converted to $(N)_{60}$ using the overburden stress correction factor, C_n. The required cone tip resistance, Q_c, were calculated from the required $(N)_{60}$ based on $Q_c/(N)_{60} = 5$.

- The required Q_c values were then compared with the in-situ Q_c values. If the required Q_c values (demand) are greater than the in-situ Q_c values (resistance) in the foundation subsoil, the factor of safety against liquefaction is less than one, and liquefaction is expected to occur during the design earthquake.

Soil liquefaction is considered possible only if a sandy soil is saturated or below the ground water level. For the current study, the ground water level along the dam crest is about 8 m below the crest surface. The ground water table is about 2 m below the surface at the toe crest berm, and is 5 m below the native ground surface at the north abutment (CPT-9).

6.2.2.2 Liquefaction Assessment Results

The results of liquefaction analyses under the MDE event are shown in Figures 7 to 9 for the CPT profiles at the dam crest, at the toe berm, and at the north abutment native ground, respectively.

The liquefaction analysis (Figure 7) shows that the native foundation soils at the south end along the dam crest are liquefiable (CPT-10 below 10.5 m depth, CPT-11 below 15.2 m depth). The dam fills are in general compacted and non-liquefiable except at CPT-11 where a zone of about 0.8 thick is considered liquefiable. At the centre of the dam crest, the CPT was unable to advance to the depth of the foundation soil at CPT-4, and CPT-12. At CPT-1 and CPT-12A, the dam fills are less dense than at other locations. But the risk associated with liquefaction of the dam fills at this location is considered low. Although the thawed foundation soil at the north end is much deeper than at the south end, the risk associated with liquefaction of the dam fill and the native foundation soil is considered very low (CPT-13, CPT-14, CPT-16).

At the toe berm (Figure 8: CPT-7, CPT-6, CPT-15, CPT-3 and CPT-8), the native foundation soils are considered liquefiable under the MDE event. The dam fills are in general compacted and non-liquefiable except at CPT-3 where a zone of about 0.5 thick is loose and is liquefiable.

At the north side native ground (Figure 9), the native soils are considered susceptible to soil liquefaction at CPT-9 and not susceptible to soil liquefaction at CPT-17.

In summary, soil liquefaction is considered likely to occur under the MDE event at the following locations:

- within the foundation soils underneath the dam crest except north of CPT-13 (but including CPT-13);
- within the foundation soils underneath the toe berm;
- at the north side native ground near CPT-9 below the ground water table;
- within some sporadic zones (0.5 m to 0.8 m in thickness) in the dam fills.

6.2.3 Stability Analyses

This section presents results of limit equilibrium analyses for the slope stability under both static and seismic conditions. For this assessment EBA has assumed that the maximum dam height cross section (centre of main embankment) was the critical section to be evaluated for stability. The location of this cross section is shown in the Figure 3. The modelled cross section is presented as Figure 10. The following different scenarios were analyzed in the stability assessment:

- Dam static stability under current thawing condition of the foundation soils.
- Dam seismic stability under current thawing condition of the foundation soils.
- Dam static stability under 5-year thawing condition of the foundation soils.
- Dam seismic stability under 5-year thawing condition of the foundation soils.

Limit equilibrium analyses were conducted for both circular and specified non-circular failure surfaces using three methods of limit equilibrium analyses (the Bishop method for circular surface, the Janbu method for specified non-circular surface, the Morgenstern-Price method for both circular and non-circular surfaces). The limit equilibrium analyses were conducted using the computer program SLOPE/W (Geo-slope International, 1998).

6.2.3.1 Thawed Depth

For the stability analysis, an average thawed depth of 2.5 m was used under current condition. A thermal analysis was also conducted to predict the thawed depth of the foundation for the critical cross section in the next 5 year window. The result of the thermal analysis indicated that the average thawed depth in 5 years would likely be about 3.0 m. The following thawed depths are used in the stability analyses

- thawed depth of foundation soil = 2.5 m for current conditions;
- thawed depth of foundation soil = 3.0 m for 5 year window.

6.2.3.2 Bulk Unit Weight and Soil Strength Parameters

Three soil types have been assumed to exist through the maximum dam cross section: tailings (sand & silt); fill (embankment) sand; and native foundation sand. For each of these earth materials both the bulk unit weight and effective stress strength parameters (C' and Ø) were required to complete the stability analyses.

The bulk unit weight of the fill sand can be estimated based on the limited compaction testing and two Standard Proctor tests completed on the fill sand during the dam construction. Base on this available information and applying engineering judgement, a bulk unit weight of 19.5 kN/m^3 was selected for use.

For the tailings material, there is no site specific database of materials tests available to assist in selecting a bulk unit weight. Therefore, EBA has reviewed the designer's assumptions and has applied engineering judgement and experience with similar tailings at other mine sites to estimate a bulk unit weight. Assuming a void ratio of 0.85 and a specific gravity of 2.66, a dry density of 1438 kg/m3 would be expected. Assuming an average water content of about 32%, the bulk unit weight of the tailings would then be about 1898 kg/m^3 or 18.6 kN/m^3.

The bulk density of the thawed foundation soil is difficult to determine as it likely differs from the frozen unit weight of the recovered cores during the original site exploration. Given that the material consists primarily of fine sands, with thin layers (<0.5 m in thickness) of organic peat and silts at shallow depth it is anticipated that the bulk unit weight would be similar but lower than that of the fill sand. For the analysis, the foundation sand was estimated to have an average unit weight of 19.0 kN/m^3.

For the soil strength parameters, EBA and the facility designers both consider all three materials to be cohesionless, hence c=0. For the angle of internal friction (ϕ), EBA reviewed the assumptions of the designer and considered empirical correlations with the CPT results. Moreover, it was recognized that a definitive value of ϕ could not be developed without laboratory test results. Therefore, either conservative values or a range of values would have to be assumed. For the tailings a conservative estimate of 28° was used in the analysis. With the fill soils, empirical correlations with the CPT results would suggest a ϕ above 40°, but a conservative value of 34° was assumed. Finally, for the thawed foundation soils, the CPT results suggested a ϕ value in the range of 28° to 30° and both these values were used in various analyses. The soil parameters used in the stability analyses are presented in Table 4.

Table 4: Soil Parameters Used In Stability Analyses

Soil Units	Bulk Unit Weight (γ) (kN/m3)	Frictional Strength (ϕ) (degrees)	Cohesion (kPa)	Foundation R_u value	Residual Strength (kPa)
Tailings	18.6	28	0		9.0
Compacted dam fill	19.5	34	0		-
Native foundation soil	19.0	28 to 30	0	0.10	14.4

6.2.3.3 Phreatic Surface within the Dam

Review of the phreatic data available from the dam (Section 6.1) indicates that the level has varied both seasonally and spatially over the life of the structure. For the stability analyses, a typical level and a high level were considered. For the typical case, the groundwater level along the dam crest is about 10 m below the ground surface, i.e., Elev.. 1141 m. The typical groundwater level is about 1.5 m to 2.0 m below the ground surface at the toe berm crest, i.e., Elev.. 1138 m (ground surface varies by 0.5 m).

A conservatively high groundwater level was also assumed in the stability analysis of the mid-embankment section of the tailings dam. For the case of high groundwater level, the groundwater level was assumed at 5 m below the ground surface at the dam crest (Elev. 1146 m) and at 0.5 m to 1.0 below the ground surface at the toe berm (Elev. 1139 m).

It is noted that EBA personnel have observed a winter breakout point for seepage near the south abutment of the dam. The RA has observed that this winter break point has also occurred near the mid-embankment section of the dam. In both cases the seepage that exits the dam slope was observed to freeze and build up into small "glaciers" below the breakout points. It is likely that this breakout of seepage is related to seasonal freezing of the active layer around the southern abutment and toe of the dam. Therefore, it is possible that during some winter months the conservatively chosen phreatic level for the mid-embankment section of the dam may be lower than the actual winter position for the slope section between the crest of the toe berm and the dam toe.

To account for this possibility, EBA has considered a third phreatic surface to represent a winter breakout phreatic surface that would match the high phreatic surface case except that it would breakout on the slope at Elev. 1135 m. Elev. 1135 m was chosen based on past observations of breakout seepage on the face of the dam in 1997.

6.2.3.4 Excess Pore Water Pressure in Foundation Soils

EBA's review of the instrumentation data at several piezometer locations has not identified any excess pore pressures in the foundation soils. However, in some cases it was not clear if the lower piezometers were providing valid results as they could have been frozen or partially frozen, so they may not have accurate readings. Nevertheless, we do not have concrete data to suggest that excess pore pressures exist within the foundation soils due to thawing activity.

For the stability analysis under static conditions, the foundation soil was assumed to have excess pore water pressure 10% of the total overburden stress, i.e., R_u =10%.

6.2.3.5 Residual Strength of Liquefied Soils

Based on findings from liquefaction assessment completed for the dam fill and foundation soils, the foundation soils and some fill soils were found to be susceptible to soil liquefaction during the MDE event. Soil liquefaction will typically cause a large reduction in shear stiffness and shear strength that could have a major impact on the post-liquefaction stability of a soil slope or an earth dam.

In order to model the post-liquefaction stability of the dam, residual shear strengths were assigned to liquefied soil materials in the stability analyses to assess the stability of the tailings dam under the design MDE seismic conditions.

The selection of residual strength remains a very controversial issue, as discussed by Finn (1998). Residual strength is not only a function of the void ratio for a given sand, but also a function of sample preparation technique, stress path followed during loading, and the effective confining pressure. Residual strengths are often obtained from back analysis of case histories (Seed and Harder, 1990; Idriss, 1998) or from direct laboratory tests (Vaid and Sivathayalan, 2000). Residual strengths proposed by Idriss (1998) have been used as the basis for the selection of residual strengths of liquefied soil for the Mt. Nansen tailings dam assessment. A residual strength of 9 kPa was selected for the liquefied tailings with an assumed SPT blow count of $(N_1)_{60}$ = 8. A residual strength of 14.4 kPa was used for the liquefied foundation soil based on $(N_1)_{60} = 11$. $(N_1)_{60}$ values in the range of 8 to 10 were assumed for the thawed foundation sands and for the tailings in the stability analysis for foundation liquefaction completed by Klohn in 1995.

6.2.3.6 Results of Stability Analyses

Factors of safety of the dam section at the current conditions and in 2005 are summarized in Table 5 for the static condition and in Table 6 for the seismic condition. Typical failure surfaces are shown in Figures 11 and 12.

Table 5: Results of Stability Analyses - Static

Cases	Variation in soil/groundwater parameters		Static Minimum Factor of Safety
	Foundation Soil Friction Angle (ϕ)	Level of Phreatic Surface	
Current Year 2000			
1	30	Typical level [1]	1.61
2	30	High level [2]	1.44
3	29	Typical level	1.56
4	29	High level	1.40
5	28	Typical level	1.52
6	28	High level	1.36
7	28	High level	1.67 [3]
5-Year Window Year 2005			
8	30	Typical level	1.59
9	30	High level	1.43
10	28	High level	1.34
11	28	Slope breakout [4]	1.23

Notes:
1 Typical level: groundwater level at El. 1141 m at dam crest, at El. 1138 m at toe berm.
2 High level: groundwater level at El. 1146 m at dam crest, at El. 1139 m at toe berm.
3 For a non-circular slip surface
4 High level with seepage breakout on the downstream face at Elev. 1135 m.

Table 6: Results of Stability Analyses - Seismic

Year	Post Liquefaction Static Factor of Safety	Post Liquefaction Pseudo Static Factor of Safety (0.27g)
Current Year 2000	0.70	<< 0.7
5-Year Window Year 2005	0.70	<< 0.7

Based on the results of stability analyses, it is therefore concluded that

- The factor of safety of the critical dam section varies from 1.52 to 1.61 under current observed level of phreatic surface (groundwater level within the dam). This range of factor of safety is considered adequate under static conditions for dam stability.
- The factor of safety under the high level of phreatic surface could decrease to 1.36 and is considered inadequate for dam stability under permanent condition. The groundwater level at the toe berm is critical to the dam stability.
- The perspective of dam stability for the 5-year window (to 2005) can be viewed as similar or the same as for the current condition. Moreover, the worse case phreatic level scenario (slope breakout at Elev. 1135 m) was considered for the 2005 condition and was found to decrease the factor of safety from 1.34 to 1.23.
- Under the design seismic condition (MDE event), the risk of the dam failure along the liquefied foundation soil is considered to be high. The post liquefaction static factor of safety was calculated to be about 0.70 indicating an unstable slope condition.

The above conclusions with regard to the stability of the dam do not address any potential safety issues associated with piping due to excessive underground water seepage. Moreover, they are based on assumed soil strength parameters. Actual parameters are likely to vary above or below those used in this assessment, although the range of values for the native sand is supported by empirical correlations to cone penetration resistance[2]. It should also be noted that the analysis using the "slope break out" phreatic case does not account for any seasonal refreezing of the thawed foundation soils. The seasonal refreezing of the thawed soils is expected to produce the breakout seepage that has been observed near the south abutment, it is also expected to strengthen the otherwise "thawed" foundation soil zone of the dam. As always in limit equilibrium analyses, the factor of safety has some undefined level of uncertainty.

2 "Interpretation of Piezocone Test Data for Geotechnical Design", Soil Mechanics Series Nos. 157 & 158, University of British Columbia, September 1995.

Irrespective of the results of the limit equilibrium analyses, the observed horizontal deformations on the downstream face of the dam, particularly in the area of the south abutment, suggests that the static factor of safety of the downstream slope is less than would be expected in order to ensure an acceptable level of performance from the dam in the long term.

6.3 South Abutment Subsidence

As discussed in Section 6.1, features typically associated with subsidence have been observed along the southern edge of the downstream face of the dam. These features were first observed following filling of an erosion channel that developed along the interface between dam fill and native soils after the first year of operation. There are a variety of possible causes for this subsidence. These include thaw settlement, piping, settlement of loosely placed, poorly graded fills, and downslope creep of the materials due to inadequate localized stability.

The available data on thawing underneath the south abutment would suggest that thawing of native soils is not great. Moreover, EBA's revised thermal analysis suggested that the greatest amount of thawing would have occurred during the initial year of the dam's life; therefore, the observed subsidence after the fall of 1997 may not be attributable to thaw.

Piping of embankment sands through the loosely placed coarse graded fills is another possible contributing factor; however, significant piping would have had to occur in the winter of 1997 and spring of 1998 and would then have had to stop or decrease greatly in magnitude as no more observable subsidence has occurred. This seems unlikely, as a high piping rate would be expected to continue unless some intervention occurred.

It is possible that that some of the observed subsidence is due to the fill materials being poorly graded and placed in a very loose state. This mechanism would be consistent with the fact that no subsidence areas exist upslope of the crest of the emergency toe berm or of the crest access road, as those materials would have been compacted through hauling of material and equipment traffic. It is quite likely that materials placed below the crest of the berm and below the downslope edge of the road were loosely dumped and spread. Subsequently these materials could have settled and cracked to form the subsidence features that are now observed.

That poor construction could be a significant factor in the original development of the subsidence and deformation features of this area seems quite likely. However, some amount of thaw subsidence and piping (removal of sand) may have also contributed to the development of these features.

Even considering these three potential causes, the downslope creep of the area as determined by survey measurements suggest that another factor is involved. As discussed in Section 6.2, the determined downslope creep of the area suggests that inadequate stability of the slope may also be a contributing factor and that this must be considered in future observation and maintenance of the area.

Piping

While not included within the original work scope, EBA has been asked to comment upon the likelihood of piping phenomenon occurring at the tailings dam or its abutments. Piping within earthfill dams can take two forms: "heave" piping, which can take the form of sand boils, due to high uplift/seepage forces; or "conduit" piping that develops from internal erosion and migration of material through or underneath a structure.

The heave type of piping was observed in 1997 along the north abutment slope above the reclaim pond. Heaving at the base of the slope lead to slumps and small slope failures above the seepage pond. It is also believed that this type of piping was initiated at the downstream toe of the dam in 1997 prior to placement of the emergency toe berm.

Heave type of piping can be controlled using inverted filters to prevent particle migration while also increasing the vertical effective stress in the heave area. The emergency toe berm and north abutment slope armouring were constructed in 1997 to serve as inverted filters, thereby preventing further heave failures. The slope armouring and emergency toe berm were designed without the benefit of detailed engineering studies and constructed rapidly without full supervision by engineering personnel. They were also not designed to serve as long term solutions to the potential piping problems. Thus, it is conceivable that piping could be occurring to some extend underneath the hastily constructed filters. Certainly, the south side of the emergency toe berm should be considered suspect as piping of embankment sand into voids within the waste rock and residual soil fill could be occurring.

The "conduit" form of piping was also previously observed at the site. During the 1997 failure of the Dome Creek Diversion spillway, a conduit developed through the internal erosion of sand underneath seasonally or perennially frozen soils on the north side of the spillway. The conduit allowed the direct connection of flow from the higher elevation temporary diversion channel to the lower elevation spillway. More importantly, it also led to the transportation of significant amounts of sand from the conduit into the spillway. Subsequent to the thawing of the frozen "roof", the conduit collapsed.

The likelihood of a similar conduit developing on the dam or its abutments is a function of the ability of the embankment and foundation soils to form a "roof", of whether an unprotected exit point exists, and of whether there is a sufficient hydraulic gradient to initiate the erosion of the sand.

It is generally accepted that cohesionless materials such as the sand used to construct the dam cannot form conduits without some buried feature such as an outlet pipeline to provide support. In the case of the observed piping in the spillway, frozen sand provided the "roof" support that enabled the conduit to form. The issue of hydraulic gradient has not be quantitatively reviewed by EBA; however, it is clear from observation of the site that transport of the native sand occurs at even low gradients as witnessed in the 0.1% grade diversion ditch. Therefore, it is reasonable to assume that a sufficient gradient exists to lead to piping if an unprotected exit point and roof support were to exist. In terms of roof support, the only reasonable condition that could support a "roof" in the dam for a sufficiently long period of time to initiate a breach would be seasonally frozen soils.

