Mount Nansen Project

Update to Water Licence Application QZ94-004
Supporting Documentation

Volume I of II
Sections I through IV

Submitted to:

Yukon Territory Water Board

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December 1st 1995

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General

DIAND produced the final Screening Report, dated November 15, 1995 resulting from their environmental assessment carried out under the federal Environmental Assessment and Review Process Guidelines Order (EARPGO) of BYG Natural Resources Inc.'s Mount Nansen Project.

In the report DIAND addressed all of the concerns, voiced to date, about the project and, where DIAND felt that the concerns had been adequately addressed by BYG said so, and where DIAND felt the concerns had not been adequately addressed, made recommendations for those concerns to be addressed in the Water License Application or in supplemental material to that application.

BYG has now prepared further documentation and made further commitments, in addition to those already encompassed in the Water License Application, in order to adequately deal with these concerns.

All of DIAND's recommendations and BYG's response to those recommendations are listed below by section.

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SECTION I

Ref: Screening Report Section 6.1
Screening Report Section 6.6.1

Also Ref: Tailings Impoundment
Final Design Report Sections 9, 10 & 11

CONSTRUCTION QUALITY ASSURANCE MANUAL
(Tailings Impoundment
Waste Rock and
Diversion Channels)

General

This manual addresses constrcution quality assurance (CQA) for construction of the tailings impoundment and diversion channels and for the disposal and placement of waste rock, for the Mount Nansen Mine, Yukon Territory. This manual is to be used in conjunction with the proper technical specifications and construction drawings. The QA Monitor and all contractors are required to become fully familiar with the technical specifications and contract drawings. Responsibilities of the various parties involved int he construction are defined along with the testing and documentation for the following construction and installation.

Tailings impoundment Site Clearing

Borrow areas Site Clearing

Seepage recovery dam Site Clearing

Diversion ditch Site Clearing

Emergency Spillway Site Clearing

Tailings impoundment Foundation Preparation

Permafrost excavation

Tailings impoundment Embankment Construction

Seepage recovery dam Embankment Construction

Tailings impoundment Geosynthetic Clay Liner Installation

Seepage dam Geosynthetic Clay Liner Installation

Diversion channel Erosion Protection

Tailings impoundment crest Erosion Protection

Tailings impoundment downstream slope Erosion Protection

Seepage dam downstream slope Erosion Protextion

Emergency spillway Erosion Protection

Pneumatic piezometer Insatllation

Thermistor Installation

Waste Rock Placement

Quality control will be the responsibility of the manufacturers, suppliers and installation contractors, all of whom will be responsible to the General Contractor, referred to in the specifications and this manual as the Contractor.

Definition of operations and Responsiblities

Parties who may be involved in the production, delivery and installation of materials and construction with on-site materials for the project are listed and defined below.

Designers:
Klohn Crippen. Responsible for the design, contract drawings and specifications for the constuction listed above.

Contractor:
Responsible for all the work on the Project. The contract has not yet been let.

Supplier:
Responsible for the manufacture and supply of geosynthetic products to the Installer. Depending on the Installer the supplier may also be the manufacturer and fabricator of the product.

Installer:
Responsible for field handling, storing, placing, seaming and other site specific aspects of geosynthetics. The Installer may also be responsible for transportation to site. Responsible for anchor trenches and all temporary anchoring or loading required to suport the geosynthetics during installation. Installer will be responsible to the Contractor.

Owner:
BYG Natural Resources Inc. is the Owner and will be responsible for operating and maintaining the facility and interfacing with the regulatory agencies. The owner will appoint a representative to coordinate filed activities. The Contractor will be responsible to the Owner's representative.

Engineer: — activities compliance with design + Designer
To be a party independent of the Owner and the Contractors, responsible for the observation and documentation of activities pertaining to the assurance of the quality of the installation of manufactured products and the construction of on-site products.

Quality Assurance Monitors:
working for and reporting to the Engineer.

Communications:

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During construction the QA Monitors will report items complying with the specifications and those not complying, directly to the Owner, who will direct the Contractors in the actions to be taken to enforce compliance. Remedial action will be tracked and reported to the Owner by the QA Monitors. QA Monitors will report only compliance or non compliance and will not direct or instruct the Contractors.

Rpt. to Engineer + Owner

Meetings

An initial meeting will be held at site with all parties involved in the construction and quality assurance. This meeting will be used to review critical details in design, construction scheduling, and quality control and quality assurance procedures, responsibilities and authorities of the parties, lines of communication, methods for documenting and reporting and for the distribution of such documents and reports.

The Owner will hold meetinga at least once a week with the Contractors, and the QA Monitors to review progress, scheduling, items of concern and the resolution of any outstanding issues relating to Quality Assurance.

Site Clearing/Foundation Preparation

Site clearing and foundation preparation is required in the areas of work including the borrow areas, dam footprint area, seepage recovery dam, diversion ditch and emergency spillway. Performance of this work should be carried out as follows:

1. Remove all trees, shrubs and bushes and stockpile separately in the designated disposal areas shown on the drawings or approved otherwise by the Engineer.

2. Excavate surficial organic materials in areas of work including moss, peat and organic silts to expose native sands. Surficial organic soils may be used for future reclamation purposes and should therefore be stockpiled separately from surface vegetation in designated areas shown on the drawings. Excavation of organic soils in the base of Dome Creek valley underlying the dam embankment should be a minimum of 1 m and may vary up to a maximum of 2 m.

3. Remove all frozen lumps, ice or otherwise unsuitable material from the foundation area. Seepage from excavation side slopes should be controlled by positive drainage or pumping.

4. Foundation excavation and preparation in the base of Dome Creek valley should only be carried out after diversion of the creek flows and at such time as fill material is ready for placement on the prepared frozen subgrade. Immediately after the subgrade is exposed, the fill material should be placed and compacted in lifts as specified up to at least the original grade.

5. Subcuts are required along the upstream toe of tailings dam and seepage recovery dam for placement of the geosynthetic clay liner. These excavations should only be completed at such time as the liner material and backfill material are ready for placement.

Embankment Construction

Compacted earthfill embankments are required for the main tailings dam and seepage recovery dam. Performance of this work should be carried out as follows:

1. Supply Zone 1 embankment fill consisting of native sand material with a maximum of 50% fines passing the 75 mm size. The maximum size should not exceed 150 mm except in layers adjacent to the geosynthetic clay liner where the maximum size should not exceed 50 mm.

2. Ensure all Zone 1 fill materials are thawed prior to compaction.

3. Aerate or add water where necessary or mix dry soils with wet soils to achieve moisture contents within ± 3% of optimum moisture content as determined by ASTM D698.

4. Condition moisture content in Zone 1 embankment fill by blading, rolling or otherwise working material with a dozer or grader.

5. Place Zone 1 embankment fill in near horizontal lifts not exceeding a compacted lift thickness of 250 mm.

6. Compact embankment fill material to a minimum of 95% standard Proctor maximum dry density as determined by ASTM D698.

7. Compact embankment fill materials with a vibratory smooth or tapered pad drum roller with a minimum operating weight of 8000 kg.

8. Overbuild the upstream slope of the main tailings dam and seepage recovery dam to allow trimming and final grading of the fill surface prior to placement of the geosynthetic clay liner. Trim slopes for placement of liner using a smooth bucket excavator.

9. Ensure stones or rocks in excess of 50 mm are not exposed on the upstream fill surface prior to placement of the geosynthetic clay liner.