On the north abutment there would be some risk that open work gravel layers within the native fluvial sand deposits could assist in forming a thin conduit zone that would allow connection between the reservoir and some exit point on the north abutment slope. It is known that concentrated seepage exits from the north abutment and the north side of the dam toe. Therefore, it is possible that concentrated seepage along gravel layers is already occurring. Fortunately, there has not been any sign of sand transport associated with this concentrated seepage.

Based on these factors, it is believed that there is some un-quantifiable hazard of piping related phenomenon developing at the tailings dam. The hazard exists wherever seepage exits the structure without a proper filter in place to prevent the erosion and transport of sand. Based on EBA's knowledge of the site, this is most likely to occur on the north abutment with its open work gravel layers and highly concentrated seepage flows or on the south abutment that has a high gradient and suspect filter construction. Regardless, the type of piping that could develop on the dam is more likely to be a heave type of piping, unless seasonally frozen embankment materials or open work gravel zones enable a conduit to form.

7.0 CONCLUSIONS AND RECOMMENDATIONS

7.1 Conclusions

This assessment and evaluation of the safety of the Mount Nansen Tailings Impoundment has identified significant dam safety deficiencies in its current state and these deficiencies are not anticipated to improve in the next five years. The dam deficiencies include:

- The estimated range of the static stability of the downstream slope, within the normal range of pond levels and phreatic levels experienced by the dam, is lower than recommended by CDA standards.
- Inadequate spillway stability for the any event greater than a 20 year return storm.
- Inadequate spillway capacity for any event greater than a 200 year return storm.
- Inadequate dynamic downstream slope stability.
- Some un-quantifiable potential for piping.

In addition, the seepage rate through the tailings dam is high and well in excess of design estimates. Due to the level of contaminants within the seepage, seepage control in the long term is required to avoid further environmental degradation. Deformation of the dam, particularly along the south abutment, supports the conclusion that the factors of safety against slope failure is less than required to ensure acceptable long term performance.

Finally, permafrost thaw underneath the north abutment, while not quantitatively predicted, is expected to continue to increase, leading to the potential for increased seepage, settlement, and possible piping phenomenon.

The greatest hazard to the structure would be the occurrence of a significant seismic or hydrological event (flood). And while these are mathematically remote events, it is important to consider that these could lead to a rapid failure that could not be readily prevented once initiated.

In terms of the hazard under normal operating conditions, piping poses some potential hazard that cannot be well defined. Fortunately it is likely that piping would be observable by monitoring in its development and could be mitigated if sufficient on site forces were available. More importantly, the diversion spillway is inadequate for even normal flow conditions and its reliability after two partial reconstructions is suspect. Continued maintenance of the spillway is expected to be an ongoing activity.

7.2 Recommendations

This dam safety assessment has identified safety deficiencies at the Mount Nansen tailings facility. These deficiencies can be grouped into those associated with the current operating conditions at the facility and those associated with extreme events that could occur. The deficiencies associated with current or normal shutdown conditions are the un-quantified piping potential, the lower than recommended downstream slope static stability, and inadequate spillway capacity. For the first two of these deficiencies, continued monitoring and surveillance is the recommended approach to mitigate the hazards in the short term period considered for this assessment. Specific surveillance and monitoring recommendations are presented in Section 7.2.1.

For the spillway deficiency, surveillance, monitoring, as well as continued upgrading is recommended. The minimum upgrading activities recommended by in the 2001 Spillway Assessment by B.K. Hydrology (Appendix B) should be implemented in the summer of 2002.

For the extreme event safety deficiencies, further assessment and study is necessary in order to determine whether the facility should be upgraded to resist those events or be decommissioned. In addition, both of these options would require a suitable implementation timeframe that limits hazard to a defined and acceptable level. Specific recommendations regarding extreme events are presented in Section 7.2.2.

7.2.1 Maintenance and Surveillance

Even under normal operating conditions piping and slope stability are possible safety concerns for the Mount Nansen dam. Should piping or slope movement develop there should be visible signs that they are occurring. Specifically, settlement, deformation, cracking, and/or seepage carrying sand as a bedload should be observed at the dam. Therefore, continued surveillance and monitoring is essential in order to identify slope movement and/or piping and to prevent these actions from leading to a safety incident. The following actions are recommended:

- Maintain the pond level at the lowest possible level throughout the year.
- Complete daily inspections of the dam by trained caretaking personnel.
- Monitor the spillway condition daily during the spring, summer, and fall months (open flow conditions) and weekly during the winter months (after freeze-up).
- Continuously record and review seepage volumes that are pumped back from the reclaim pond.
- Regularly monitor and evaluate phreatic levels, pond levels, and ground temperatures.
- Annually draw down the seepage pond to observe seepage inflows and any sand deltas that may be hidden within the pond.

- Complete semi-annual surveys (late spring and late autumn) to identify settlement and deformation of the dam. The precision of these surveys should be improved to at least ±10 mm for horizontal movement and ±5 mm for vertical movement.

7.2.2 Extreme Event Studies

As indicated by the conclusions of this study, some action is required to either upgrade the tailings dam to handle the design seismic event and the PMF flood event or to decommission the facility. It is understood that a decision to either upgrade the facility to an acceptable closure condition or to fully decommission the facility will be implemented by 2005. Nevertheless, according to the former operating water licence for this facility, the current shutdown phase of the mine should now be considered a permanent closure. Therefore, a decision as to the future of the facility may be necessary prior to 2005. In order to facilitate this decision the following assessments are recommended:

- Complete a probabilistic risk assessment to identify risk levels associated with the occurrence of extreme events over the implementation time frame(s) that may wish to be considered. Such a study would assist in identifying how quickly action should be taken to mitigate these risks.

- Complete a cost benefit analysis comparing the possible upgrading costs to the decommissioning costs. The cost benefit analysis would require the consideration, at a conceptual level, of options to both decommission the facility and to upgrade the facility.

It is EBA's opinion that both of these assessments need to be completed; however, Water Resources may elect to complete either or both.

8.0 CLOSURE

EBA trusts that this report meets with your approval. Please do not hesitate to contact the undersigned should you have any questions or comments.

Respectfully submitted,
EBA Engineering Consultants Ltd.

Guoxi Wu

June 14/2002

Guoxi Wu, PhD. P.Eng. (BC)
Project Engineer
(Phone (604) 685-0275)
(e-mail:gwu@eba.ca)

Reviewed by:

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ENGINEER

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(Phone (250) 756-2256)
(e-mail: bpatrick@eba.ca)

CRH/crh

R03-00-14618damR7

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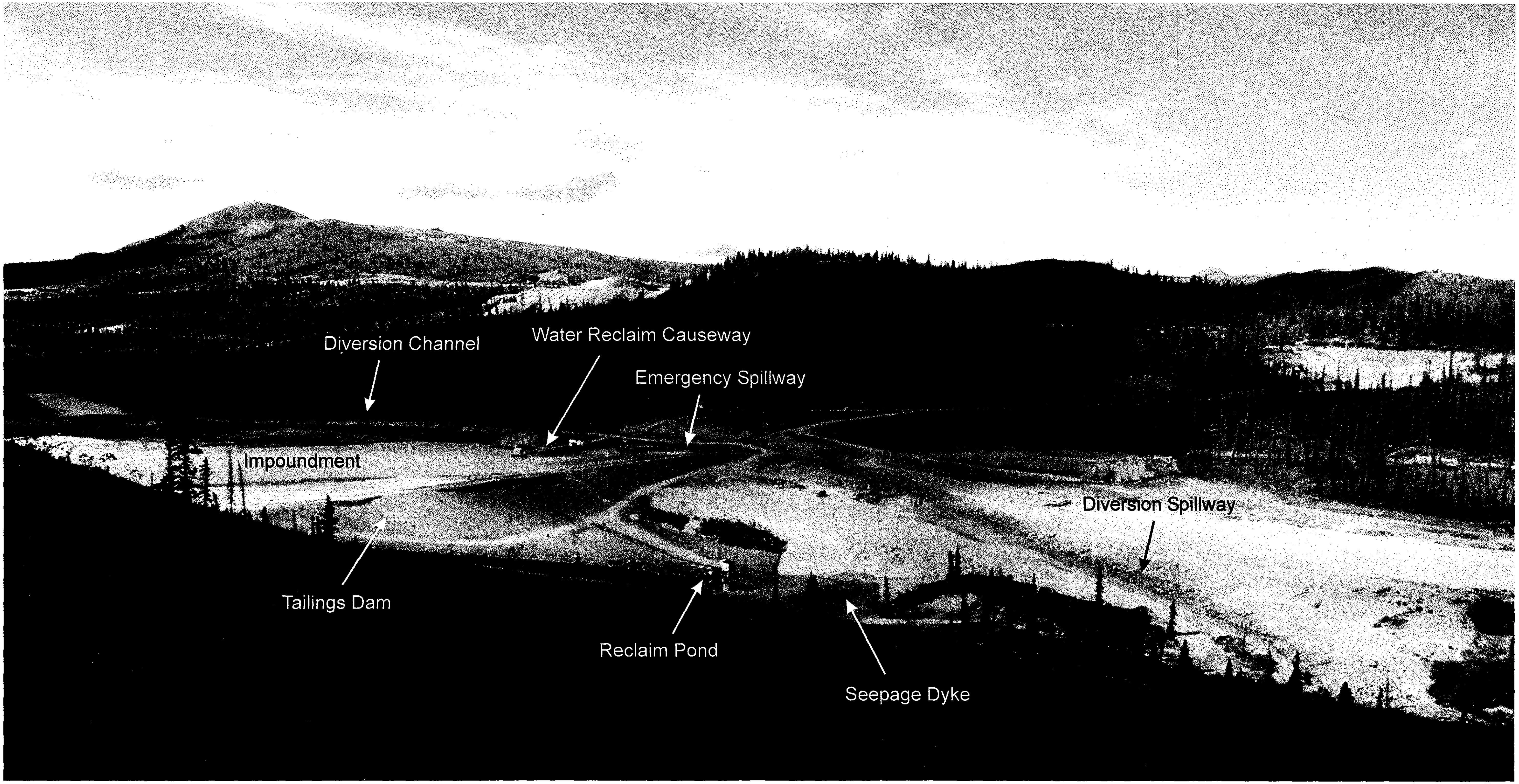
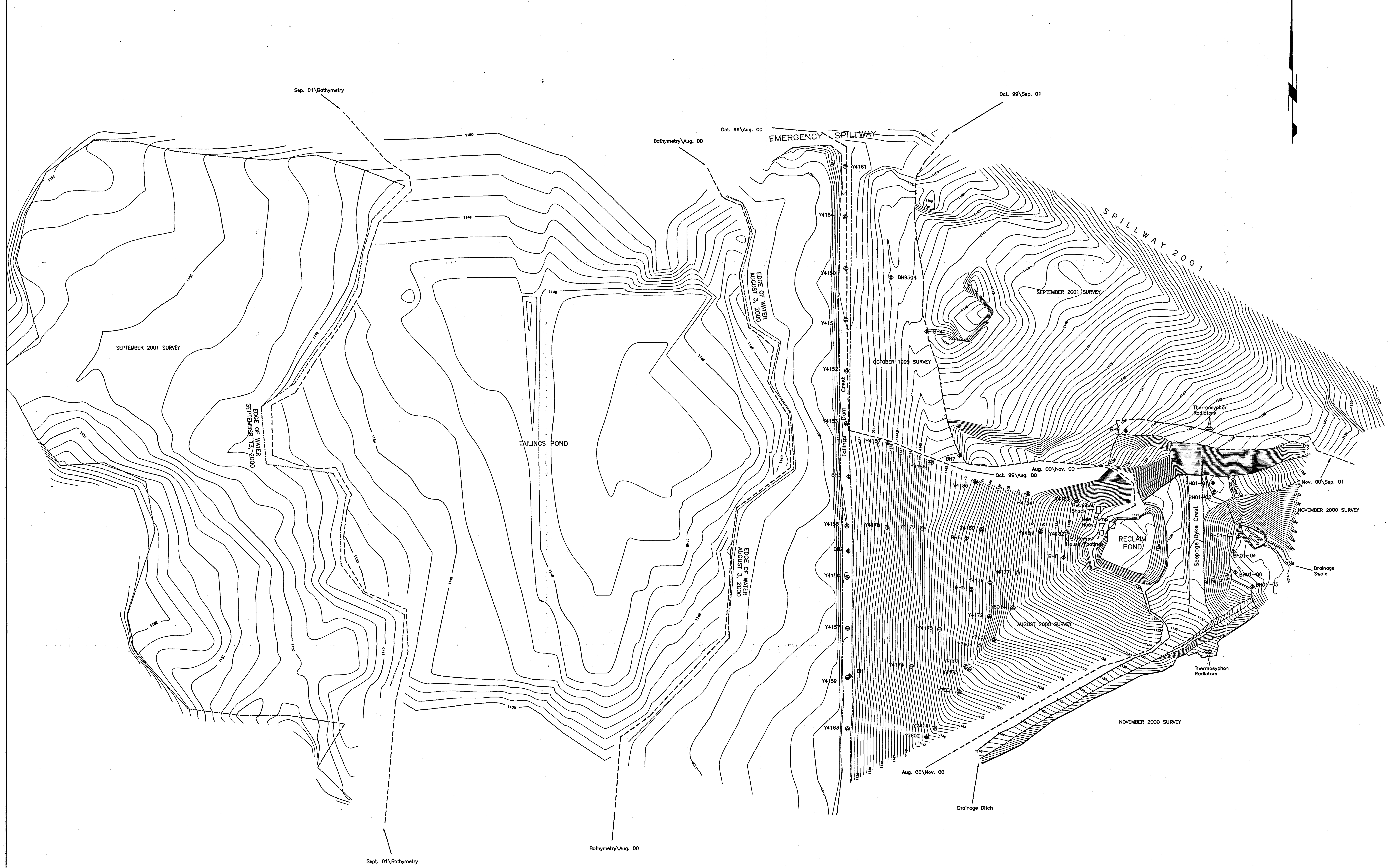


EBA Engineering Consultants Ltd.			PROJECT: MOUNT NANSEN DAM SAFETY ASSESSMENT		
CLIENT: DIAND			TITLE: TAILINGS IMPOUNDMENT FACILITY		
DATE: 01/04/18	DWN: JSB	CHKD: CRH	FILE NO.: 0201-00-14618	DWNG: FIGURE 1	REVISION:



SITE PLAN DRAWING

NOTES

1. ELEVATIONS DERIVED FROM MINE CONTROL #4001(1160.890)
2. COORD'S AND AZIMUTH DERIVED FROM POINT #4004 AND #1006
3. '00, '01 CONTOUR INTERVAL=0.25m
4. '99 CONTOUR INTERVAL=0.50m
5. DRAINAGE AND SEEPAGE POND ELEVATIONS AT THE TOE OF SLOPE ON TOP OF ICE.
6. TOP OF LINER ELEVATION WITHIN SEEPAGE DYKE IS 1130m.
7. THE CONTOURS FROM VARIOUS SURVEYS COMPLETED IN OCT. 1999, AUG. 2000 AND NOV. 2000 WILL BE SLIGHTLY DIFFERENT DUE TO THE NEW CONSTRUCTION AND CHANGED GROUND CONDITIONS.
8. SEEPAGE DYKE ASBUILT SURVEY COMPLETED ON NOV. 14, 2000.
9. MAIN DAM AND POND MODEL MERGED WITH EXISTING SEEPAGE DYKE DRAWING.
10. CONTOURS OF TAILINGS POND WERE DERIVED FROM DIGITIZING EXISTING CONTOUR PLAN PRODUCED BY LABERGE ENVIROMENTAL SERVICES.

LEGEND

- Bore\Drill Holes
- Survey Control
- Thermosyphons
- Top of Dyke
- Drainage Ditch
- Edge of RipRap
- Top of Bank
- Edge of Water
- Survey Limits From Different Dates

DATE	DESCRIPTION	BY	APP'D
NOV '01	ISSUED FOR REVIEW	JMG	

REVISIONS & ISSUES

DRAWN: AWG
CHECKED: CRH

SCALE: HOR: 1 : 750

Yukon Engineering Services

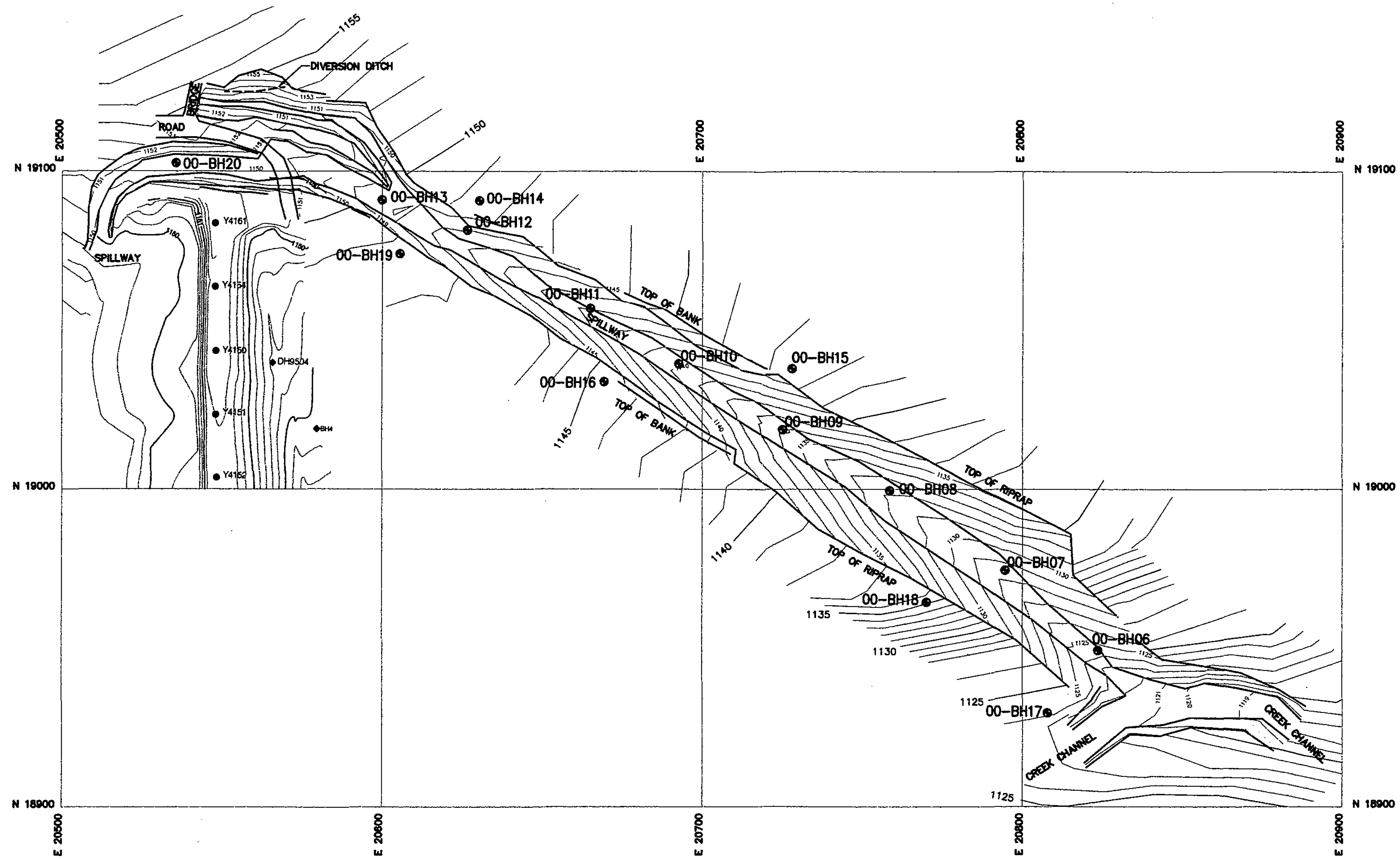
EBA Engineering Consultants Ltd.

FIGURE 2
2001 AS-BUILT PLAN
Mt. NANSEN TAILINGS
IMPOUNDMENT

SHEET 1 OF 1

CONSULTANT DWG. NO.: 99075\overall_mas.dwg
REV.: 0

EBA Engineering Consultants Ltd.			PROJECT MT. NANSEN DAM SAFETY ASSESSMENT		
CLIENT DIAND			TITLE SITE PLAN SHOWING TESTHOLE LOCATIONS		
DATE APRIL 2002	DWN. JSB	CHKD. CRH	FILE NO. 0201-00-14618	DRWG. FIGURE 3	

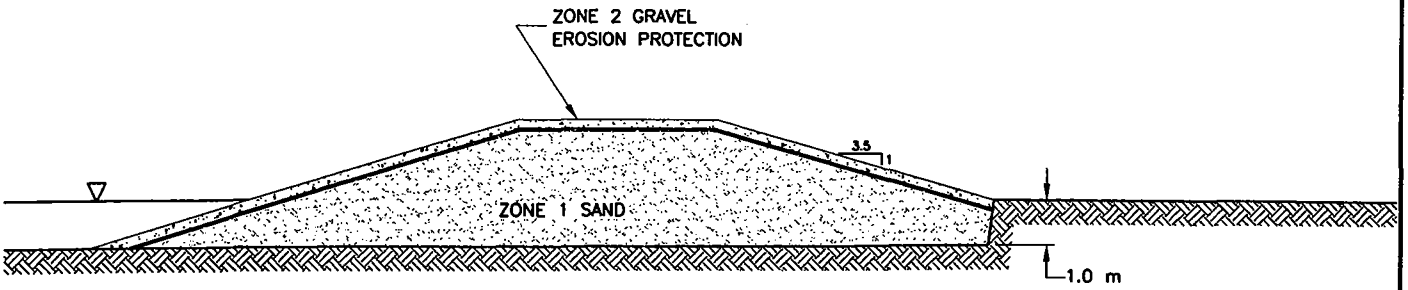


NOTES:
1. ELEVATION CONTOURS IN METRES.
2. GRID IS LOCAL MINE GRID.
3. CONTOURS BASED ON AUGUST, 2000 SURVEY, BY YUKON ENGINEERING SERVICES.

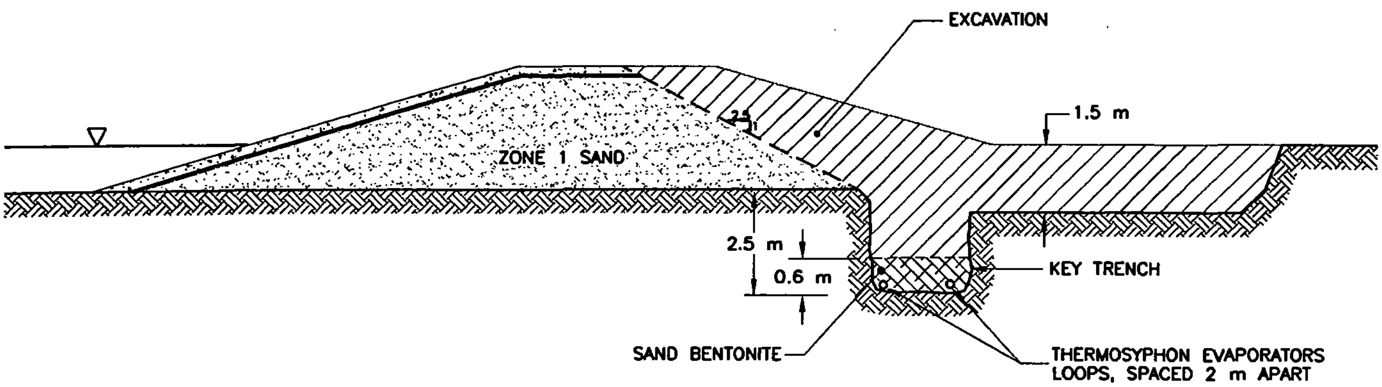
0 50 100

SCALE 1:2000

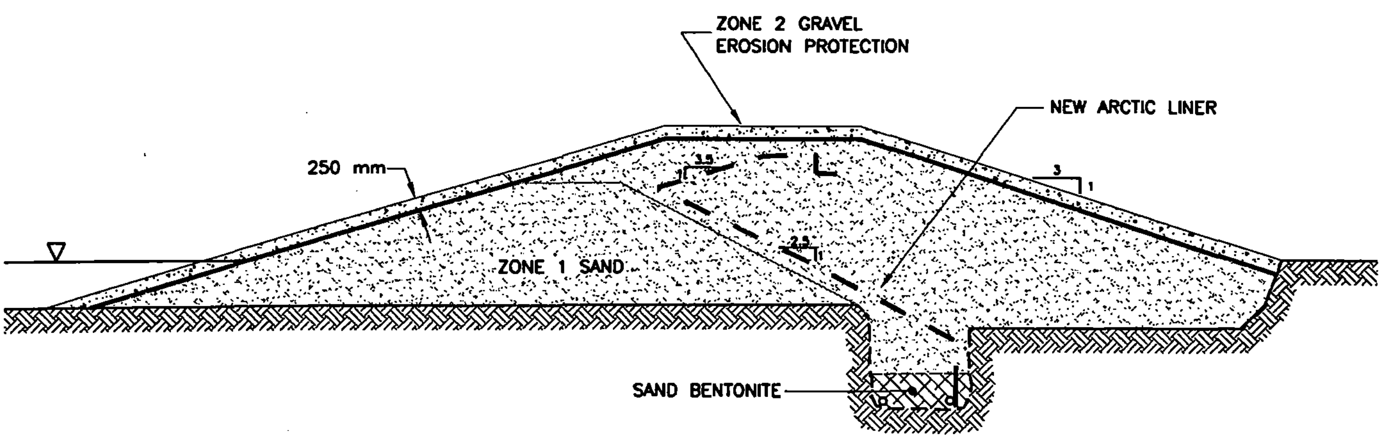
EBA Engineering Consultants Ltd.						PROJECT	MT. NANSEN DAM SAFETY ASSESSMENT
CLIENT	DIAND					TITLE	SPILLWAY PLAN AND TESTHOLE LOCATION PLAN
DATE	APRIL 2002	DWN.	JSB	CHKD.	CRH	FILE NO. 0201-00-14618	DRWG. FIGURE 4



EXISTING DAM



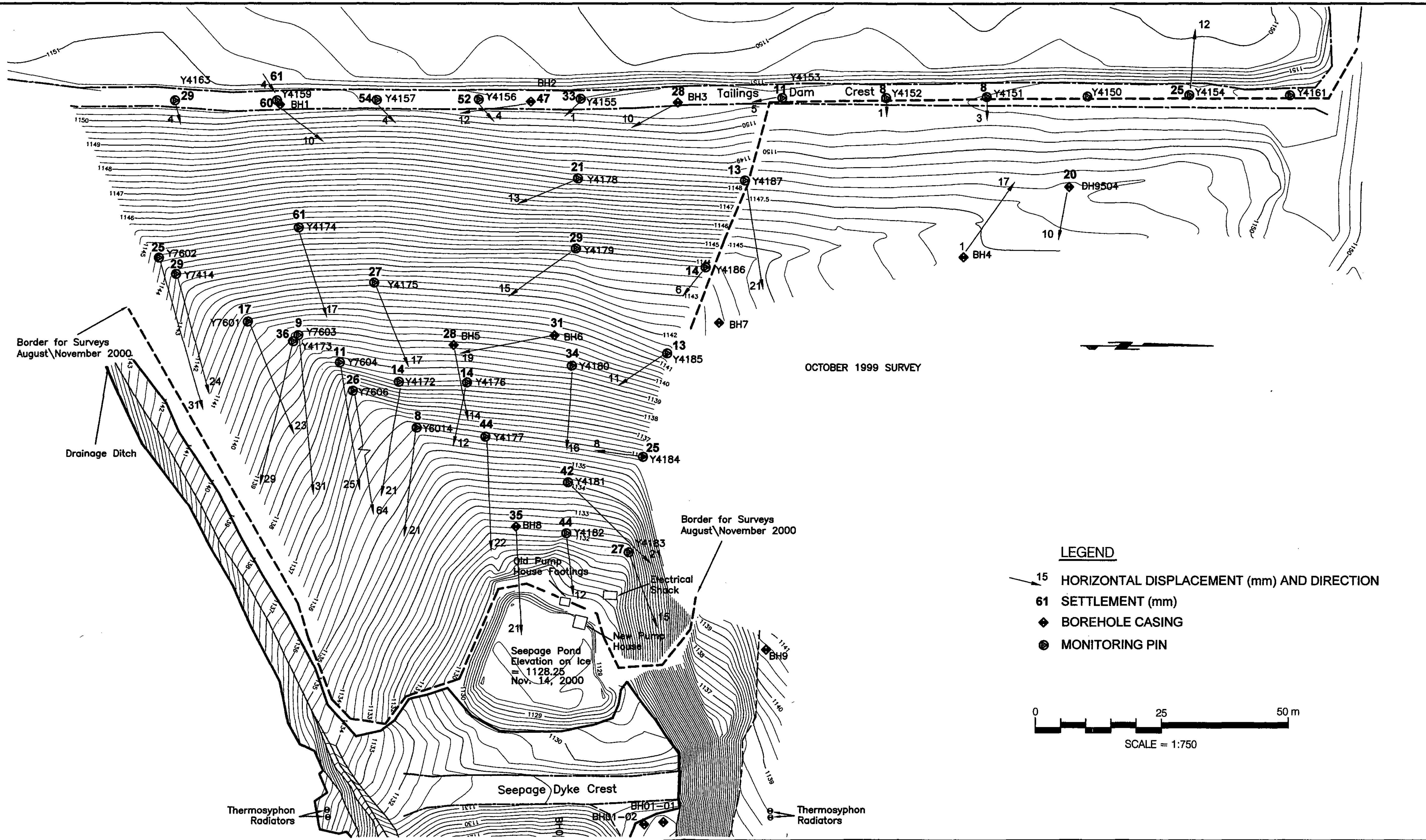
REMEDIATION PLAN STEP 1



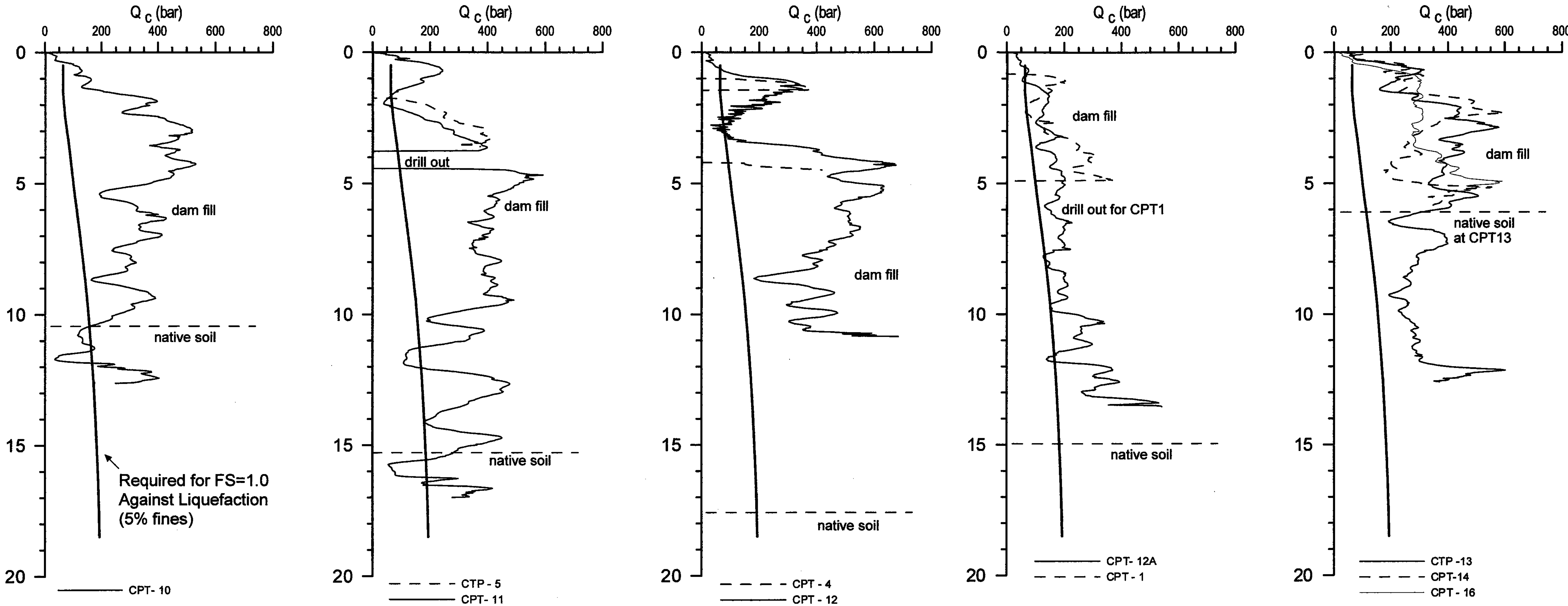
REMEDIATION PLAN STEP 2

SCALE = 1:200

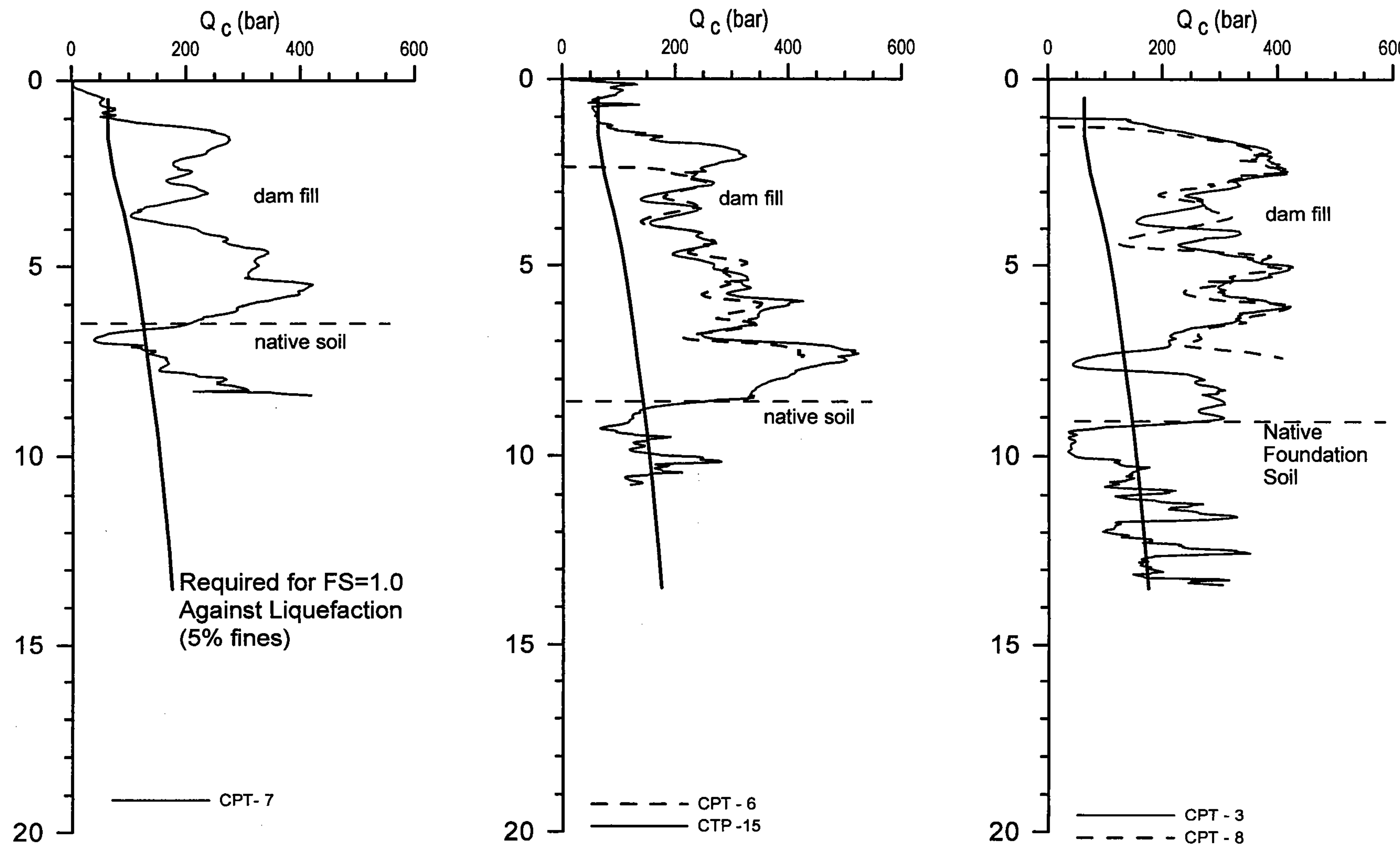
EBA Engineering Consultants Ltd.	PROJECT MT. NANSEN DAM SAFETY ASSESSMENT
CLIENT DIAND NORTHERN AFFAIRS PROGRAM	TITLE SEEPAGE CONTROL DYKE DESIGN CROSS SECTION
DATE APRIL 2002 | DWN. JSB | CHKD. CRH	FILE NO. 0201-00-14618 | DRWG. FIGURE 5