Geosynthetic Clay Liner (GCL) Installation

A geosynthetic clay liner is required on the upstream slope of the main tailings dam and seepage recovery dam. Supply and installation of the GCL should be performed as follows:

1. The geosynthetic clay liner should consist of Bentofix NW as produced by Terrafix Geosynthetics Inc. and should be supplied in rolls 4.62 m wide by 30.5 m long (packaged weight approximately 600 kg). BYG will supply the Bentofix GCL for installation by the Contractor.

2. During all stages of shipping, handling and storage of the Bentofix NW GCL, exercise care to prevent puncturing or other damage to the protective covering and GCL material and to protect against wetting of the rolls. Provide adequate storage facilities for the GCL materials while on-site prior to installation.

3. Remove protective coverings only immediately prior to deployment of the materials.

4. Any GCL materials that are exposed to precipitation or are otherwise wetted, should be set aside for examination by the Engineer to establish the degree of damage. Prior to deployment, examine any rolls of Bentofix that become contaminated with foreign materials to ensure that the material has not been compromised. In some cases, the material may be salvaged, but significant wetting of the Bentofix may complicate or even impede deployment and the normal course of action should be that the material be discarded.

5. In areas along the upstream toe of dam where the Bentofix is to be deployed directly above the in situ soil materials, remove all roots, large (>50 mm diameter) rocks, frozen lumps, debris or other foreign materials at the surface of the subgrade.

6. Ensure the final subgrade surface is smooth. If the subgrade is frozen and irregular, place a levelling layer of fine sand and roll with a smooth drum roller. The Engineer should record the soil type for the subgrade surface and approve the condition of this prepared surface prior to the deployment of any Bentofix.

7. Install sections of the Bentofix from the highest point to the lowest.

8. On slopes, install Bentofix down, not across the slope.

9. Cut Bentofix using only approved cutters, and take care to ensure that materials underlying them are not damaged during cutting.

10. The bentofix should be kept as clean as possible at all times up to and including the time of placement of the next layer of material covering them. Cover joint areas with a temporary layer of Terrofix 600 R geotextile until such time as joints are ready for placement of overlying layers of Bentofix.

11. Place adjacent panels of Bentofix parallel and overlap a minimum of 150 mm along the side joints, and 300 mm at end joints.

12. Treat overlap joints by the addition of powdered bentonite comprised of the same bentonite as used in the Bentofix sandwich, both between the overlapped sheets and along the exposed edge of the joint.

13. At the perimeter of the installation, as well as at the top of slopes, anchor the Bentofix layer into a trench. This anchor trench should be located at least 600 mm beyond the crest of the slope, and should be a minimum of 450 mm deep. Place the Bentofix into this trench extending down the inside face and across the bottom of the trench. Secure the material by the controlled placement and compaction of backfill into the trench in such a manner as to not damage the composite. In all cases, provide appropriate anchorage to prevent the ingress of any surface runoff beneath the Bentofix, and to secure the GCL for stability and performance.

14. Mark all areas requiring repair due to damage during shipping, handling, or deployment, or manufacturing flaws in the materials. In cases where the material is pervasively damaged and repair is impractical, the material so affected should be marked accordingly, removed, and set aside or removed from the site so as to avoid reuse. All repairs made by the placement of a patch of the same material over the flaw or damage should extend at least 300 mm beyond the flaw or damage in every direction.

15. Within 48 hours, after placement of the Bentofix GCL, completely cover the GCL with a layer of Zone 1 sand and protect the GCL against damage. Ensure all procedures for placement of the GCL and cover layer are carried out in a manner which does not tear or damage the GCL.

Erosion Protection

Gravel armour is required to prevent erosion in the diversion channel upstream of the dam centerline and also on the crest and downstream slope of the main tailings dam and seepage recovery dam. Riprap is required for erosion protection on the steep gradient sections of the diversion channel and emergency spillway on the north abutment. Supply and installation of erosion protection materials should be carried out as follows:

1. Zone 2 gravel armour should consist of the following:

Size	Percent Passing by Mass
75 mm	100 %
40 mm	55% - 90%
20 mm	40% - 65%
5 mm	20% - 40%
1.25 mm	7% - 25%
80 mm	0% - 5%

2. Zone 3 riprap should consist of the following:

Size	Percent Passing by Mass
300 mm	80% - 100%
200 mm	40% - 60%
100 mm	10% - 30%
50 mm	0% - 10%

3. Zone 4 riprap should consist of the following:

Size	Percent Passing by Mass
500 mm	80% - 100%
300 mm	40% - 60%
150 mm	10% - 30%
50 mm	0% - 10%

4. The ratio of maximum to minimum dimension for riprap particles should not exceed 3.0.

5. Gravel armour and riprap should consist of sound, hard particles, free of organic matter and which meets the following minimum requirements for soundness and durability:

Test	Method of Test	Minimum Requirements
Abrasion	Los Angeles Machine	Less than 40% loss of weight after 500 revolutions
Resistance	ASTM C535	
Soundness	Magnesium Sulphate Solution, ASTM C88	Less than 10% loss of weight after 5 cycles
Specific Gravity	ASTM C127	2.60 minimum
Absorption	ASTM C127	2.0% maximum

6. Supply Terrafix 600R geotextile on smooth prepared subgrade underlying areas of riprap.

7. Overlap joints of Terrafix 600 R geotextile by minimum of 600 mm. Joints should be oriented in a manner such that the upper geotextile layer is placed upstream of the lower geotextile layer.

8. Place armour and riprap materials in continuous layers of uniform thickness.

9. Place primary riprap and bedding materials on slopes by backhoe or similar lifting equipment. End dumping of materials on slopes and pushing into place will not be permitted. Adequately interlock particles and dress slopes as required.

10. Place riprap and bedding materials in a manner such that smaller stones shall fill the voids between larger stones so that there is no unfilled space which could permit the escape of lower layers of placed materials. Compaction of materials is not required, however, there shall be no tendency of stones to move or slide after placement.

11. Construction traffic will not be permitted upon placed riprap for armour layers.

12. Do not place riprap until the riprap bedding or underlying geotextile has been accepted by the Engineer.

13. Do not break individual pieces of riprap after placement either by blasting or mechanical methods.

14. Place riprap to its full course thickness in one operation and in such a manner as to avoid disturbance or displacement of the underlying bedding materials or geotextile material. Remove riprap to repair disturbed or displaced bedding materials or geotextile.

15. Obtain the desired distribution of the various sizes of particles throughout the mass by selective loading at the quarry or other stockpile source, by controlled dumping of successive loads during placing, or by other methods of placement which will produce the specified results.

16. Place riprap and armour layers with a thickness at least as great as that shown on the Drawings.

Instrumentation

Installation of new instruments and extension of cables for existing instruments is required within the tailings dam. Performance of this work should be carried out as follows:

1. BYG will supply the instruments for installation by the Contractor. Drilling is required for installation of pneumatic piezometer tips below foundation grade. Installation of pneumatic piezometers will be supervised by the Engineer. Cooperate with Engineer and provide access for drilling and installation of pneumatic piezometers in the foundation.

2. Splice existing thermistor cables with additional cable sufficient to terminate at the downstream toe of dam. Splicing should be carried out in accordance with manufacturer's recommendations under the supervision of the Engineer. Pneumatic piezometers will be supplied with sufficient tubing at the time of installation to allow extension to the downstream toe of dam.

3. Excavate shallow trenches approximately 1 m deep in the Zone 1 sand fill for installation of the pneumatic piezometer tubing and thermistor cables.