EBA Engineering Consultants Ltd.					PROJECT MT. NANSEN DAM SAFETY ASSESSMENT	
CLIENT DIAND					TITLE DAM MOVEMENT SURVEY RESULTS (1999 - 2001)	
DATE APRIL 2002		DWN. JSB		CHKD. CRH	FILE NO. 0201-00-14618	DRWG. FIGURE 6



EBA Engineering Consultants Ltd.		PROJECT:	Mt. Nansen Dam Safety Assessment
CLIENT:	DIAND	TITLE:	Soil Liquefaction Assessment Across the Dam Crest
DATE: 22/12/2000	DWN.: CHKD: GW	FILE NO.: 0201-00-14618	FIGURE 7



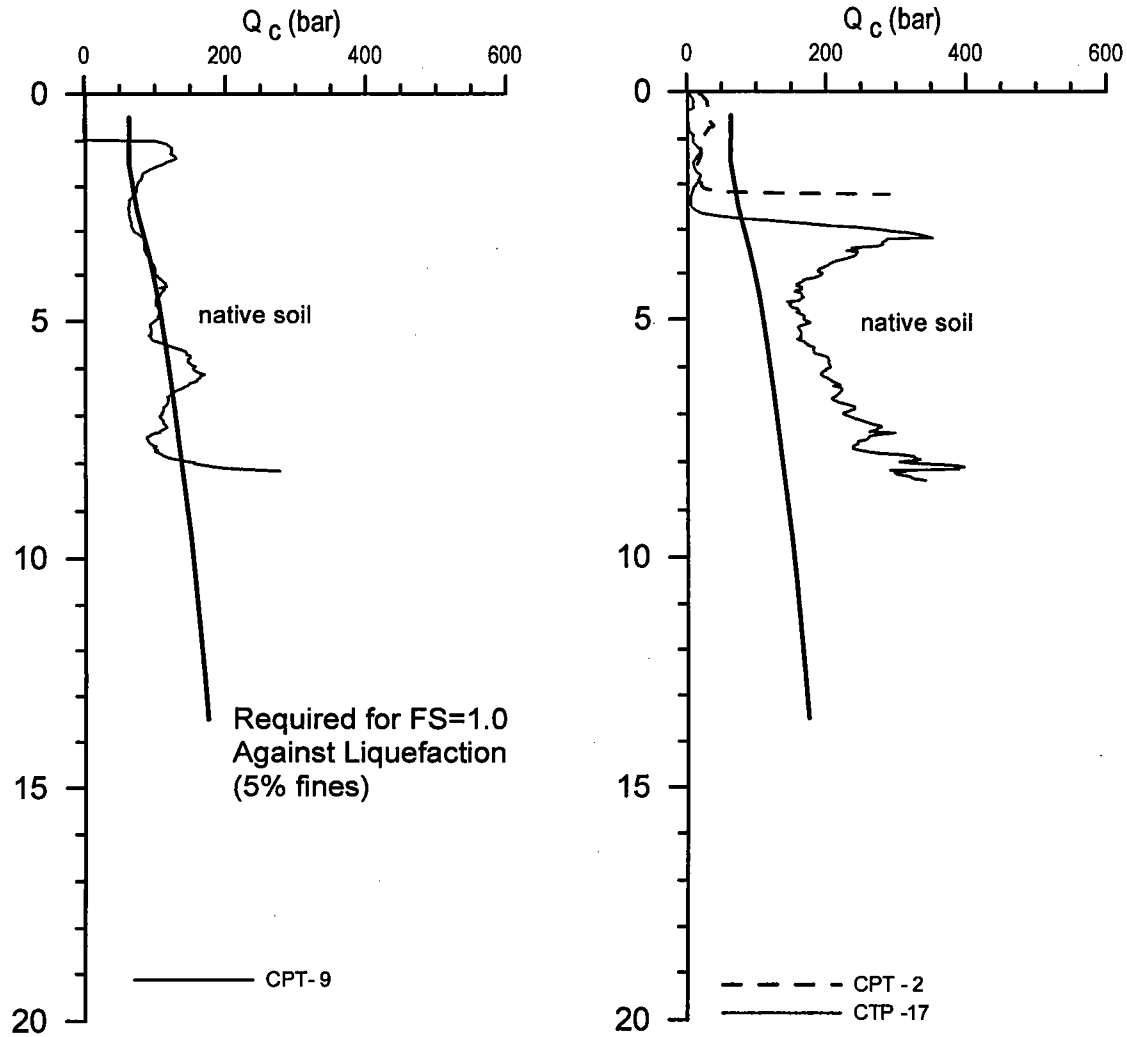
EBA Engineering Consultants Ltd.

PROJECT: Mt. Nansen
Tailings Dam Assessment

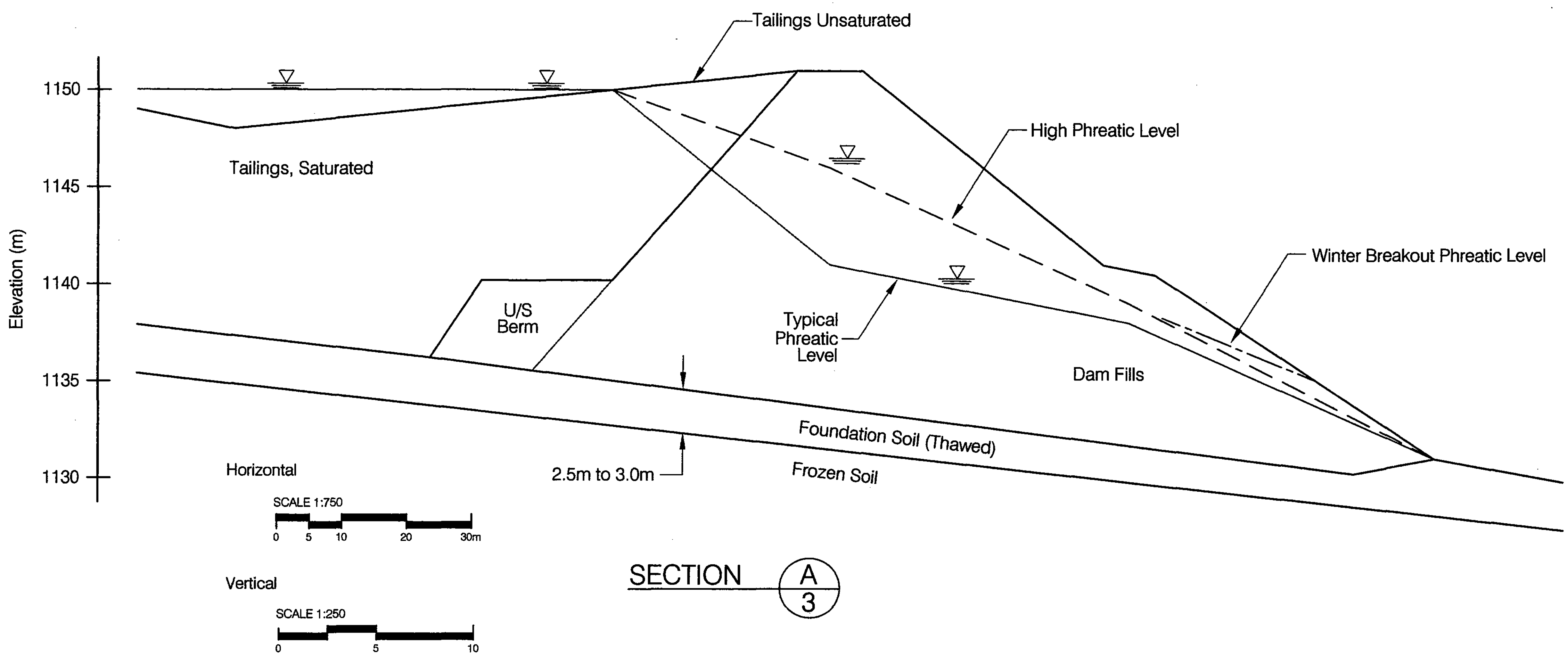
CLIENT: DIAND

TITLE: Soil Liquefaction Assessment
At the Toe Berm

DATE:	22/12/2000	DWN.:	CHKD:	GW	FILE NO.:	0201-00-14618	FIGURE 8



EBA Engineering Consultants Ltd.		PROJECT: Mt. Nansen Tailings Dam Assessment	
CLIENT: DIAND		TITLE: Soil Liquefaction Assessment At the North Native Ground	
DATE: 22/12/2000	DWN.:	CHKD: GW	FILE NO.: 0201-00-14618	FIGURE 9



SECTION A 3

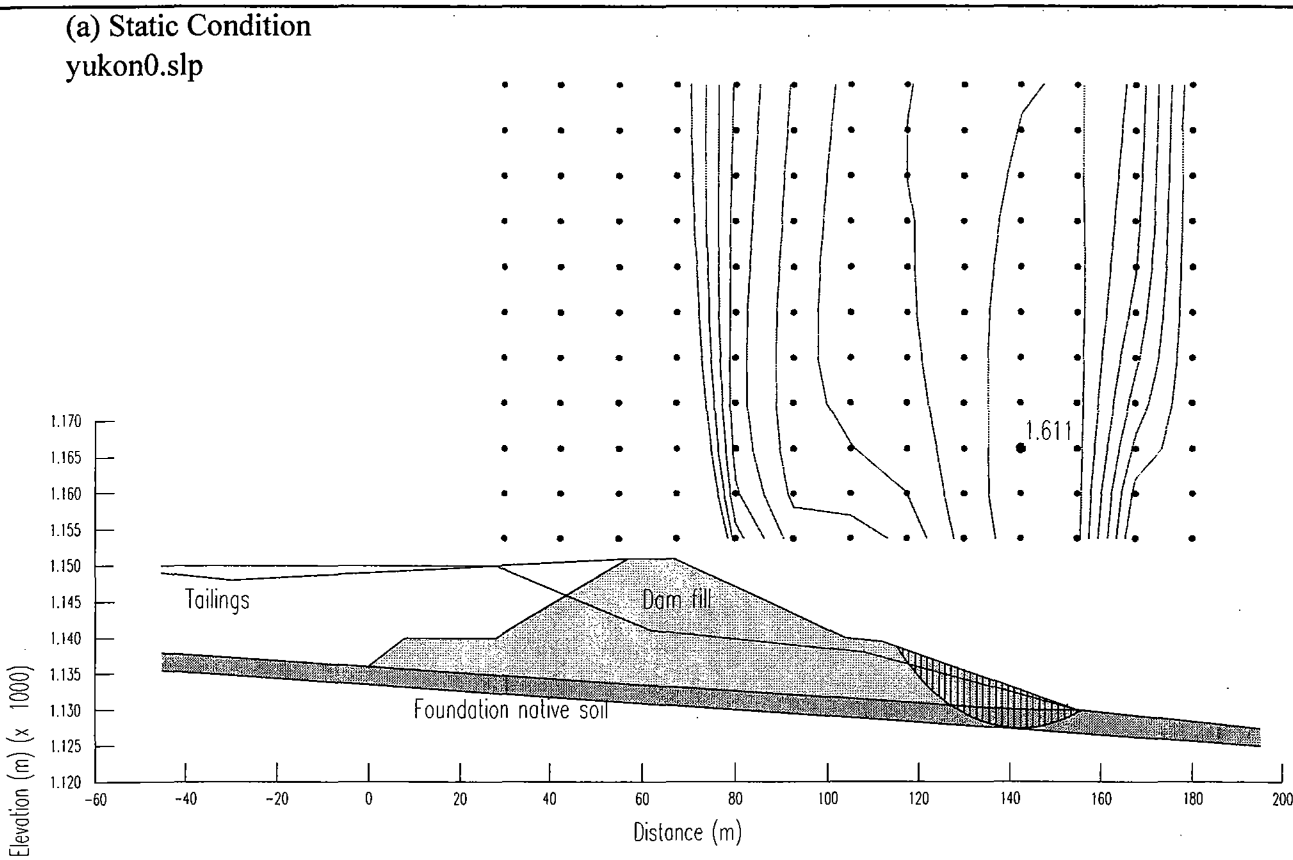
SUMMARY OF SOIL PARAMETERS USED IN LIMIT EQUILIBRIUM ANALYSES

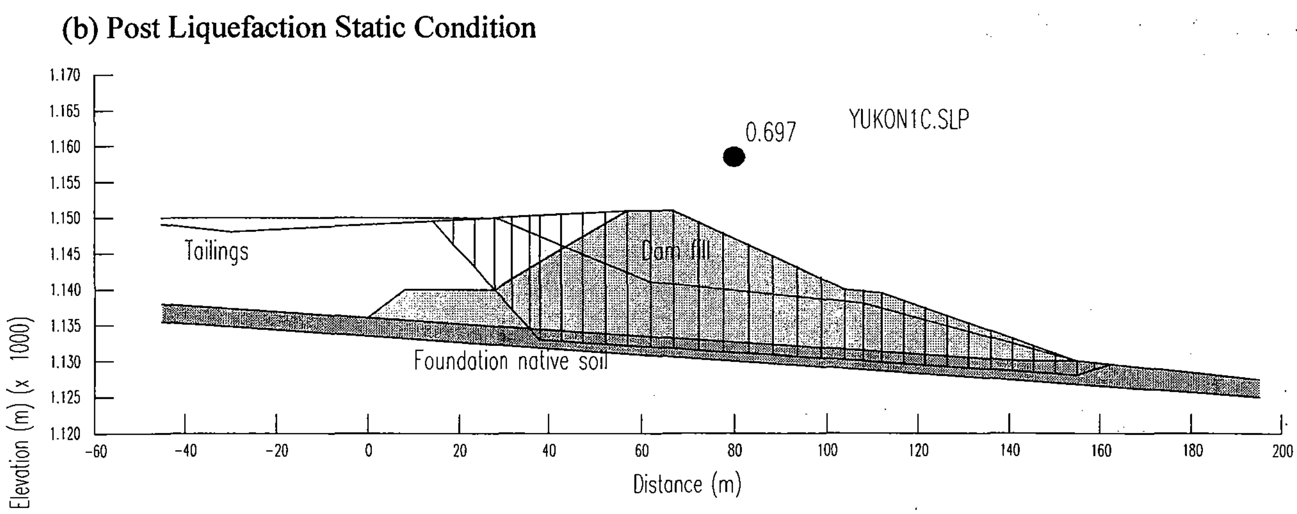
Soil Units	Bulk Unit Weight (kN/m3)	Frictional Strength (degree)	Cohesion (kPa)	Foundation Ru Value	Residual Strength (kPa)
Tailings	18.6	28	0		9.0
Compacted Dam Fill	19.5	34	0		
Native Foundation Soil	19.0	28 - 30	0	0.10	14.4

SUMMARY OF FACTORS OF SAFETY FROM STABILITY ANALYSES

Year	Static Minimum Factor of Safety	Post Liquefaction Static Factor of Safety	Post Liquefaction Pseudo Static Factor of Safety (0.27g)
Current Year 2000	1.36 - 1.61	0.70	<< 0.7
5 Year Window Year 2005	1.23 - 1.59	0.70	<< 0.7

EBA Engineering Consultants Ltd.		PROJECT	MT. NANSEN DAM SAFETY ASSESSMENT
CLIENT	DIAND	TITLE	RESULTS OF SLOPE STABILITY ANALYSIS
DATE	APRIL 2002	FILE NO.	0201-00-14618
DWN.	JSB	DRWG.	FIGURE 10
CHKD.	CRH		

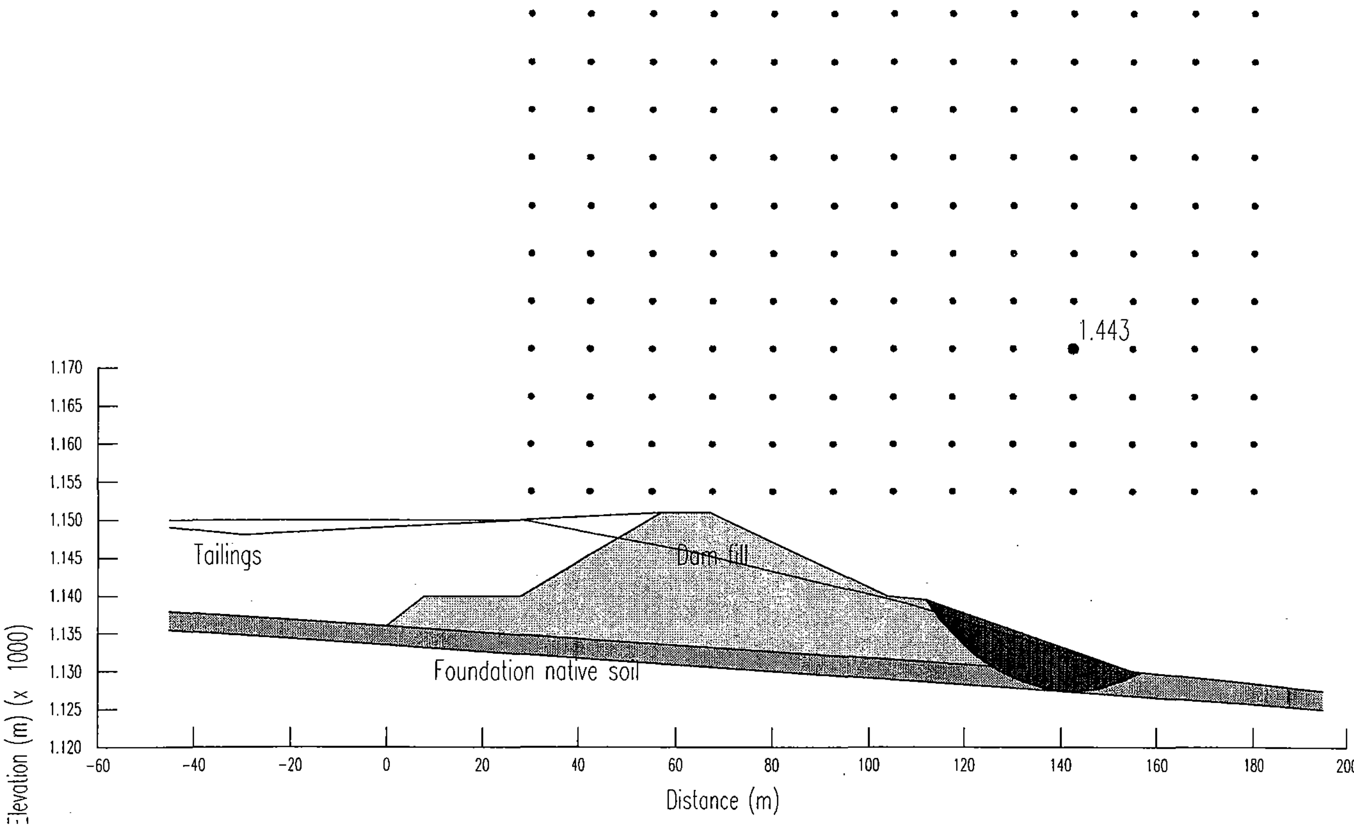




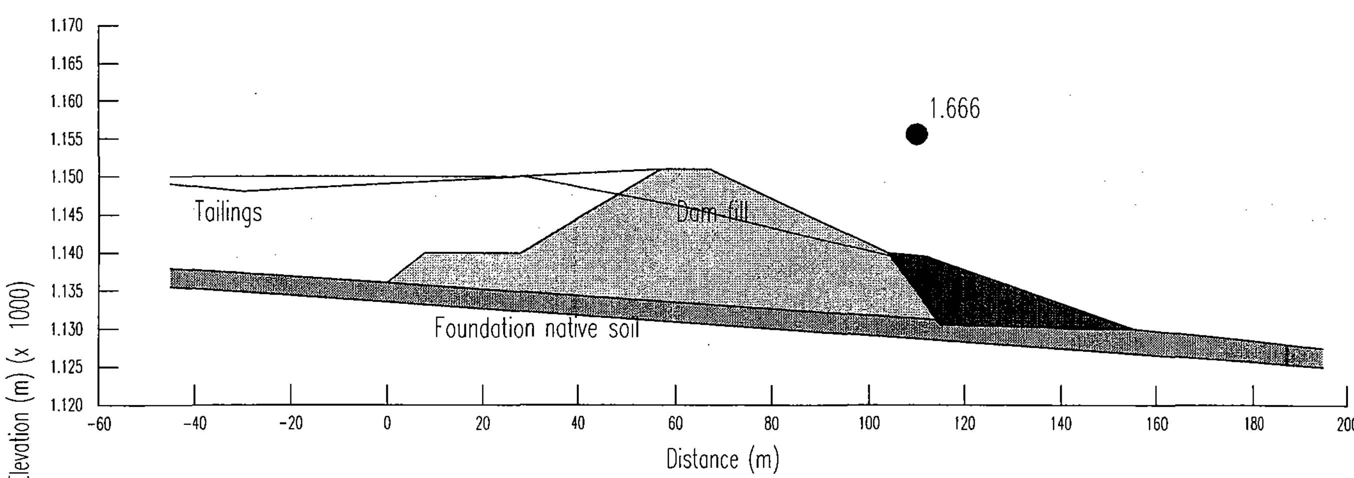
EBA Engineering Consultants Ltd.	PROJECT: Mt. Nansen Dam Safety Assessment
CLIENT: DIAND	TITLE: Stability Analyses Failure Modes

DATE: 22/12/2000	DWN.:	CHKD.: GW	FILE NO.:	0201-00-14618	FIGURE 11

(a) Static Condition (high phreatic level)
yukon0b.slp



(b) Static Condition (high phreatic level) - specified non-circular slip surface
yukon0I.slp



EBA Engineering Consultants Ltd.		PROJECT: Mt. Nansen Dam Safety Assessment	
CLIENT: DIAND		TITLE: Stability Analyses Parametric Analysis	
DATE: 28/03/2001	DWN.:	CHKD.: GW	FILE NO.: 0201-00-14618	FIGURE 12

EBA Engineering Consultants Ltd. (EBA)
GEOTECHNICAL REPORT - GENERAL CONDITIONS

This report incorporates and is subject to these "General Conditions"

A.1 USE OF REPORT AND OWNERSHIP

This geotechnical report pertains to a specific site, a specific development, and a specific scope of work. It is not applicable to any other sites nor should it be relied upon for types of development other than that to which it refers. Any variation from the site or development would necessitate a supplementary geotechnical assessment.

This report and the recommendations contained in it are intended for the sole use of EBA's client. EBA does not accept any responsibility for the accuracy of any of the data, the analyses or the recommendations contained or referenced in the report when the report is used or relied upon by any party other than EBA's client unless otherwise authorized in writing by EBA. Any unauthorized use of the report is at the sole risk of the user.

This report is subject to copyright and shall not be reproduced either wholly or in part without the prior, written permission of EBA. Additional copies of the report, if required, may be obtained upon request.

A.2 NATURE AND EXACTNESS OF SOIL AND ROCK DESCRIPTIONS

Classification and identification of soils and rocks are based upon commonly accepted systems and methods employed in professional geotechnical practice. This report contains descriptions of the systems and methods used. Where deviations from the system or method prevail, they are specifically mentioned.

Classification and identification of geological units are judgmental in nature as to both type and condition. EBA does not warrant conditions represented herein as exact, but infers accuracy only to the extent that is common in practice.

Where subsurface conditions encountered during development are different from those described in this report, qualified geotechnical personnel should revisit the site and review recommendations in light of the actual conditions encountered.

A.3 LOGS OF TEST HOLES

The test hole logs are a compilation of conditions and classification of soils and rocks as obtained from field observations and laboratory testing of selected samples. Soil and rock zones have been interpreted. Change from one geological zone to the other, indicated on the logs as a distinct line, can be, in fact, transitional. The extent of transition is interpretive. Any circumstance which requires precise definition of soil or rock zone transition elevations may require further investigation and review.

A.4 STRATIGRAPHIC AND GEOLOGICAL INFORMATION

The stratigraphic and geological information indicated on drawings contained in this report are inferred from logs of test holes and/or soil/rock exposures. Stratigraphy is known only at the locations of the test hole or exposure. Actual geology and stratigraphy between test holes and/or exposures may vary from that shown on these drawings. Natural variations in geological conditions are inherent and are a function of the historic environment. EBA does not represent the conditions illustrated as exact but recognizes that variations will exist. Where knowledge of more precise locations of geological units is necessary, additional investigation and review may be necessary.

A.5 SURFACE WATER AND GROUNDWATER CONDITIONS

Surface and groundwater conditions mentioned in this report are those observed at the times recorded in the report. These conditions vary with geological detail between observation sites; annual, seasonal and special meteorologic conditions; and with development activity. Interpretation of water conditions from observations and records is judgmental and constitutes an evaluation of circumstances as influenced by geology, meteorology and development activity. Deviations from these observations may occur during the course of development activities.

A.6 PROTECTION OF EXPOSED GROUND

Excavation and construction operations expose geological materials to climatic elements (freeze/thaw, wet/dry) and/or mechanical disturbance which can cause severe deterioration. Unless otherwise specifically indicated in this report, the walls and floors of excavations must be protected from the elements, particularly moisture, desiccation, frost action and construction traffic.

A.7 SUPPORT OF ADJACENT GROUND AND STRUCTURES

Unless otherwise specifically advised, support of ground and structures adjacent to the anticipated construction and preservation of adjacent ground and structures from the adverse impact of construction activity is required.

A.8 INFLUENCE OF CONSTRUCTION ACTIVITY

There is a direct correlation between construction activity and structural performance of adjacent buildings and other installations. The influence of all anticipated construction activities should be considered by the contractor, owner, architect and prime engineer in consultation with a geotechnical engineer when the final design and construction techniques are known.

A.9 OBSERVATIONS DURING CONSTRUCTION

Because of the nature of geological deposits, the judgmental nature of geotechnical engineering, as well the potential of adverse circumstances arising from construction activity, observations during site preparation, excavation and construction should be carried out by a geotechnical engineer. These observations may then serve as the basis for confirmation and/or alteration of geotechnical recommendations or design guidelines presented herein.

A.10 DRAINAGE SYSTEMS

Where temporary or permanent drainage systems are installed within or around a structure, the systems which will be installed must protect the structure from loss of ground due to internal erosion and must be designed so as to assure continued performance of the drains. Specific design detail of such systems should be developed or reviewed by the geotechnical engineer. Unless otherwise specified, it is a condition of this report that effective temporary and permanent drainage systems are required and that they must be considered in relation to project purpose and function.

A.11 BEARING CAPACITY

Design bearing capacities, loads and allowable stresses quoted in this report relate to a specific soil or rock type and condition. Construction activity and environmental circumstances can materially change the condition of soil or rock. The elevation at which a soil or rock type occurs is variable. It is a requirement of this report that structural elements be founded in and/or upon geological materials of the type and in the condition assumed. Sufficient observations should be made by qualified geotechnical personnel during construction to assure that the soil and/or rock conditions assumed in this report in fact exist at the site.

A.12 SAMPLES

EBA will retain all soil and rock samples for 30 days after this report is issued. Further storage or transfer of samples can be made at the client's expense upon written request, otherwise samples will be discarded.

A.13 STANDARD OF CARE

Services performed by EBA for this report have been conducted in a manner consistent with the level of skill ordinarily exercised by members of the profession currently practising under similar conditions in the jurisdiction in which the services are provided. Engineering judgement has been applied in developing the conclusions and/or recommendations provided in this report. No warranty or guarantee, express or implied, is made concerning the test results, comments, recommendations, or any other portion of this report.

A.14 ENVIRONMENTAL AND REGULATORY ISSUES

Unless stipulated in the report, EBA has not been retained to investigate, address or consider and has not investigated, addressed or considered any environmental or regulatory issues associated with development on the subject site.

APPENDIX B

Spillway Assessment
(B.K. Hydrology Services Ltd.)

April 16, 2001

EBA Engineering Consultants Ltd.
Calcite Business Centre, Unit 6
151 Industrial Road
Whitehorse, Yukon
Y1A 2V3

Attention: Cord Hamilton, M.Eng., P.Eng.

Re: **Mount Nansen Gold Mine**
Spillway Assessment

This letter provides information on site observations, design calculations and proposed spillway modifications. This information is to be used to plan the modifications to the spillway channel to be constructed later this fall. The probable life of this proposed spillway channel is about five years. Within this five-year period, decisions will be made to either upgrade the spillway channel to handle the Probable Maximum Flood event or the tailings and tailings dam will be removed eliminating the need for the spillway channel. The information is provided in point form.

1. The diversion channel located upstream of the spillway has partially silted in with fine sands and silts. This material has washed off the exposed slopes draining to the diversion channel. The original design indicated that the diversion channel would be lined with a 300 mm layer of gravel consisting of 75 mm minus pitrun gravel. It is likely that this layer is still in place under the sand and silt.

2. Grass seeds should be broadcast on all exposed sand and silt material. The vegetation should help stabilize this material and reduce the wash-off of this material into the diversion channel. Seeding can be carried out till mid-September with a second seeding next spring.

3. The drainage area to the downstream end of the diversion channel is estimated at 3.07 km^2. The estimated 1:20 year maximum instantaneous peak flow in the diversion channel is 1.2 m^3/s. The estimated 1:200 year maximum instantaneous peak flow in the diversion channel is 3.0 m^3/s. Further details on the hydrology calculations and comparison to previous peak flow estimates are listed in the Appendix.

BK Hydrology Service
5610 - 56A Street, Beaumont, Alberta, T4X 1A7
Phone (780) 929 8325 Fax (780) 929 5985

4. Flow depths and velocities within the diversion channel were computed based on the following:

   a. Channel slope of between 0.1% and 0.2%
   b. Trapezoidal channel shape with a 2 metre bottom, 3H:1V side slopes and 1.5 metres deep
   c. Manning's n = 0.030

   For the 1:20 year peak flow of 1.2 m^3/s, flow depths are between 0.6 and 0.7 metres and velocities vary between 0.6 and 0.8 m/s. For the 1:200 year peak flow of 3.0 m^3/s, flow depths are between 0.8 and 1.0 metres and velocities vary between 0.8 and 1.0 m/s. The sand and silt within the channel will wash away when velocities are greater than about 0.3 to 0.5 m/s. The 75 mm minus pitrun gravel layer should withstand velocities of about 1.2 m/s. Therefore, the diversion channel should be able to handle the 1:200 year peak flow as long as the gravel layer as shown in the original design drawings is in place.

5. At present, the riprap material within the spillway varies from gravel to 500 mm rock. Along most of the spillway, the larger riprap material is near the top of the spillway side slope and located one or more metres above the bottom of the spillway channel. At the bottom of the spillway channel, almost all of the rock is less than 100 mm in size. At some locations, only gravel size material is located within the channel bottom.

6. An August 2000 survey of the spillway channel is attached. Information from this plan includes the following:

   a. The slope of the spillway channel varies between 7% and 15% with the majority of the channel length between 8% and 12%.
   b. The channel bottom width varies between 5 metres to 10 metres except near at the exit of the spillway. About 5 metres upstream of where the spillway channel enters the natural creek channel, the spillway channel narrows down from a width of 5 metres to a width of about 3 metres. This narrow section of channel is about 9 metres in length.
   c. At the top of the spillway, the channel bottom is around elevation 1150 metres and at the bottom around elevation 1122 metres.
   d. The spillway channel length is about 315 metres.
   e. The spillway channel side slopes are about 3H:1V.
   f. The depth of the channel varies from about 1.5 metres to 5.0 metres

7. Normal flow depths and velocities with the spillway channel were computed based on the following:

   a. Channel slope of 7%, 10% and 15 %
   b. Trapezoidal shape with 5 metre bottom and 3H:1V side slopes. Flow depths and velocities would be less for a wider channel bottom.
   c. Manning's n = 0.060 (set high to account for high channel roughness in relation to channel flow depths)

   Table 1 below lists flow depths and average velocities for a range of spillway flows. For a flow of less than 0.1 m^3/s, the sand and silt material within the spilllway channel would be washed away. Starting at a flow of about 0.6 m^3/s, the gravel within the channel bottom will start to move within the steeper sections of the spillway. At present, much of the spillway channel bottom is covered by gravel. Therefore, the present spillway could start to fail at a flow of about 0.6 m^3/s. For the 1:200 year maximum instantaneous peak flow of 3.0 m^3/s, the flow depth is about 0.24 metres and the flow velocity is about 2.2 m/s. Because of the shallow flow depth, localized velocities at small obstructions in the flow could be as high as 3.0 m/s or more.

Table 1
Spillway Flow Depths and Velocities

Flow (m^3/s)	7% Slope		10% slope		15% slope	
	Depth (m)	Velocity (m/s)	Depth (m)	Velocity (m/s)	Depth (m)	Velocity (m/s)
0.1	0.04	0.5	0.04	0.6	0.03	0.6
0.6	0.11	1.0	0.10	1.1	0.09	1.2
1.2	0.17	1.3	0.15	1.4	0.14	1.6
3.0	0.29	1.8	0.27	2.0	0.24	2.3

8. Based on the above, the present spillway channel can probably handle a peak flow of about 0.6 m^3/s. The channel needs to be upgraded to handle the 1:200 year peak flow of 3.0 m^3/s. Two options were considered. The first option is to line the entire spillway channel with a 400 mm (7% to 9% slope) to 500 mm (greater than 9% slope) median size rock riprap. This rock riprap layer would be about 1.0 metre thick. The second option is to use drop structures along the spillway channel to reduce the flow velocities within the spillway. Considering

the various size ranges of available rock and the cost associated with each option, the second option was chosen.

The following modifications to the spillway channel based on the second option are recommended:

a. A gravel layer is present along most of the spillway channel but the thickness of this layer is not known. The gravel layer should be a minimum 300 mm in depth and is required as a filter layer between the rock riprap and the underlying sands and silts. The actual thickness of the gravel layer should be confirmed in the field using a small backhoe or power auger. Additional gravel should be brought in where the depth of gravel is less than 300 mm. Small erosion gullies (less than 200 mm in depth) in the bottom of the spillway channel should also be filled in with gravel.

b. There is a significant quantity of larger size rock (200 mm or greater in size) located along the upper slopes of the spillway channel. All rock located more than about 1.5 metres (equal to the computed design high water level plus a 0.5 metre freeboard) above the bottom of the spillway channel should be moved down. This rock should be placed on top of the gravel layer. The rock riprap layer should be about 0.6 metre thick.

c. If not enough rock is available on the upper channel slopes, additional rock should be brought from the rock stockpile. This rock should be durable and of good quality. The median size rock should be about 300 mm, 80% of the rock should be 200 mm in size or greater and all the rock should be less than 450 mm in size (Class I riprap specification).

d. 14 rock drop structures should be constructed along the spillway. The first drop structure would be placed at elevation 1148 metres and the last drop structure would be placed at elevation 1122 metres. Drop structures would be placed at each 2 metre drop in the bottom of the spillway channel (1148 m, 1146 m, 1144 m, ... 1124 m, and 1122 m). Spacing between the drop structure is between 15 and 25 metres depending on the local channel slope. Schematics of the drop structure layout are attached. These drop structures should minimize the extent of any localized failure within the spillway. For the 1:200 peak flow, maximum depths upstream of the drop structure will be about 1.0 metre.

e. The narrow 3.0 metre wide channel bottom near the spillway exit should be increased to a minimum 5.0 metre width. The channel section is about 9.0 metres in length. There is adequate space available to widen the channel to a 5 metre width.