4. Place all pneumatic piezometer tubing and thermistor cables on the base of trench in a zigzag manner to allow sufficient length for any deformations due to settlement. Backfill overtop tubing and cables in specified lift thicknesses. Compact lifts with compaction equipment not exceeding 1000 kg unless otherwise approved by the Engineer.

5. Drive steel bar settlement pins at the downstream edge of the dam crest with a stick-up of approximately 50 mm.

Performance Monitoring

In order to assess the long term requirements of the dam, it is imperative that performance of the embankment and foundation be monitored throughout the mine operation period. The field monitoring program should include the following:

1. Record temperatures for each of the thermistor installations to determine rate and extent of thaw.

2. Record pore pressures for each of the pneumatic piezometers in embankment and foundation zones which are unfrozen. This will assist in determining seepage through the dam and will allow measurement of actual thaw-induced pore pressures for stability assessment.

3. Record settlements of dam crest by regular surveys of settlement pins. Settlements will be correlated with the depth of thaw and pore pressure dissipation to provide a basis for future settlement estimates.

4. Record daily maximum and minimum temperatures at the Mt. Nansen site. Comparison of data at the site with similar data recorded at the Environment Canada Station at Carmacks will provide a basis for adjustment of temperature data used in long term prediction models.

5. Record precipitation data at the Mt. Nansen site to determine runoff inflows.

6. Record tailings pond levels as well as inflow and outflow volumes. Quantities required for water balance assessment should include reclaim, seepage, evaporation and spillway discharges.

7. Record pumping rates for determination of spigot, seepage and reclaim volumes.

8. Record rainfall and measurement of snow pack in late winter should be required to assess runoff inflows.

9. Record spillway discharges during operation to determine volumes of water which are diverted past the tailings pond. Comparison of diversion flows to runoff quantities will also provide an estimate of groundwater seepage inflows through the active zone.

10. Monitor the tailings beach development by survey to determine volume requirements and assist in planning of spigotting operations.

11. Record quantities and measured profiles of settled tailings to determine tailings densities and assess storage volume requirements.

12. Sample and measure the tailings gradation for assessment of the settling rate and permeability.

13. Monitor winter operations and tailings placement methods. Subaqueous deposition has been assumed for winter operations in order to minimize reservoir storage requirements.

14. Monitor performance of diversion and spillway channels to determine maintenance requirements. Monitor and remove snow in the diversion channel to prevent blockage.

15. If thawing of foundation soils occurs under the dam embankment, then test hole drilling should be carried out for measurement of SPT blow counts or cone penetration resistance. This data will allow assessment of seismic stability.

Waste Rock Characterization and Handling

Assessment of Construction Materials

The upper portion of the pit consists mainly of altered granodiorite which is a weathered friable material without any significant potential for generation of acidic drainage. This waste rock will be used for construction purposes since it has a mean NPR of 4.2 and Mean Net NP of 29. All waste rock used for construction will be sampled and tested for ABA using approximately 30 representative samples. These test results will be used to verify the use of these materials for construction purposes. The location and rock types for these samples will also be recorded to assist is assessing expected pit wall characteristics as discussed above in Section 2.0. Altered granodiorite waste rock will be used in the construction of the haul road to the mill, the tailings dam and the waste dump base.

Assessment of the Minerals Responsible for Neutralization

The analyses used to characterize the acid generation potential of the Brown-McDade waste rock consisted of standard acid base accounting procedures including paste pH using the "EPA" method (Sobek *et al.*, 1978). Total Sulphur was determined via the Leco Furnace method while Neutralization Potential (NP) was determined by the conventional EPA titration method. The results of these tests were provided in Tables 4-4 and 4-5 of the IEE. Recent ARD prediction work has focused on the development of new methods aimed at measuring both the reactivity of sulphide and the speciation of the minerals present that are responsible for neutralization. For example, in reference to neutralization capacity, calcite is generally considered to be reactive and readily available to neutralize acid under neutral and acidic pH conditions while silicates which are considerably less reactive, are generally not considered to responsible for neutralization until porewater pH drops. Research is currently being conducted to develop procedures to differentiate between the different types of minerals responsible for neutralization (Lawrence, 1995). One procedure currently being developed involves addition of acid at a fixed rate to a slurry of pulverized material together with continuous measurement of the resultant pH. The hypothesis is that if neutralization is due to the presence of calcite, the slurry pH will be buffered at a higher pH than if neutralization capacity is due to silicates. This approach will be followed in a similar research program using three samples of pulp from Brown-McDade waste rock. The procedure will be as follows;

1) Assay pulps of Footwall Granodiorite will be sourced. Samples selected are 88-83 24-30m, 88-97 23-27 and 88-72 40-42m (see Table 4-4 and 4-5 of the IEE).

2) The total quantity of acid that should be consumed will be estimated from the original NP test results.

3) An automatic titrator will be set-up to add acid at a fixed addition rate such that the total quantity of acid estimated from the above calculation is delivered over a 4hr period.

4) The resultant pH will be recorded continuously using a strip chart recorder.

The results of this testwork will be assembled into a lab report and distributed to the RERC for review and comment once the work is complete. However it should be noted that these procedures are currently in the developmental stage and the results may not be conclusive. The data generated from the titration procedure may however provide useful qualitative information. In the absence of a recognized standard procedure for assessing the minerals responsible for neutralization, the ABA data generated using the EPA titration method will be used for overall assessment and planning purposes.

Waste Management Plan

The handling of waste rock will be controlled by a Waste Management Plan. This program will be supervised by a qualified geologist. The individual rock types will be identified during mining based on a combination of location, visual characteristics, Au assay, ABA and other suitable analyses. The geologist will maintain detailed records for all materials brought to surface with the information logged according to quantities, waste characteristic and disposal locations.

Waste Rock ARD Characteristics

An ARD assessment program was conducted in 1994 to provide comprehensive information on the rock types to be encountered during mining and to confirm the previous preliminary results for Brown-McDade waste rock (described in the IEE, Section 4.6). The 1994 program involved the collection and assay of 37 drill core samples representative of major rock types from the Brown-McDade pit and the actual waste to be removed during mining. All samples were subjected to ABA assays and both ICP and Whole Rock metals analyses. The testwork report was included in Appendix V of the IEE.

The waste rock samples for the 1994 ABA program, were obtained from "HQ" sized diamond drill hole (DDH) core, drilled in 1988. The diamond drill holes (DDH) selected for sampling spanned the strike length of the deposit (433 m) with at least one DDH from each section line (section line spacing is 33 m). The samples were selected randomly from pieces of broken core over the designated intervals. For the wider intervals, (10-15 m) core was sampled at roughly 0.5 m intervals. For shorter intervals (1-5 m) a tighter sampling density was used. Rock types fell into three general lithological categories:

- Altered (weathered, partially oxidized) granodiorite; (Type 1)
- Altered (weathered, partially oxidized) feldspar porphyry (Type 2);
- Fresh (least altered, unoxidized) granodiorite (Type 3).

Based on the assessment of available data, personal inspection and sampling of the drill cores, it was believed that these samples adequately reflected the range of lithologies which would collectively represent the waste rock to be mined from the proposed open pit (Melling, 1994). Description of these samples, were summarized in Table 4-4 of the IEE. The acid base accounting testwork data, completed by PRA, was reported in Table 4-5 while the metal data was provided in Table 4-6.

The waste rock is generally low in sulphide with almost all the samples less than 0.5% S. There were one sample each of Lower Altered Granodiorite, Lower Feldspar Porphyry and Footwall Granodiorite that exhibited theoretical potential for acid generation. Only the Footwall Granodiorite sample contained appreciable sulphide, i.e. >1%. However, this sample was collected at the footwall contact with the mineralized zone - therefore it is likely that this material will be included as dilution with the ore and not be deposited in the waste dump. Concerns regarding the potential for acid generation from waste rock are negligible.