9. If the diversion channel was breached (not expected to occur), the flows would enter the tailing dam reservoir. Routing the runoff through the reservoir would reduce the peak flows to about 20% of the peak flow in the diversion channel (1:20 year peak flow of about 0.24 m^3/s and 1:200 year peak flow of about 0.6 m^3/s). These relatively low flows can be handled by the existing spillway channel located upstream of the intesection of the diversion channel and the spillway.

If you have any questions about the above material, please give me a call.

Sincerely,

Bernie Kallenbach

Bernie Kallenbach, M.Eng., P.Eng.
President

January 4, 2002

Our File: 157

Your File:

EBA Engineering Consultants Ltd.
Calcite Business Centre, Unit 6
151 Industrial Road
Whitehorse, Yukon
Y1A 2V3

Attention: Cord Hamilton, M.Eng., P.Eng.

Re: **Mount Nansen Gold Mine**
2001 Spillway Assessment

This letter provides information on site observations, design calculations and proposed spillway modifications. This information is to be used to plan additional modifications to the spillway channel to be constructed within 2002. The probable life of this proposed spillway channel is about five years. Within this five-year period, decisions will be made to either upgrade the spillway channel to handle the Probable Maximum Flood event or the tailings and tailings dam will be removed eliminating the need for the spillway channel. The information is provided in point form.

1. A site visit was conducted by Bernie Kallenbach of BK Hydrology Service on October 5, 2001. A visual assessment was made of the spillway channel and photos were taken of the constructed drop structures. Additional survey data obtained during the summer 2001 was provided by EBA Engineering Consultants Ltd.

2. A previous letter by BK Hydrology Service dated April 16, 2001 provided estimates of the flood peaks for the spillway channel. The estimated 1:20 year maximum instantaneous peak flow in the spillway is 1.2 m^3/s. The estimated 1:200 year maximum instantaneous peak flow in the spillway is 3.0 m^3/s. Further details on the hydrology calculations are available in the April 16, 2001 letter.

3. The April 16, 2001 letter also suggested seeding exposed sand and silt areas with grass. The vegetation should help stabilize this material and reduce the wash-off of this material into the diversion channel. A grass vegetation cover has not been established and it does not appear that this seeding was carried out. It is recommended that grass seed be broadcast in the spring.

BK Hydrology Service
5610 - 56A Street, Beaumont, Alberta, T4X 1A7
Phone (780) 929 8325 Fax (780) 929 5985

4. Based on the 2001 survey information, the bottom width of the spillway varies between 6.7 metres and 12.8 metres. The spillway side slopes vary between 3.2H:1V and 1.8H:1V. The channel slope between drop structures varies between 5.7% and 11.0%. Riprap coverage extends 1.2 or more metres in height on the spillway side slopes.

5. Normal flow depths and velocities with the spillway channel were computed based on the following:

   a. Three spillway segments were chosen to represent the range of channel slopes between the spillway drop structures. The channel characteristics for these three segments is listed in Table 1.

Table 1
Spillway Channel Segments

Location between Drop Structures	Channel Slope (%)	Channel Width (m)	Side Slope (H:V)
2-3	5.7	8.8	3.2:1
1-2*	7.4	6.7	2.4:1
12-13	11.0	8.5	1.8:1

* Note that drop structure 1 was not constructed

   b. Manning's n = 0.060 (set high to account for high channel roughness in relation to channel flow depths)

   c. Trapezoidal channel shape using the channel characteristics listed in Table 1.

Table 2 lists flow depths and average velocities for the three spillway segments. For a flow of less than 0.1 m^3/s, the sand and silt material within the spillway channel would be washed away. Starting at a flow of about 1.2 m^3/s, the gravel within the channel bottom will start to move. At present, some segments of the spillway channel bottom have significant amounts of gravel exposed beneath the rock riprap coverage. Therefore, the present spillway could start to fail at a flow of about 1.2 m^3/s. The drop structures would probably restrict the failure to local scour between drop structures. However, if severe scour occurs, the drop structures may become undermined and fail leading to extensive scour damage of the spillway. For the 1:200 year maximum instantaneous peak flow of 3.0 m^3/s, the maximum flow depth is about 0.25 metres and the flow velocity is about 1.8 m/s. Because of the shallow flow depth, localized velocities at small obstructions in the flow could be as high as 2.5 m/s or more.

Table 2
Spillway Flow Depths and Velocities

Flow (m^3/s)	5.7% Slope		7.4% slope		11% slope	
	Depth (m)	Velocity (m/s)	Depth (m)	Velocity (m/s)	Depth (m)	Velocity (m/s)
0.1	0.03	0.4	0.03	0.5	0.03	0.5
0.6	0.09	0.8	0.09	0.9	0.08	0.9
1.2	0.13	1.0	0.14	1.2	0.11	1.2
3.0	0.23	1.4	0.25	1.7	0.19	1.8

It should also be noted that for the computed shallow flow depths, individual rocks in the channel bottom are the same order in size as the channel flow depth. The channel bottom is also not perfectly horizontal and may vary by up to 0.2 metres from over the width of the channel bottom. Therefore, the computed flow depths and velocities only provide a rough estimate of the actual local flow depths and velocities.

8. The April 16, 2001 letter recommended that 14 drop structures be built along the spillway channel. The drop structures are numbered from drop structure 1 at the top of the spillway to drop structure 14 at the bottom. An assessment of the drop structures is listed in Table 3.

Table 3
Assessment of Drop Structures

Drop Structure	Assessment
1	Drop structure was not constructed. Drop structure should be constructed as recommended in April 16, 2001 letter.
2	The middle portion of the drop structure needs more rock.
3	The middle portion of the drop structure needs more rock.
4	Several rocks needed to fill in gap in the middle.
5	Several rocks needed to fill in gap in the middle.
6	Drop structure is adequate.
7	Need one or two rocks to fill gap in the middle.
8	Drop structure is adequate.
9	Drop structure is adequate.
10	Several rocks needed to fill in gap in the middle.
11	It appears that middle rocks shifted downstream during a high flow event. Leave existing rocks in place and add new rocks along center line of drop structure.
12	It appears that middle rocks shifted downstream during a high flow event. Leave existing rocks in place and add new rocks along center line of drop structure. Add two large rocks on north bank of drop structure.
13	Rocks in the middle are too small. Replace small rocks with larger rocks and place small rocks downstream of drop structure.
14	Fill in a few gaps near the north bank. Extend 300 mm rock riprap about 5 metres downstream of the present rock coverage.

9. Between the drop structures, the spillway was to be lined with a double layer of 300 mm median size rock. The observed coverage in the field is a single layer of rock which covers between 50% and 80% of the channel bottom. Also, the north bank coverage downstream of drop structure 8 is poor. Additional rock riprap should be placed to provide the recommended double layer of rocks.

If you have any questions about the above material, please give me a call.

Sincerely,

PROFESSIONAL ENGINEER ALBERTA
BERND KALLENBACH

Bernie Kallenbach, M.Eng., P.Eng.
President

April 24, 2002

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Appendix A

Hydrology Calculations

The August 10, 1995 report by Klohn Crippen estimated the peak flows in the diversion channel and the spillway channel. The report provides a table of intensity-duration rainfall values for the 1:20 year, 1:200 year and PMP event. This rainfall data was used to prepare 6 hour duration design storms (design storm values not listed). The report also indicates that the OTTHYMO program was used to determine peak flows using this rainfall information. However, the report does not list any of the runoff parameters used within the OTTHYMO program to compute the peak flows.

The Klohn Crippen report estimated the 1:20 year peak flow at 6.4 m^3/s for both the diversion channel and the spillway. The 1:200 year flood was expected to breach the diversion channel and divert flows through the tailing dam reservoir. Routing the flood peak through the tailing dam reservoir would significantly reduce the flood peak entering the spillway. The peak outflow from the reservoir during the 1:200 year event was estimated at 3.7 m^3/s. Since this was less than the 1:20 year peak flow, the spillway channel was designed to handle the 1:20 year peak flow of 6.4 m^3/s.

My peak flow estimates are considerably less than those produced by Klohn Crippen. My 1:20 year peak flow is estimated at 1.2 m^3/s (about 19% of the Klohn Crippen peak). A direct comparison cannot be made of the 1:200 year event since Klohn Crippen did not list the peak flow that would enter the tailings dam reservoir.

I've computed the peak flow using the SWMM program. Parameters used for the modelling include:

1. I developed a Chicago Distribution storm using the Table 3 intensity-duration rainfall values listed in the Klohn Crippen report. The design storm is of two hour duration, 5 minute time steps, and the peak rainfall occurs at 40 minutes from the start of the storm event. Total rainfall for the 2 hour design storm is 21.7 mm for the 1:20 year event and 30.0 mm for the 1:200 year event. I didn't use a six hour storm like Klohn Crippen since the difference in the 2 hour and 6 hour rainfall amounts was less than less than 6 mm for the 1:20 and 1:200 year event. This 6 mm of rainfall spread over a 4 hour period would not have a significant impact on peak flow estimates.

2. I divided the basin into three subbasins varying in size from 100 ha to 107 ha. Basin slopes varied between 20% and 25%. Maximum overland flow lengths were set at 500 metres. The percent imperviousness of each basin was set 0.5% (a 5 metre wide strip to account for the stream channel). The depression storage for pervious areas was set at 3.0 mm. The Horton infiltration values were set at 25 mm/hr for initial infiltration capacity, 2.5 mm/hr

for final infiltration capacity and 4 hr^{-1} for the decay coefficient. For overland flow routing a Manning's n was set at 0.90 to account for the rough terrain and vegetation cover.

3. The subbasin flows were routed through trapezoidal channels to model the flow within the creek. The channel slopes varied from 8% to 12%. The diversion channel was also modelled with a channel slope of 0.1%.

4. Model results indicate that the peak outflow from the subbasins occurs about 20 minutes after the peak rainfall. The outflow from the basin drops to about 10% of the peak flow about 4 hours after the start of the storm event.

As an independent check, I computed the 1:20 and 1:200 peak flows using the procedure listed in the report Design Flood Estimating Guidelines for the Yukon Territory by J.R. Janowicz, 1989. This report provides procedures for estimating the 1:2 to 1:100 year peak flows. I extrapolated the curve values on log-frequency paper to estimate the 1:200 year values. The 1:20 year peak flow estimate is 0.96 m^3/s and the 1:200 year peak flow estimate is 2.2 m^3/s. These values are between 20% and 27% lower than my estimates listed above. This would indicate that my flow estimates are slightly conservative.

As another independent check, I transferred the 1:200 year peak flow estimate from the Big Creek Water Survey of Canada station to our basin using the following formula:

$$Q = Q(Big) * (A(site)/A(Big))^{0.8}$$

$$Q = 350\ m^3/s * (3.07/1750)^{0.8}$$

$$Q = 2.2\ m^3/s$$

This again would indicate that my peak flow estimate of 3.0 m^3/s for the 1:200 year event is slightly conservative.

APPENDIX C

Foundation Thaw Assessment

C1.0 INTRODUCTION

Foundation thaw below the Mount Nansen tailings dam was assessed by thermal analyses and by reviewing CPT profiles and measured ground temperature data. The objective of this assessment was to evaluate the existing thermal behaviour of the tailings dam at *the section of greatest dam height* and to predict the foundation thaw at that section in five years. The results are to be used in the short-term stability assessment of the tailings dam. This memorandum describes the evaluation methodology and summarizes the results of the findings.

C2.0 MEASURED GROUND TEMPERATURE DATA

EBA has been monitoring temperature data from the dam fill and underlying foundation at select locations in the tailings dam since April 1998. As described in EBA's Geotechnical Data Review Report (1999), the general trend from most ground temperature data shows gradual warming and some level of permanent and/or seasonal thaw in the native permafrost soils. The greatest thaw was observed on the north abutment.

The ground temperature data from EBA Borehole 12861-02, drilled from the dam crest above the former Dome Creek, is representative of the temperature history at the maximum dam height and has been reviewed in detail.

Figure 1 shows the ground temperature history at this borehole location since April 1998. The period of record to date, approximately 2.6 years, indicates that the dam fill experienced anomalous warming during the winter of 1998/1999, but has otherwise shown gradual cooling. The temperature within the native organics has also cooled, but much more gradually. Below the native organics, the temperature history indicates a slight warming trend of the native foundation sand. Between late June 1999 and late November 2000, the native foundation sand has warmed by approximately 0.05°C. The mean ground temperature of the native foundation sand at Elevation 1126.1 m (approximately 6 m below the fill/native ground surface interface) is currently approximately –0.4°C. Since the dam was constructed in 1996, the fact that the measured temperatures in the native foundation have relatively stabilized indicates that most of the warming of the native foundation has already occurred. Figure 2 shows the temperature profile variations with time and confirms the cooling trend observed within the dam fill in the year 2000. The maximum thaw depth in the native foundation soil of 1.5 m was recorded in early October 1999.

C3.0 THERMAL ANALYSIS

A simple one-dimensional thermal model was developed to evaluate the thermal conditions within the dam fill and foundation. The dam profile was assumed to be fill sand overlying organics on top of native sand. The original ground temperature was assumed to be –1°C. It was assumed that 19 m of fill at a temperature of +3°C was added in September 1996. Snow on the dam crest was assumed to be wind-swept, so reduced snow cover was assumed in the thermal model. Otherwise, mean climatic conditions were applied at the dam crest. Heat transfer was assumed to be through conduction only; seepage effects were ignored.

The results of the thermal model showed that the heat load from the fill placement would initiate a thawing front into the organics and native sand. However, the fill would itself progressively cool from the dam crest and the thaw front would stabilize after a few years. The foundation thaw depth was predicted to increase by approximately 0.2 m for the first five years following dam construction, and then reduce by approximately 0.1 m over the subsequent five years. These trends generally agree with the observed trends, although the measured temperatures in the foundation are slightly colder than predicted and the measured temperatures in the dam fill below the phreatic surface are warmer than predicted. The results indicate that convective heat from seepage flow in the dam contributes to the warmer measured temperatures in the dam fill and that the thermal response of the foundation is likely influenced more from two-dimensional effects instead of the simple 1D model analyzed. Further analyses using a two-dimensional thermal model incorporating seepage in the dam fill and in the unfrozen foundation might provide a better prediction compared to observed temperature data, but is beyond the scope of work of this assessment.

C4.0 CONE PENETRATION TESTS

Measurements of tip resistance and pore pressure from cone penetration tests collected during ConeTec's 2000 field programs were used to interpret zones of unfrozen soil in the dam fill and underlying foundation. Foundation thaw depths were estimated to be between 1.5 m and 3.0 m (average approximately 2.5 m), but thaw depths up to 7.5 m were recorded on the north abutment, which was estimated to have warmer initial ground temperatures.

C5.0 FOUNDATION THAW DEPTH ESTIMATE

The results of the thermal analyses and the measured ground temperature data show that although seepage flow is warming the dam fill, it does not appear to have caused additional thaw of the native foundation soils. Seepage rates are small enough that the layer of organics near the dam fill/native ground surface interface appears to effectively insulate the underlying frozen sand from further thaw. Therefore, it appears that the current foundation thaw depth, over four years after dam construction, is at or near its maximum and should be relatively stable over the next five years.

Based on the results of the thermal analyses and cone penetration tests, in addition to ground temperature measurements collected since April, 1998, the current average foundation thaw depth is estimated to be 2.5 m at dam's section of maximum height. The current foundation thaw depth is at or near its maximum. In five years, the foundation thaw depth is conservatively estimated to be 2.8 m.

Reference

EBA Engineering Consultants Ltd., 1999. Geotechnical Data Review Report. Mount Nansen Tailings Dam Safety Evaluation. Report submitted to Water Resources Division, DIAND, Whitehorse, Yukon. File No. 0201-99-14108, December, 1999.

APPENDIX D

Seepage Evaluation

D1.0 INTRODUCTION

Seepage analyses were carried out for the tailings dam. The objectives of the analyses were two-fold:

1. To determine the effect of Klohn's (1995) selection of 1135 m as the downstream ground surface elevation on seepage rates, and
2. To back-calculate hydraulic properties based on measured piezometric levels and flow rates.

This appendix describes the results of the seepage analyses carried out using SEEP/W.

D2.0 KLOHN (1995) MODEL

Klohn (1995) carried out seepage analyses for foundation thaw depths ranging from 0 to 16 m. The mesh is incorrect because the downstream ground surface elevation is shown to be 1136 m when, in fact, the ground slopes at approximately 4.2 per cent grade and the downstream ground surface is actually 1130 m.

Klohn's estimated flow rates and the SEEP/W mesh used in the analysis are shown in Figure A5-1 of the 1995 design report. Total seepage was estimated assuming no clay liner and a 200 m wide seepage zone (corresponding to the width of the dam).

Hydraulic parameters used in the Klohn (1995) model are as follows:

Table 1. Hydraulic Parameters used in Klohn (1995) Seepage Analyses

Material	Hydraulic Conductivity (cm/s)
Frozen Sand	1×10^{-7}
Unfrozen Sand	1×10^{-3}
Dam Fill	1×10^{-3}
Tailings (Silt)	1×10^{-5}

The mesh shown in Figure A5-1 of the 1995 report was reproduced for this comparative analysis. The pond elevation was assumed to be 1149.7 m and the setback distance between the pond and the upstream dam was approximately 55 m. The foundation thaw depth was assumed to be 8 m.

The downstream ground surface elevation was assumed to be 1136 m (Klohn geometry) and 1130 m (new EBA geometry).

Figures 1 and 2 show the phreatic surfaces for the Klohn geometry and new EBA geometry, respectively. The seepage parameters below the dam crest were estimated as follows:

Table 2. Results of Seepage Analyses (Klohn Geometry)

Downstream Ground Elevation (m)	Phreatic Surface Elevation Below Dam Crest (m)	Total Flux (x 10-6 m/s)	Estimated Seepage Flow Rates (L/s) *
1136 (Klohn)	1138.7	5.16	1.03
1130 (EBA new)	1136.8	5.78	1.16

Note: seepage flow estimated by multiplying the predicted flux by the width of the dam (200 m) as done by Klohn (1995).

The results show that dropping the downstream ground elevation from 1136 m to 1130 m increased the predicted seepage flow rate from 1.03 L/s to 1.16 L/s, a 13 per cent increase. The predicted seepage rate for Klohn's geometry of 1.03 L/s is lower than that reported in Klohn (1995) for 8 m thaw depth (1.5 L/s).

D3.0 BACK-CALCULATION OF SEEPAGE PARAMETERS

EBA has installed several piezometers along various locations of the tailings dam, including along the dam crest and at the downstream toe berm. Two-dimensional seepage analyses were carried out using the existing dam geometry at the maximum dam height cross-section. This was the same section used in the stability analyses.

The dam crest was assumed to be 10 m wide and at an elevation of 1151 m. The downstream toe berm was assumed to be 8 m wide at approximately elevation 1140 m. The upstream toe berm was assumed to be 10 m wide at approximately 1140 m. The natural ground slope of the valley bottom was assumed to be 4.2 per cent. The permeability of the Geosynthetic Clay Liner (GCL) was ignored.

Across this section, piezometric levels are available at the dam crest and at the downstream toe berm. These levels vary with time, depending on the pond elevation. For simplicity, the pond elevation was assumed to be 1150 m and the piezometric levels below the dam crest and below the downstream toe berm were assumed to be 1141 m and 1138 m, respectivley. These are average values, and were the same values used in the stability assessment.

Cone Penetration tests carried out in 2000 showed variable thaw depths of the native foundation soils (typically ranging from 1.5 m to 3 m). For the analysis, the foundation was assumed to be uniformly thawed 2.5 m, again the same as used in the stability assessment.

Parametric analyses were carried out varying the hydraulic conductivities of the tailings and the dam fill/foundation sand to match the measured piezometric levels. Figure 3 shows the predicted phreatic surface for the best-fit results. The seepage parameters that best-matched the measured piezometric levels are as follows:

Table 3. Seepage Parameters Estimated from Measured Piezometric Levels

Material	Hydraulic Conductivity (cm/s)
Unfrozen Foundation Sand	2.0×10^{-3}
Unfrozen Dam Fill (Sand)	1.8×10^{-3}
Tailings (silt)	4.0×10^{-5}

The predicted piezometric levels below the dam crest and downstream toe berm are approximately elevation 1141.4 m and 1137.3 m, respectively, which are within range of the measured values of 1141 m and 1138 m, respectively. The measured piezometric level below the toe berm is higher than the predicted value; this is expected because it is more likely to be influenced here by three-dimensional flow effects than below the dam crest. The setback distance between the pond and the upstream face of the tailings dam is approximately 15 m.

The predicted total flux across this section is approximately 1.87×10^{-5} m/s, which is more than three times the flux that would have been predicted for Klohn's mesh and soil properties for a downstream ground elevation of 1130 m (Figure 2). Below the dam crest, the average seepage velocity is approximately 1.6×10^{-6} m/s

The estimated flow through and below the dam is estimated to be 3.7 L/s, assuming a constant flux of 1.87×10^{-5} m/s multiplied by the width of the dam (200 m). This is in the higher end of the 2 to 4 L/s pump-back flow rate range currently measured on site. However, this is an inaccurate means of estimating flow rates, since the seepage rates would vary with the pressure head and depth of the unfrozen foundation.

The results of the analysis, which ignored the GCL, indicate that the GCL has not reduced seepage in the dam, although it may affect the time-dependent response of pore pressures within the dam. This is outside the scope of this assessment and has not been verified.

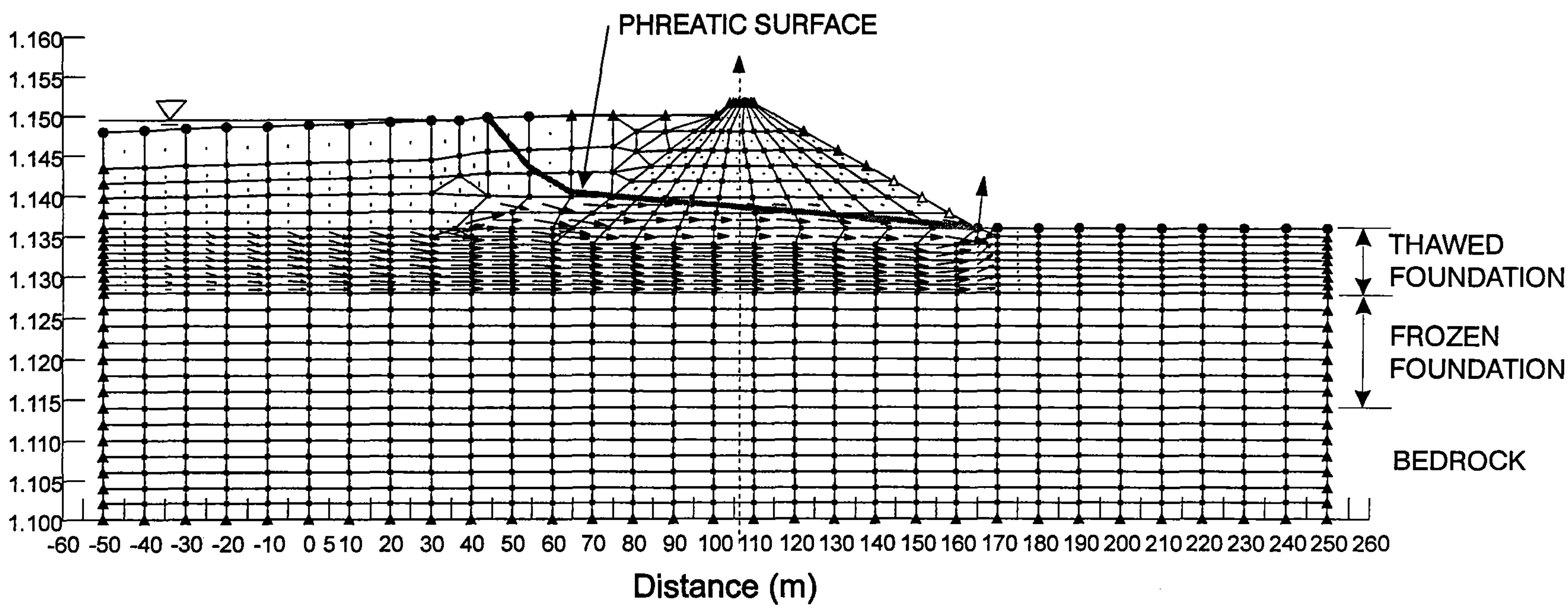
References

EBA Engineering Consultants Ltd., 1999. Geotechnical Data Review Report. Mount Nansen Tailings Dam Safety Evaluation. Report submitted to Water Resources Division, DIAND, Whitehorse, Yukon. File No. 0201-99-14108, December, 1999.

Klohn-Crippen, 1995. Mount Nansen Gold Project. Tailings Impoundment Final Design Report. Report submitted to BYG Natural Resources Inc. File No. PM 5314 0504. April, 1995.

201-14618 Mount Nansen Tailings Dam #1 Seepage Analysis
December 8, 2000
By: JTCS
File: DAM-1A.SEP
Pond @ El. 1149.7 m; Downstream W.L. @ El. 1136.0 m
Duplicate Klohn-Crippen (1995) seepage analysis, Figure A5-1
Soil properties same as assumed in Klohn Crippen (1995) report

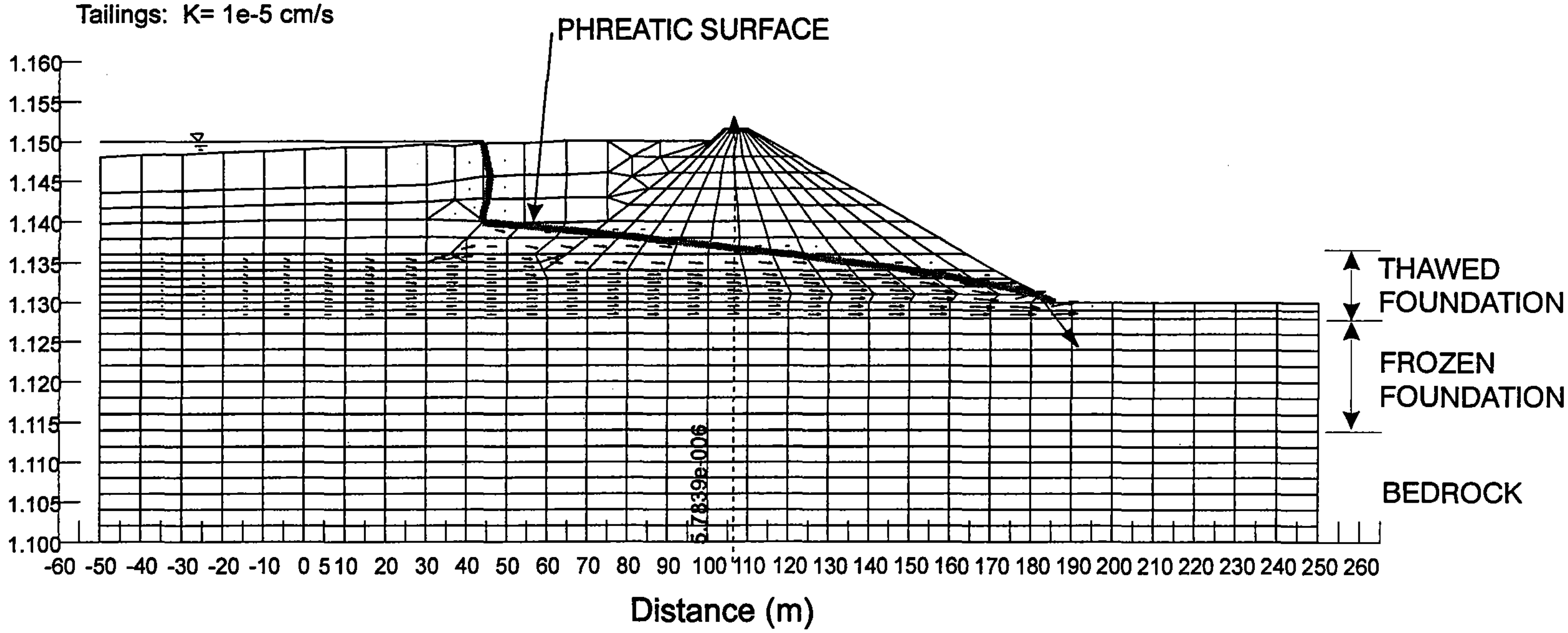
Unfrozen Foundation Sand: K = 1e-3 cm/s
Frozen Foundation Sand: K = 1e-7 cm/s
Dam Fill: K = 1e-3 cm/s
Tailings: K = 1e-5 cm/s



EBA Engineering Consultants Ltd.			PROJECT: MT. NANSEN TAILINGS DAM ASSESSMENT		
CLIENT: DIAND			TITLE: SEEPAGE ASSESSMENT, DESIGN MODEL		
DATE: 01/11/29	DWN: JPB	CHKD: CRH	FILE NO.: 0201-00-14618	DWNG: FIGURE 1	REVISION:

201-14618 Mount Nansen Tailings Dam #1 Seepage Analysis
December 8, 2000
By: JTCS
File: DAM-3A.SEP
Pond @ El. 1149.7 m; Downstream W.L. @ 1130.0 m
Re-run of Klohn-Crippen (1995) seepage analysis from design report
Downstream ground surface moved from 1136.0 m to 1130.0 m (actual)
Soil Properties same as assumed in Klohn Crippen (1995) report

Unfrozen Foundation Sand: K = 1e-3 cm/s
Frozen Foundation Sand: K = 1e-7 cm/s
Dam Fill: K = 1e-3 cm/s
Tailings: K= 1e-5 cm/s

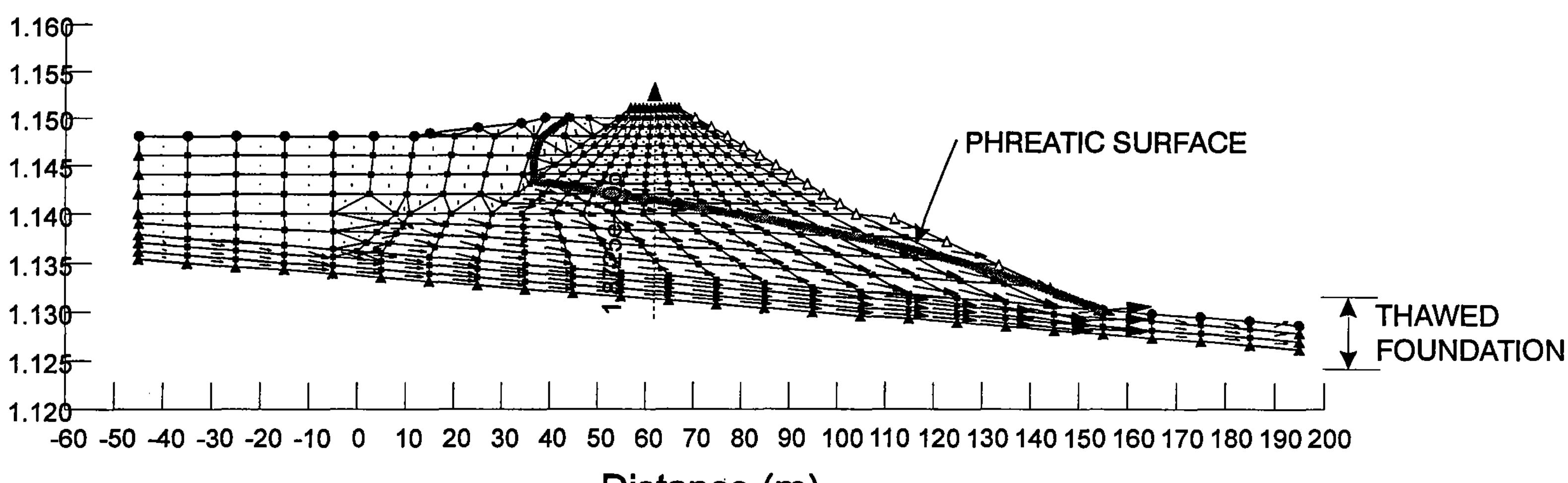


EBA Engineering Consultants Ltd.			PROJECT: MT. NANSEN TAILINGS DAM ASSESSMENT		
CLIENT: DIAND			TITLE: SEEPAGE ASSESSMENT, CORRECTED TOE ELEVATION		
DATE: 01/11/29	DWN: JPB	CHKD: CRH	FILE NO.: 0201-00-14618	DWNG: FIGURE 2	REVISION:

201-14618 Mount Nansen Tailings Dam #1 Seepage Analysis
December 11, 2000
By: JTCS
File: DAM-4G.SEP
Pond @ El. 1150 m; Downstream W.L. @ ground surface
Use same geometry as Guoxi's liquefaction analysis
Soil Properties same as assumed in Klohn Crippen (1995) report

Unfrozen Foundation Sand: K = 2e-3 cm/s
Frozen Foundation Sand: K = 1e-7 cm/s
Dam Fill: K =1.8e-3 cm/s
Tailings: K=4e-5 cm/s

Average Measured Piezometric Levels
Pond Level: 1150 m
below Dam Crest: 1141 m
below downstream toe berm: 1138 m



EBA Engineering Consultants Ltd.	PROJECT: MT. NANSEN TAILINGS DAM ASSESSMENT
CLIENT: DIAND	TITLE: SEEPAGE ASSESSMENT, AS-BUILT GEOMETRY

DATE: 01/11/29	DWN: JPB	CHKD: CRH	FILE NO.: 0201-00-14618	DWNG: FIGURE 3	REVISION:

APPENDIX E

Seismic Hazard Assessment

Seismic Hazard Analysis for Nansen Gold Mine Area

prepared by Alison Bird, Seismologist

What follows is the material required to perform an evaluation of the seismic stability of the tailings dam at the Nansen Gold Mine site. The seismic activity and major faults in the area are presented, along with a probabilistic and a deterministic analysis of associated seismic hazard.

It should be noted that neither site conditions nor structural engineering are considered in this analysis.

Tectonic Setting

Convergence between the Pacific and North American Plates, in the region of the Gulf of Alaska, produces crustal shortening and uplift which extends well into Yukon Territory and into most northwestern British Columbia (Figure 1). A great proportion of the relative motion between these plates occurs on the Queen Charlotte–Fairweather transpressional fault system and westward on the subduction zone along Alaska's south coast. Additional transform motion is accommodated along the Denali Fault, further inland in southwestern Yukon.

The Tintina Fault lies along the Tintina Trench which severs the Yukon Plateau. It is believed that this fault has been an active transform boundary for a geologically significant length of time.

Recorded Earthquakes

All seismicity which has been located in the region between 1980 and 2000 is shown in Figure 2, with all events larger than magniture 5 represented by a star. Figure 3 shows all located events with a magnitude of at least 5 which have occured since 1898 (when the first seismographs were installed in western North America). In both maps, the Nansen Gold Mine site is indicated with a red triangle.

Superimposed on the above maps are the primary local faults to which the deterministic analysis will be applied. The closest of these is the Denali transform Fault with the largest recorded earthquake a magnitude 6.2. Further to the southwest is the complex subduction–transform plate boundary, of which the Fairweather Fault is the surface trace of the transform component, with a magnitude 8.0 earthquake recorded in 1899. To the northeast is the Tintina Fault, with no earthquake recorded above magnitude 5.