The estimated quantities of each rock type as a percentage of total waste are as follows:

Upper Altered Granodiorite	-	60%
Lower Altered Granodiorite	-	10%
Upper Feldspar Porphyry	-	15%
Lower Feldspar Porphyry	-	5%
Footwall Granodiorite (Unaltered)	-	10%

To demonstrate the overall characteristics of the dump, a plot of projected cumulative distribution for Net NP of the waste rock to be deposited in the dump is provided in Figure 2. This figure was generated on a weighted average basis using the ABA data provided in Table 4-5 of the IEE and the above projected waste rock type percentages. The plot assumes variation within rock type is represented by the samples collected and listed in Table 4-4 of the IEE. Figure 2 demonstrates the fact that the quantity of potentially acid generating waste in the dump will be small - approximately 4% with a Net

NP less than 0. However as mentioned above, some of the rock with negative Net NP values will likely report as dilution with ore due to its location on the footwall. Figure 3 provides a similar plot based on projected cumulative distribution of NPR for waste dump materials analogous to Figure 2. This plot demonstrates that approximately 10% of the dump will consist of waste rock with and NPR less than 2:1.

Waste Rock Monitoring During Operation

To verify waste rock characteristics, additional representative samples will be collected for each rock type and analyzed for ABA. The mine superintendent or geologist will be responsible for collecting samples of the waste rock that are considered representative of the waste rock deposited in the stockpile. This data will be used to update projections on the overall characteristics of the waste.

Special Handling Considerations

Special handling procedures will not be required for most of the waste rock other than the need to sample and characterize the materials according to ABA. However when the pit encounters Footwall Granodiorite at the base it may be necessary to utilize some extra measures if the ABA data indicates that the Footwall Granodiorite has a significant theoretical potential to generate acid. In this case, the rock would be either milled, left on the floor of the pit or encapsulated in the dump depending on its Au content and ABA characteristics.

Supplemental ARD Test Work

In summary the following supplemental ARD testwork will be conducted:

1) A research program involving titration procedures will be conducted to assess the minerals responsible for neutralization using three Footwall Granodiorite samples. The results of this program will be provided to the RERC as soon as they are available.

2) All waste rock used for construction will be sampled and tested for Acid Base Accounting using approximately 30 representative samples. The location and rock types for these samples will also be recorded to assist is assessing expected pit wall characteristics.

3) After completion of the pit approximately 20 samples will be collected from the pitwall to demonstrate that the wall does not contain potentially acid generating materials.

4) During operation, representative samples will be collected for each rock type and analyzed for ABA to update projections on the overall characteristics of the waste dump.

Future Considerations

The current design assumes a three year mine operation after which no further tailings will be deposited. Based on this operation plan, the thermal analyses indicate limited thawing under the upstream portion of the dam with eventual freezeback of the tailings after mine operations are ceased. Should the mine life be extended it is anticipated that thaw penetration will continue which must be accounted for in future assessments of the settlement, stability and seepage. Incorporation of a downstream berm has been considered as a future solution to several of the technical concerns. The need for a downstream berm should be determined based on the observations of the performance monitoring program.

Raising of the dam will require new diversion and emergency spillway facilities at a higher elevation. Excavations at higher elevations will be more difficult due to steeper slopes further up the valley slope. In addition, the existing diversion channel design does not include an impervious liner. If seepage losses are excessive, a liner may be required in the future.

Raising of the dam must also consider the method of future embankment construction. Raising of the dam on the existing centerline will require embankment construction of the upstream slope of dam overtop of the tailings beach which may have technical concerns. Shifting the dam crest centreline downstream will allow all construction to occur above the existing compacted embankment, however quantities of new fill will be greater and the downstream toe of dam may encroach on the seepage recovery dam.

The winter tailings deposition method will also affect the tailings density and beach slope. Subaqueous deposition is assumed, however, if beach spigotting occurs during the winter significant ice may build up, thereby reducing available storage densities and potential reclaim water.

A closure spillway is eventually required on the south abutment where bedrock can be exposed at the channel invert for long term protection. The downstream outlet channel of the spillway may, however, be constructed on overburden soils which will require erosion protection. It is anticipated that the riprap protection will be much larger than the riprap installed at the temporary emergency spillway. Waste rock which is large in size and may be suitable as riprap should therefore be salvaged from the mine operation over the next three years. The alignment of the closure spillway will be dependant on the eventual dam crest elevation. The invert of this channel should be situated in bedrock at least as far as the downstream toe of dam before descending the slope in the outlet channel. In order to finalize the alignment of the closure spillway, it will be necessary to eventually obtain additional rock depth information in the area downstream of the dam centreline on this abutment.

REFERENCES

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CONSTRUCTION QUALITY ASSURANCE
REPORTING FORMS

Inspection Report Number__________ Page______ of______

Project	File
Location	Date
Owner	Shift/Time
Submitted to	Weather

QA Monitor

Meeting Notes

Project	File
Owner	Date
Contract	Page of
Present	Recorder

No	Description	Action By	Date

Issued by ______________________ Received by ______________________

QA Monitor *Owner*

Notice to Owner Number________ Page______of______

Project	File
Owner	Date
Contract	q Site Instruction
Subject	q Transmittal
	q Attachments

Issued by ______________________ Received by ______________________

QA Monitor *Owner*

Document Transmittal

To:	Date
	Our File
	Your File
Attn:	Page of

We are sending the following:

o Prints o Reproducibles o Originals o Specifications

o Reports o Letters o Computer Files o Other

For:

o Your information o Construction o Your Approval o Other

Method:

o Enclosed o Separate Cover o Mail o Courier

o Other

No. Copies	Item and Revision	Date	Remarks

These documents were also transmitted to:

Moisture Content & Visual Description

Location	Sample	w, %	PL	LL	Description

Test Hole Log

Project:	Date:
Project No:	Equipment:
Location:	Ground El (m):
	Coords (m):

Depth (m)	Blow Count	Sample		Notes	Piezo Detail
		No	Type		

Laboratory Test Summary

Sample	Date	Lab Compaction (ASTM D-　　)		Particle Size, mm, % Finer						Comments
		γ_{max}	w_{opt}	150	25	9.5	4.75	2	0.075	

Field Density Tests

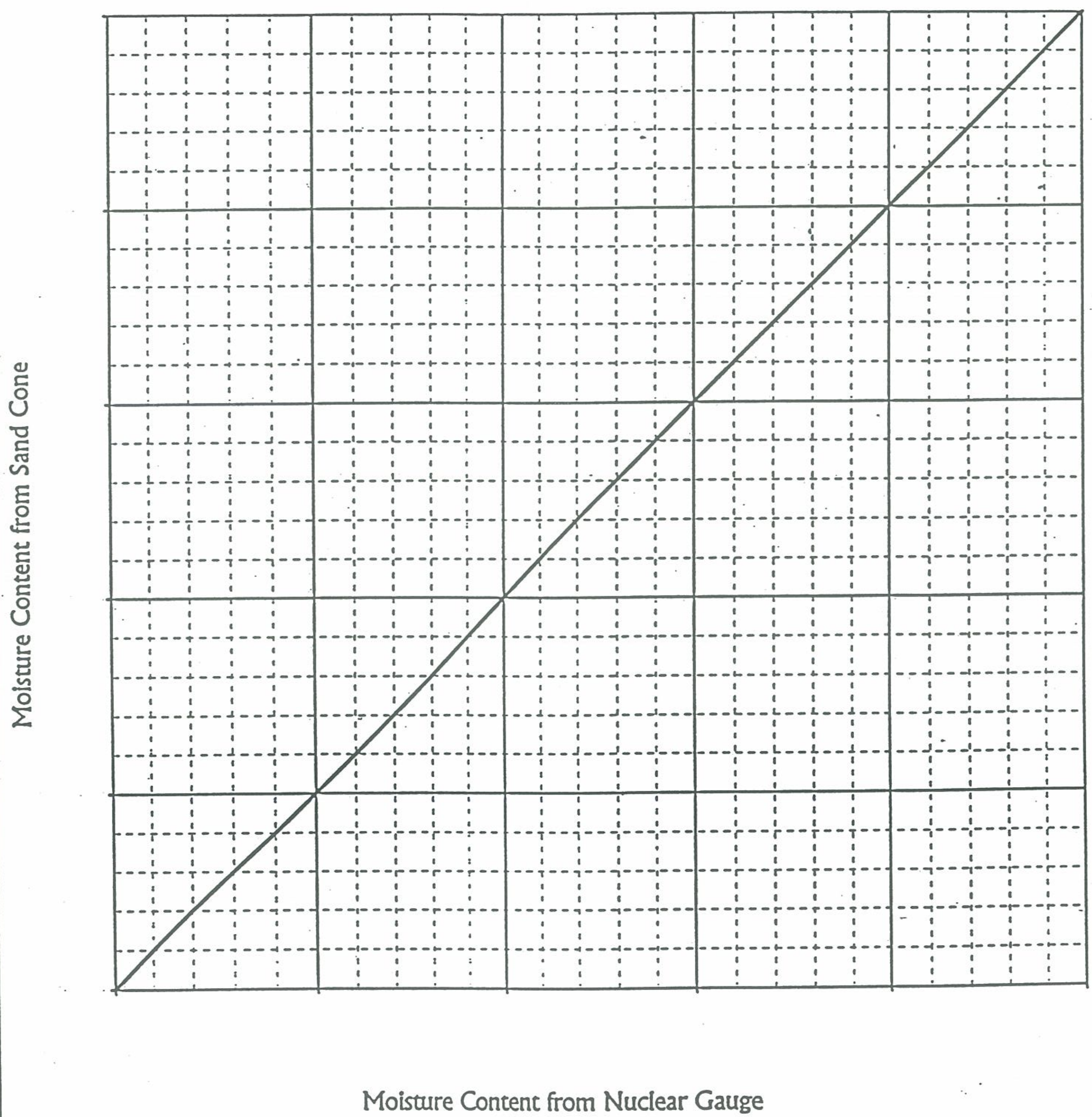
Material:

Density Specification: % ASTM D-

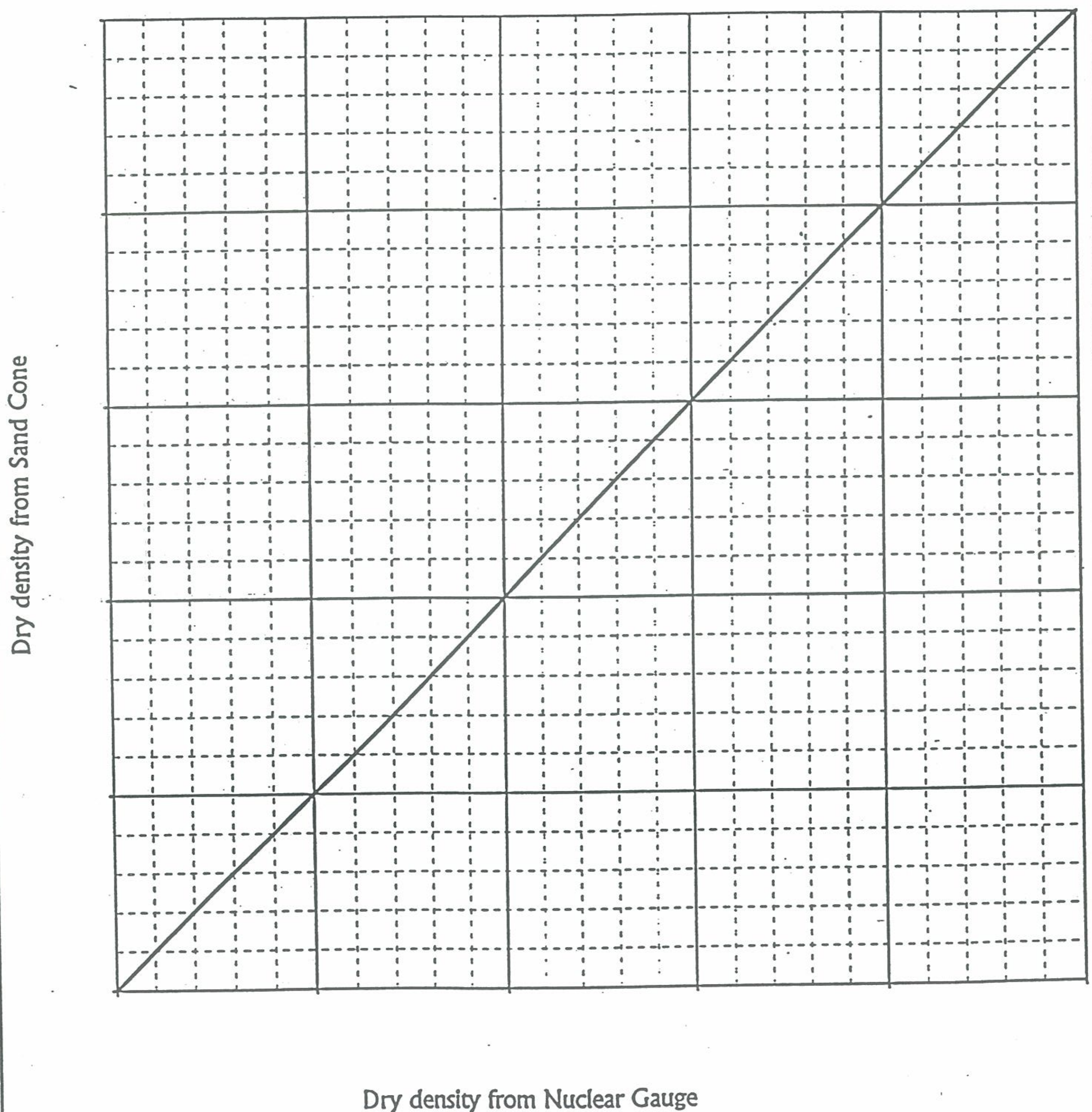
Moisture Specification: % of optimum

Test No	Date	Location	Elev	Moist %	Dry Density	Max Dry Density	Opt Moist, %	% of Max Density	Remarks