Probabilistic Analysis

The probabilistic analysis of the Mount Nansen site (62.10°N, 137.31°W) is a standard procedure of the Geological Survey of Canada's Seismic Risk Calculation routine. The results from this are presented in Table 1.

If this data is presented in a log-log plot, it can easily be extrapolated to determine a peak ground acceleration (PGA) of 0.27 g and peak ground velocity (PGV) of 0.61 m/s, for a return period of 10,000 years (Figure 4).

Table 1: Results of probabilistic seismic hazard calculation.

Return Period (years)	100	200	475	1,000
Probability of Exceedence per annum/ Probabilité de depassement par année	0.010	0.005	0.0021	0.001
Probability of Exceedence in 50 years/ Probabilité de depassement en 50 ans	40%	22%	10%	5%
Peak Horizontal Ground Acceleration (g)	0.058	0.075	0.096	0.119
Peak Horizontal Ground Velocity (m/s)	0.127	0.164	0.218	0.265

The zoning for this site, using the 1990 NBCC/CNBC, is Acceleration Zone 3, corresponding to 0.15 g; Velocity Zone 4, corresponding to 0.20 m/s. These figures are for a probablility level of 10% in 50 years.

Deterministic Analysis

Three faults are examined as posing significant seismic threat to the Mount Nansen site (Figure 1): Fairweather, Denali, Tintina. For the purposes of evaluating the maximum potential ground motion from each of these faults, determined using fault dimensions (Adams *et al.*, 1999), the attenuation relations of Youngs *et al.* (1997) are employed. These calculations provide an *upper* limit of ground shaking. Results are given in Table 2.

The closest of the three faults is the Denali Fault at a minimum distance of 125 km from the site. With a maximum predicted magnitude of 7.3, this would provide a PGA of roughly 0.125g. The Fairweather Fault, although farther at 254 km, suggests a similar threat as the predicted maximum magnitude 8.7 event within the adjacent subduction portion of the plate boundary could induce a PGA of 0.131g.

Finally, to the other side of the site, the Tintina Fault has the same postulated maximum magnitude as the Denali Fault, but at a slightly greater distance this naturally provides a lesser threat. The fault is included in the interest of completeness.

Table 2: Results of deterministic seismic hazard calculation.

Fault	distance to site (km)	maximum magnitude	PGA (cm/s^2)	PGA (g)
Fairweather	254	8.7	128.11	0.131
Denali	125	7.3	122.76	0.125
Tintina	132	7.3	114.45	0.117

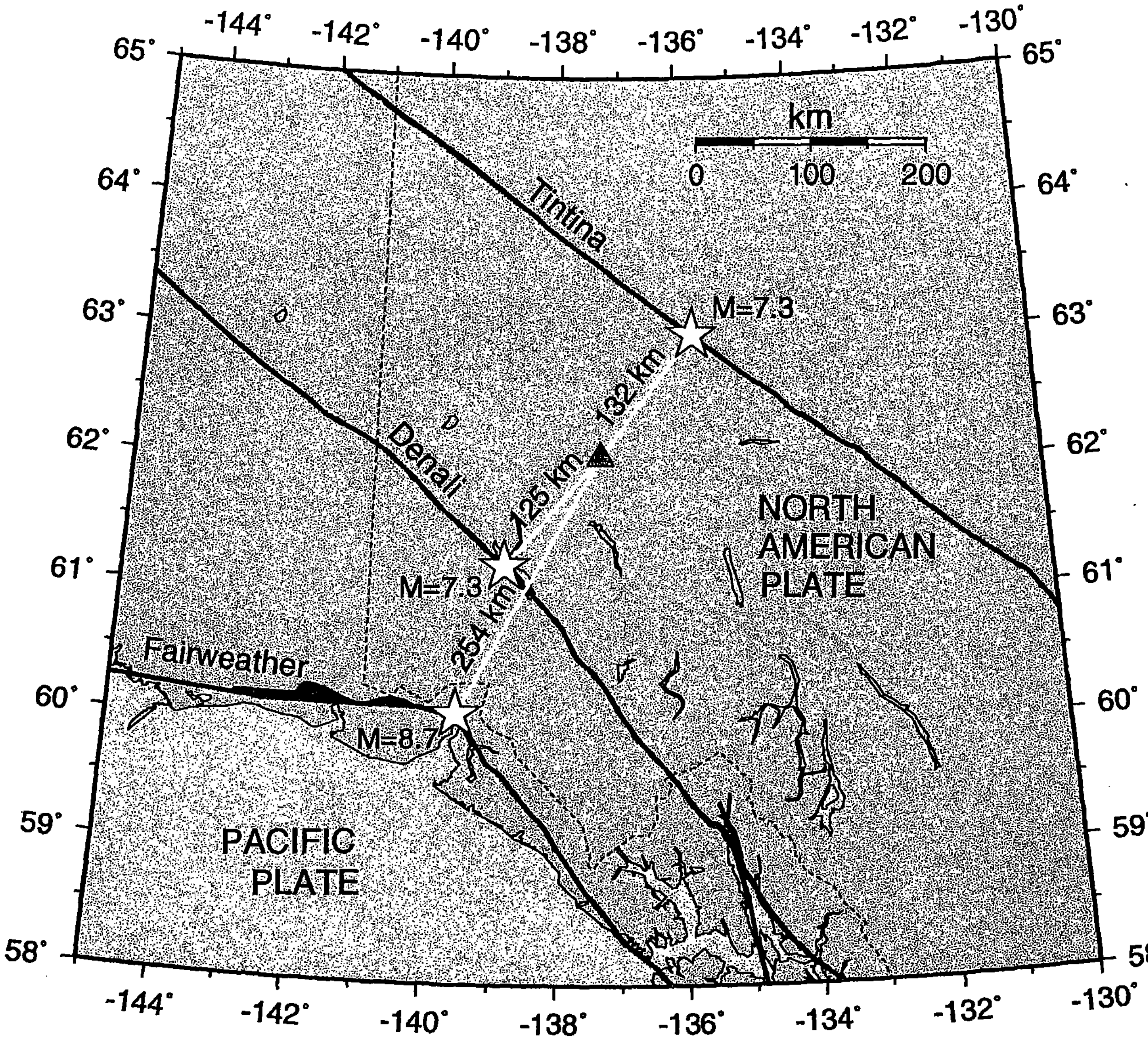


Figure 1: Tectonic setting of the Mount Nansen site, with faults of interest to a deterministic analysis of the site. Mount Nansen site is denoted by a red triangle; maximum postulated magnitude and site–fault distances are also noted.

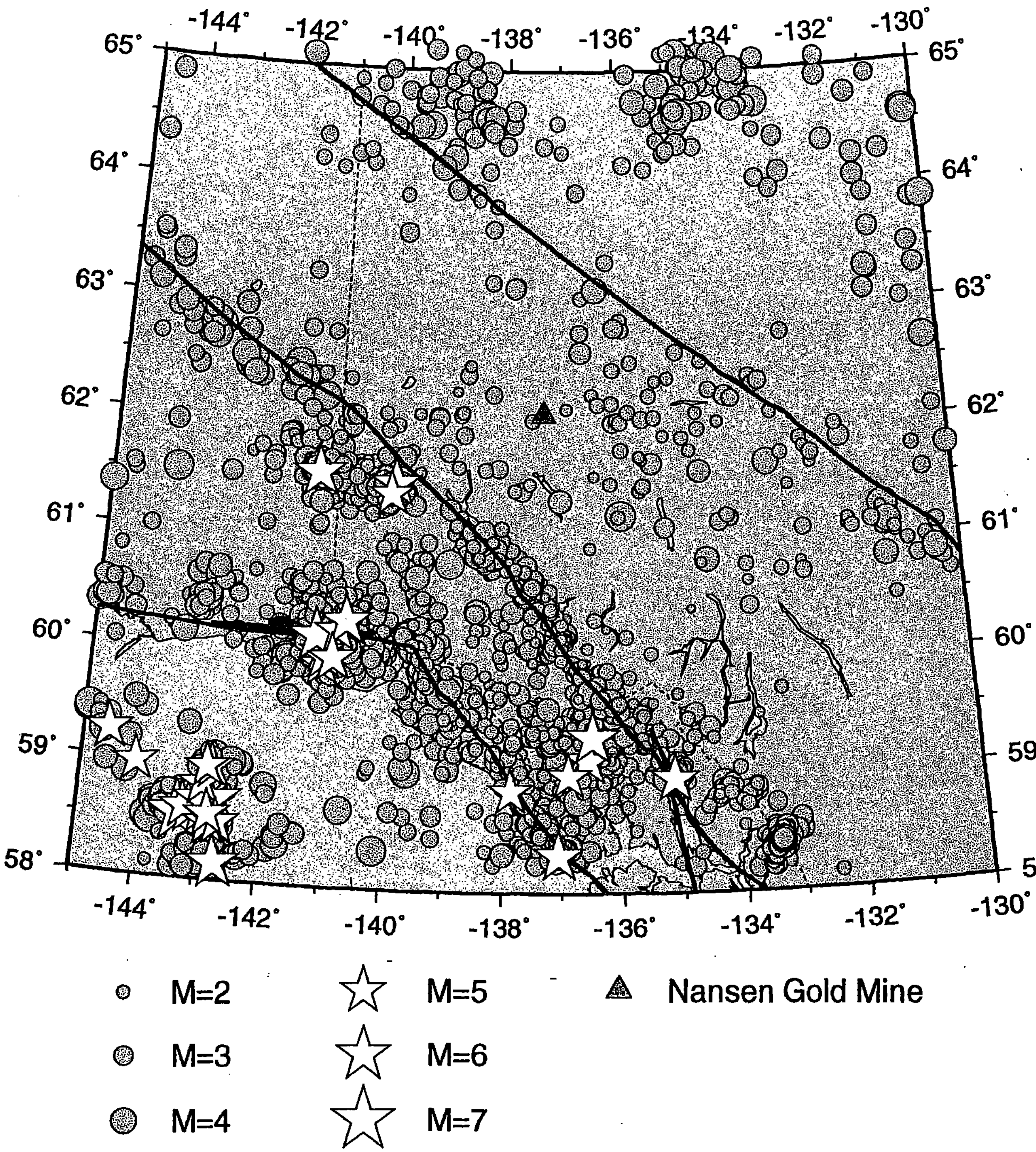


Figure 2: Map of area of study, showing seismic activity from 1980 to 2000.

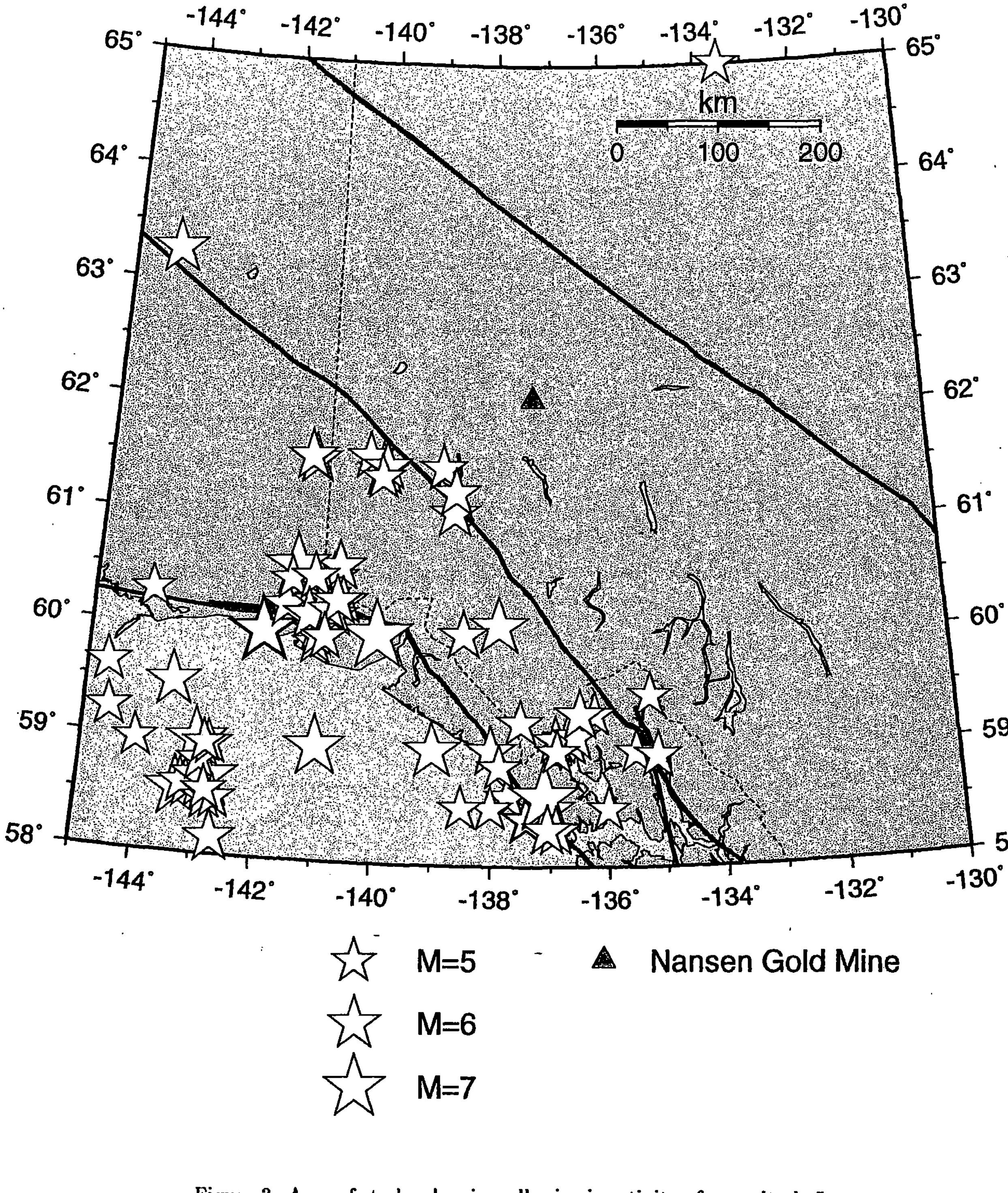


Figure 3: Area of study, showing all seismic activity of magnitude 5 and above, for the period 1898 to 2000.

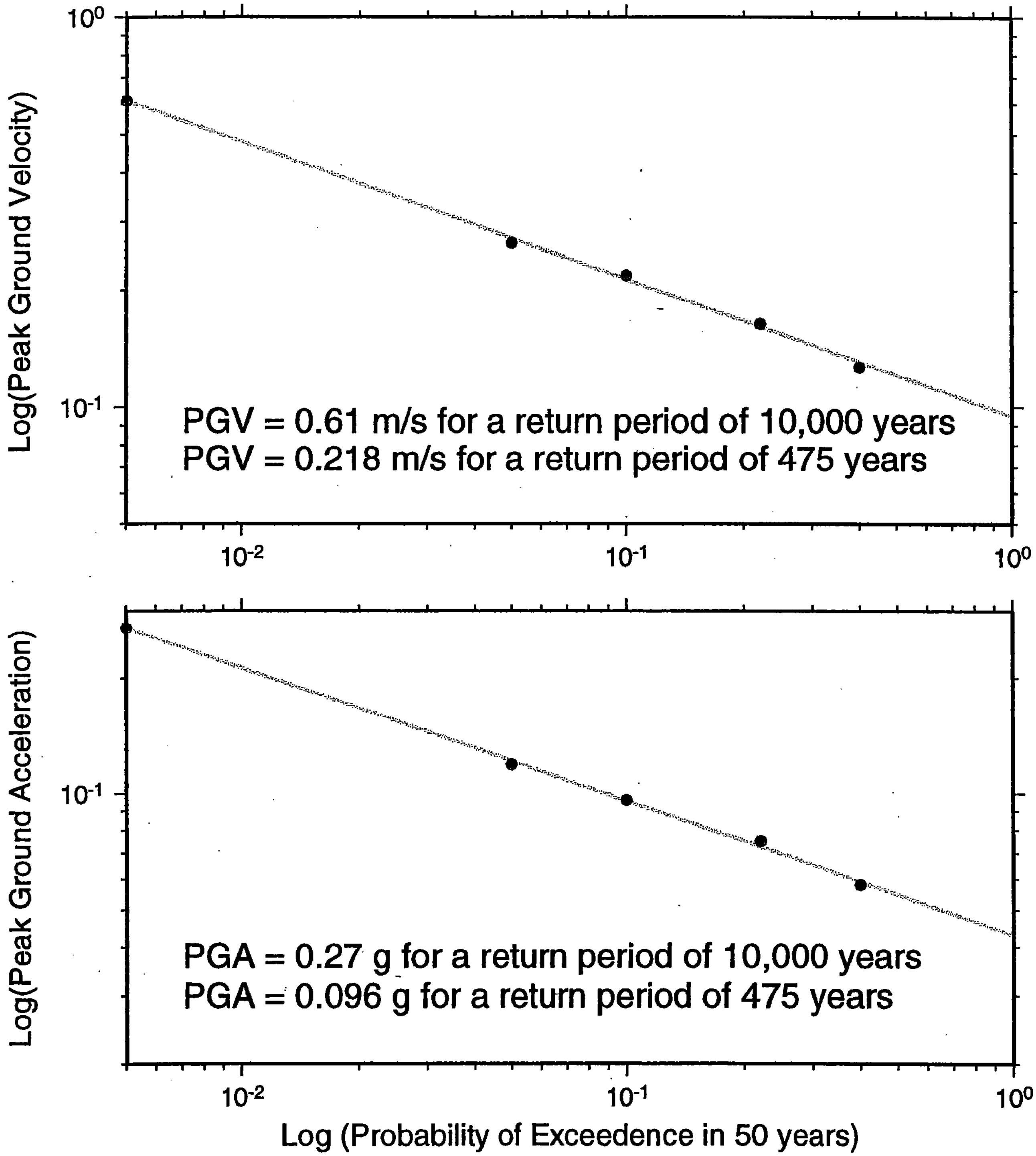


Figure 4: Log-log plots of Peak Ground Velocity and Peak Ground Acceleration versus Probablility of Exceedence.

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