Material


Moisture Content from Sand Cone
Moisture Content from Nuclear Gauge

Material



Acceptance of Geomembrane Subgrade

Number____________________

Project	File
Owner	Date
Installer	

Area Accepted

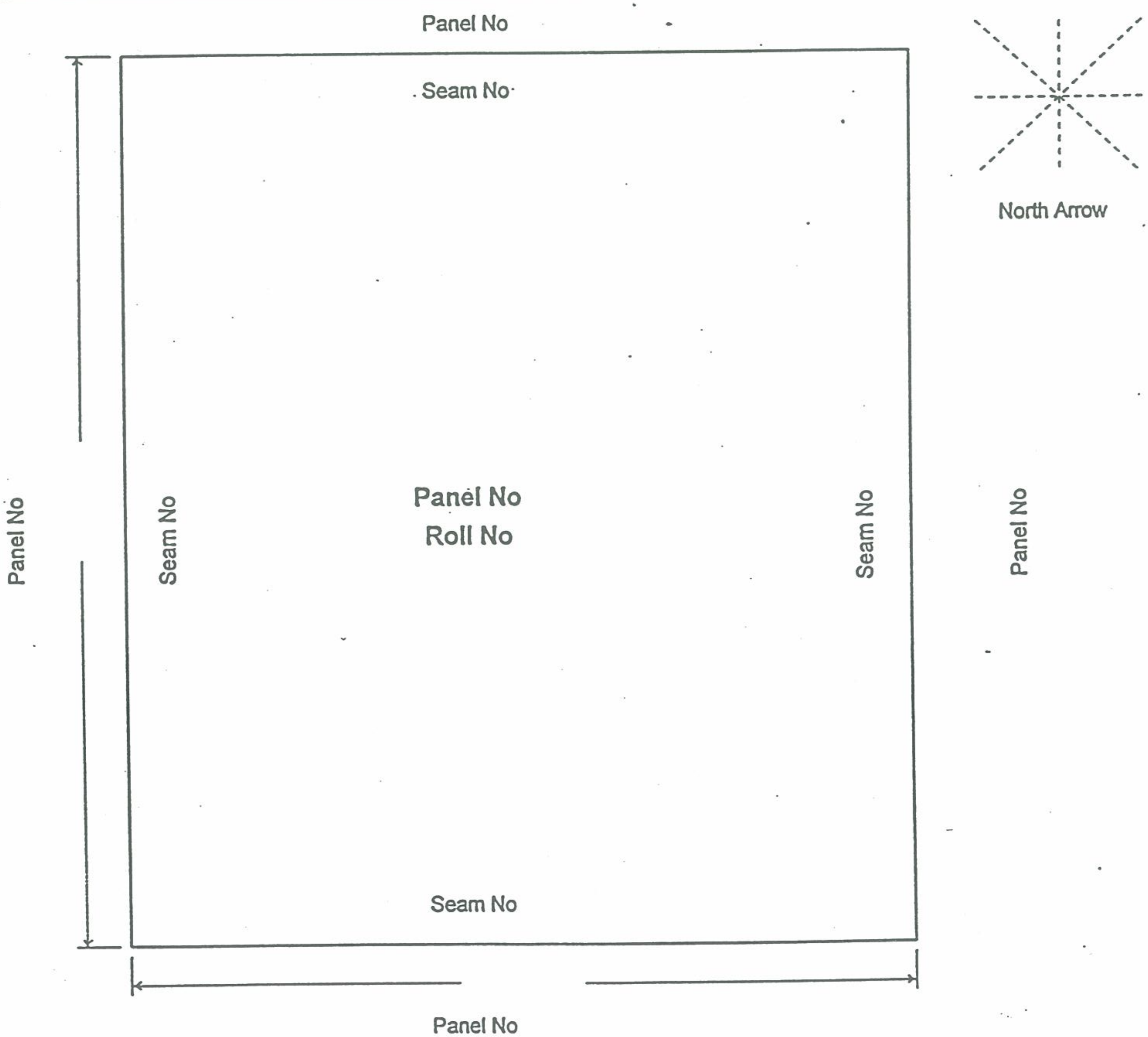
I, the undersigned, an authorized representative of the installer, accept the conditions of the soil subgrade surface and shall be responsible for maintaining its integrity and suitability according to the specifications. I do not accept responsibility for the condition or character of the subsurface soil, or any effect it might have on the liner system.

Issued by______________________________ Received by ______________________________

Installer *Owner*

Geosynthetic Panel Log

Project	Date Delivered	Date Deployed
Project No	Material Type	
Owner	Roll Certificates OK?	
Installer	Damage? (show on diagram below)	
Location	Date Approved for Cover	
Conformance Test (y/n)	Conformance Test No	Pass/Fail



QA Monitor

Geomembrane Seam Sample Log

Project	Owner
Project No	Installer

Loc'n	Material	Thickness	Shear Spec	Peel Spec

Date	Sample	Seam	Interval	Welder	Brk Type	Shear	P/F	Peel	P/F

QA Monitor

Geomembrane Non-Destructive Seam Test Log

Project	Seam No
Project No	Panel No
Owner	Material Type
Installer	Date Seamed
Location	Welder

Initial Test						Repair		
Interval	Test Date	Start Press.	ΔP/5 min	P/F	Comment	Date	Test Date	P/F

QA Monitor

SECTION II

Ref: Screening Report Section 6.1

Also Ref: Tailings Impoundment
Final Design Report Sections 7 & 8

DESIGN CRITERIA REPORT

TABLE OF CONTENTS

1. INTRODUCTION

B.Y.G. Natural Resources Inc. (BYG) intends to develop an open pit gold mine on the Mt. Nansen property located approximately 60 km west of Carmacks, Yukon (see Drawing A-5314-001). Development of the gold mine will require construction of stream diversion facilities and an embankment dam for tailings impoundment on Dome Creek. Several earlier investigations were carried out at the Mt. Nansen property by Klohn-Crippen, in order to evaluate potential dam sites for tailings impoundment and select a preferred site. Results of these earlier investigations by Klohn-Crippen are summarized in the following reports:

- Letter report related to Mt. Nansen Project, Tailings Disposal and Leach Pad Sites, dated November 1985.
- Mt. Nansen Gold Project, Tailings Dam Preliminary Design Report, dated December 1988.
- Mt. Nansen Gold Project, Tailings Facility, Preliminary Design Report, dated May 1990.
- Mt. Nansen Gold Project, Nansen Creek, Road Upgrading, dated September 1994.
- Mt. Nansen Gold Project, Tailings Dam Storage Study, dated September 1994.
- Mt. Nansen Gold Project, Tailings Dam Cost Estimate, dated October 1994.
- Mt. Nansen Gold Project, Tailings Impoundment Feasibility Design Update, dated April 1995.

Based on the results of the earlier investigations and evaluation of several alternatives, Site #4 was recommended by Klohn-Crippen as the preferred site due to the overall economy of a smaller dam combined with greater storage when compared to other site options. BYG has subsequently decided to proceed with final design and construction of a tailings impoundment facility at Site #4.

The scope of this final design investigation and report included the following:

- Summarize work carried out to date and provide details of site selection.
- Carry out test hole drilling at Site #4 to investigate in detail the foundation areas for the tailings dam, seepage collection dam, diversion ditch, emergency spillway, and closure spillway.
- Install thermistor strings to monitor and record the ground temperatures at several locations at the tailings dam site.
- Carry out field and laboratory tests to measure index properties of overburden soils including bulk density, moisture content, grain size, compaction density and permeability.
- Carry out analyses for final design including thermal modelling, static and seismic stability, seepage and settlement of the tailings dam due to thaw settlement.
- Review hydrology and water balance for impoundment accounting for details of tailings disposal, runoff inflows and reclaim water.
- Determine final design requirements for the tailings dam, seepage recovery dam, diversion ditch and spillways.
- Provide Construction Quality Assurance (CQA) requirements for construction of the tailings impoundment structures.
- Identify performance monitoring requirements and future design considerations.

Authorization to proceed with this investigation and final design were received from Mr. Jim Smith of BYG during a meeting with Mr. Bill Chin on June 5, 1995.

2. TAILINGS DAM DESIGN

.1 General

The Site #4 tailings impoundment dam is designed with a capability of storing 300 000 tonnes or 240 000 m^3 of tailings based on a settled tailings density of 1.25 tonnes/m^3. The reservoir storage-elevation curve for this site as shown on Drawing A-5314-006 indicates that the flat tailings level for this volume of tailings is elevation 1149.2 m. In order to provide sufficient additional storage for a minimum two month millwater circulation volume of 30 000 m^3 throughout the initial three year mine life, the design pond level should be at elevation 1149.7 m. The design requires a temporary emergency spillway to pass flood flows until such time as the mine operations are complete and a closure spillway is constructed. The temporary spillway is located on the north abutment in order to utilize the same outlet channel as the diversion ditch. The closure spillway will eventually be located on the south abutment due to the presence of suitable rock at this location.

The emergency spillway invert should be set at the pond level of 1149.7 m in order to discharge excess inflows during operation. The dam crest is set at elevation 1151.5 m, 1.8 m above the invert level in order to safely pass the design 200 year flood flow and at least 50% of the Probable Maximum Flood (PMF). The dam is considered to be in the low to significant hazard potential category and therefore a capacity discharge of 50% PMF is considered an appropriate safety standard.

An additional allowance for settlement is required for the initial dam crest elevation to ensure adequate freeboard at the end of mine operation. The settlement allowance should be at least 0.6 m resulting in a final dam crest elevation of 1151.5 m. The settlement allowance provides sufficient freeboard only for any short term settlements which occur in the first three years of operation. It is understood that prior to the end of year three of operation, the decision to raise the dam or alternatively to install a permanent closure spillway will be made. If a permanent closure spillway is constructed, raising of the dam may also be required to accommodate further long term settlements which may occur. Any decisions regarding raising of the dam at that time will be based on the dam performance as observed during the initial three year monitoring period.

The dam cross-section as shown on Drawing B-5314-007 includes a 6 m wide crest, an upstream slope at 2.5H:1V with toe berm and a downstream slope at 3.5H:1V with no berm. The downstream berm has been eliminated from the feasibility design on the basis that the thermal analyses indicate the shallow foundation soils are thaw-stable and the deeper soils below the active layer will not thaw in the long term. The downstream berm would only be required to control seepage and to enhance seismic stability for the event of deep foundation thawing. In the event, the downstream foundation area does eventually thaw, the need for the berm will be determined by further investigations to characterize the dynamic properties of the foundation soils. If necessary for stability purposes, the downstream berm can be constructed with waste mine rock or sand borrow.

The dam cross-section will include a Bentofix NW geosynthetic clay liner on the upstream slope to restrict seepage through the dam embankment. Details of the geosynthetic clay liner are shown on Drawing B-5314-008. A beach will be spiggotted against the upstream face of the dam during summer months to provide long term seepage control in the event that the geosynthetic clay liner deteriorates in the long term. To avoid ice lensing, all tailings will be placed subsequently during winter. Should seepage be a concern at a later date after a period of operation, a downstream berm could be constructed to include drains into the berm cross-section. The need for drains to control seepage at the downstream toe is best determined after observation of the dam performance and instrument monitoring. In order to assist in any future decisions regarding the dam performance, an instrumentation plan has also been incorporated with the dam design. The instrumentation system will include thermistors, pneumatic piezometers and settlement pins, as shown on Drawing B-5314-009.

Gravel protection will also be placed on the downstream slope to prevent runoff erosion of the fine sands in the embankment.

The following sections discuss the analyses carried out to arrive at the design cross-section for the dam.

.2 Thermal Analyses

.1 General

Thermal analyses were carried out to predict the short-term and long-term thermal changes in the tailings dam foundation during the operation and closure phases of the tailings impoundment. The principal objectives of the analyses were to predict the depth and rate of thawing of the foundation soils and to estimate the magnitude of the resulting settlements for determination of freeboard allowance and thaw-induced excess pore pressures for stability assessment as presented in subsequent sections.

The thermal parameters for soils at the Mt. Nansen site were estimated by comparison with published correlations for similar materials as shown on Figure A3-1 in Appendix III. Based on the available data, the thermal parameters summarized on Figure A3-2 were adopted. A check on the suitability of these parameters was made by analyzing the existing site conditions as discussed in the following sections.

.2 Analysis of Existing Site Conditions

A one-dimensional thermal analysis of the existing site was carried out using the computer program THERM1 by Nixon Geotechnical Ltd. The soil column and assumed thermal properties of the soils are shown on Figure A3-3. The soil column was modelled with an initial temperature of -1.5°C based on thermistor readings observed to date and the model then subjected to cyclic thermal boundary changes at the ground surface. The assumed air temperatures were assumed to be equivalent to the mean monthly air temperatures for Carmacks less 3°C to account for an elevation variation of approximately 600 m.

Ground surface and snow surface temperatures were derived from the corresponding air temperature by multiplying by the thermal "n" factor. A winter freezing n-factor of 0.9 was assumed to determine snow surface temperatures based on ambient air temperatures. The average snow cover thickness was estimated based on Environmental Canada records for Carmacks. The summer n-factor was assumed to be 1.3 in peat covered areas and a value of 1.5 was used for areas without peat such as the main dam and tailings area. The resulting ground temperature profile for the months of January, April, July, and October at test hole location DH95-10 was calculated by THERM1 as shown on Figure A3-3 for a time period of 25 years. The depth of the active layer and the temperature profile at depth did not change significantly after 25 years. The graph on Figure A3-3 shows good correlation between the estimated and measured ground temperatures as observed at the end of July. The thermal model was then analyzed for a typical soil column on the terrace slope in the vicinity of test holes DH95-04 and DH95-05 where ground temperatures were warmer at approximately -0.2 to -0.5°C. At these locations the sand soils with little to no organic cover showed the calculated and measured values both as having deep active zones with temperatures slightly below freezing in the permafrost zone.

In the event of complete thawing, the one dimensional thermal model also shows freezeback for each of the typical soil columns, however, the rate of freezeback is expected to be very slow and significant freezeback should not be expected in the sand fill or tailings areas for several decades after the end of operation.

.3 Thaw-Induced Pore Pressures

Thaw-induced pore pressures in the foundation soils may influence the stability of the tailings dam slopes, if the rate of thaw is faster than the rate of pore pressure dissipation at the thaw front. A conservative assessment of the rate of thawing of the foundation soils beneath the upstream dam slope was made by conducting a one-dimensional analysis of the rate of thaw beneath the reservoir with water only present and no tailings. The soil column and soil properties considered are shown on Figure A3-4. The temperature of the pond water was varied monthly using the reported reservoir temperatures at the Kelsey Generating Station in Northern Manitoba. These temperatures, given on Figure A3-4, agree with our experience at other lakes in the NWT were seasonal temperatures have been measures at the bottom of the lakes.

The predicted rate of thaw for a period of 25 years is plotted on Figure A3-4 in terms of the square root of time. The slope of the plot is the thaw constant (α). The predicted value of α is 6.2×10^{-4} $m/s^{0.5}$ (0.62 $mm/s^{0.5}$) which falls in the middle of the typical range of α between 0.2 and 1.0 $mm/s^{0.5}$ (Nixon and McRoberts, 1973).

The amount of excess pore pressure generated on thaw is related to the rate of thaw, and the soil's ability to dissipate pore pressure represented by the coefficient of consolidation, as follows:

$$u = \frac{\gamma'.d}{1 + \frac{1}{2R^2}}$$

Where:	u	= excess pore pressure (kPa)
	γ'	= submerged unit weight = $\gamma_t - \gamma_w$ (kN/m^3)
	d	= depth (m)
	R	= thaw consolidation ratio = $\alpha / 2\sqrt{c_v}$
	α	= rate of thaw ($m/s^{0.5}$)
	C_v	= coefficient of consolidation (m^2/s)

This relationship has been plotted on Figure A3-5. Using a lower bound C_v from Figure A2-4 in Appendix II of 1×10^{-6} m^2/s and $\alpha = 6.2 \times 10^{-4}$ $m/s^{0.5}$. The predicted excess pore pressure from the chart is 16% of the initial effective stress. Pore pressures of this amount were accounted for in the subsequent stability analyses.

.4 Two-Dimensional Analysis of Long-Term Dam Conditions

In order to evaluate and demonstrate the long-term thermal performance of the tailings dam, a two-dimensional finite element analysis of the tailings impoundment was carried out using the computer program TEMP/W developed by Geo-Slope International. The two-dimensional mesh used in the analyses is shown on Figure A3-6. The design includes an upstream berm which reduces the rate of thaw and enhances stability at the upstream toe of dam. This area will be subjected initially to thawing due to ponding of water and tailings disposal during the initial mine operations startup and the berm will therefore provide additional thermal protection.

The analysis conservatively considered the impact of the three year operating impoundment life by assuming a full pond water level with no tailings for the entire three year period. Closure conditions starting in Year 4 were then modelled by a tailings beach with the pond set back 50 m from the dam crest. The monthly varying ground surface temperatures were derived for each typical soil column (i.e. tailings sand, sand fill and native foundation soils with peat cover) using the one dimensional analysis and these values then input into the two dimensional model. The analyses were then extended for a total duration of 25 years. Figure A3-7 shows the representative sections which were analyzed and the depths of the thaw boundaries (0°C isotherm) after years 1, 2, 3, 5, 10, 15, 20 and 25 years.

The results of the analyses show:

- The zone of thawing in the foundation soils will extend only one-third of the width of the tailings dam from the upstream toe. The thawing is indicated to be confined to the upper 1 to 2 m of the foundation soils in the area underlying the upstream slope of the dam and up to 5 m under the pond. The subsequent thaw settlement analyses presented in the next section, assume a conservative depth of thaw of 4 m under the dam.

- In the event the tailings pond is operated for only three years, complete freeze back of the foundation soils beneath the dam is predicted after approximately 25 years. Extension of the mine life will however increase depths of thaw and the time for freeze back.

- Partial freezing of the downstream portion of the dam fill is predicted during the operating phase of the impoundment. The rate of freezing can be accelerated by removal of snow on the downstream slope of the dam.

- In the long-term, complete freezing of the dam is predicted with the final thaw boundary lying within the upstream deposited tailings.

To better assess the potential for thawing in the foundation soils, an additional analysis was made which took the operating conditions with full pond to a long-term operation life of 50 years. The results of this analysis, shown on Figure A3-8, showed that the thaw front (0°C isotherm) beneath the impoundment would extend down through the foundation soils into the underlying bedrock. The analyses however, estimate that the thaw front would not extend downstream of the dam centreline.

The above analyses do not consider the effects of convective heat transfer by seepage beneath the dam. The effects of such seepage will be minimized by the presence of a tailings beach and the geosynthetic clay liner in the dam which will cutoff seepage through the dam and the upper part of the foundation. Seepage under the dam will be restricted by the presently ice-saturated, frozen foundation soils.

The thermal analysis yields two important conclusions:

- Complete thaw of the foundation soils beneath the dam is not predicted during the operational phase of the impoundment thereby reducing potential thaw-settlements; and

- After mine closure, the regional permafrost will aggrade upwards into the dam in the long-term resulting in a strong and impermeable dam.

Irrespective of the above, the design takes into consideration long term measures which are required in the event of complete thaw of the foundation soils under the worst case condition. The thermal performance of the dam and underlying foundation and the influence of the tailings will be monitored throughout the operational life of the impoundment to determine which long term solutions are required.

.3 Thaw-Settlement Analyses

Settlement of the dam will occur in areas where the foundation soils thaw and consolidate under the embankment loading. Thermal analyses predict that the foundation soils under the upstream slope will only partially thaw during mine operations due to the warming influence of the tailings pond. These analyses predict that during the three year design life of the mining operation, thawing of the foundation soils will occur to a maximum depth of approximately 5 m under the pond area and less than 2 m below the upstream portion of the dam. The depth of thaw for the settlement calculations has therefore conservatively been assumed as 4 m.

In assessing the settlement due to thaw consolidation, the in situ frozen bulk densities were compared to empirical relationships which have been derived from published laboratory measurements and case history data of thaw settlement for various frozen soils ranging from frozen clays and silts to frozen sands. A series of correlations are shown on Figure A3-9 which show the relationship between the initial frozen bulk density and the normalized thaw consolidation ratio (i.e. ratio of settlement to initial height). For a given bulk density, higher plasticity soils generally speaking yield a higher thaw settlement than non-plastic silts and sands.

The median relationship between thaw settlement and frozen bulk density (γ_f) for all soils is given by the following equation:

$$\frac{\Delta H_T}{H} = 0.57\,(1.97 - \gamma_f)$$

The upper bound relationship between thaw-settlement and frozen bulk density for sands is given by:

$$\frac{\Delta H_T}{H} = 0.57\,(2.1 - \gamma_f)$$

The area considered to have the greatest potential for thaw settlement is located in the base of the creek valley. Three test holes including DH95-06, DH95-08 and DH95-09 were drilled in this area. An average of the frozen bulk densities in the upper 4 m of these three holes indicated an initial bulk density of approximately 1760 kg/m^3. Comparison of this initial bulk density to the design graphs for thaw consolidation on Figure A3-9 indicates an estimated settlement after three years of pond operation ranging from 0.5 m for the median case to 0.8 m for the upperbound case.

Bulk density values in the lower sand deposits below a depth of 4 m indicated an average bulk density of 1900 kg/m^3. Based on an assumed thickness of 16 m, the estimated additional settlement due to complete thawing of the foundation could vary from 0.7 m to 1.7 m for the median and upper bound cases respectively. In the event of complete thaw of the foundation soils, total settlements are estimated to vary from 1.2 m to 2.5 m for the median and upperbound cases.

The proposed dam cross-section allows for a potential 0.6 m settlement during the first three years based on the estimated thaw settlement in the upper 4 m. The dam performance over the first three years will be monitored for various behaviour aspects including the depth of thaw and dam crest settlement. Prior to completion of three years of operation, BYG are required to make a decision regarding raising of the dam for additional tailings storage or construction of the closure spillway for long term abandonment of the tailings impoundment. At that time the dam crest height should be reviewed and any additional allowance for longterm settlement made at that time. Should the dam performance and instrument monitoring suggest freezeback will occur after mine closure; then additional raising of the dam may not be required for settlement purposes.

.4 Stability Analyses

.1 General

Stability of the main dam embankment will initially be controlled by the presence of permafrost in the underlying foundation soils. The tailings dam overlying a frozen foundation is expected to behave more than adequately and stability is not an issue of concern. The design must however consider the long term possibility of the foundation thawing. Stability analyses were therefore carried out to determine the cross-sectional geometry required to ensure stability of the dam under static and seismic conditions with an unfrozen foundation.

Limit equilibrium analyses were used throughout the stability evaluation for the main tailings dam. The computer program SLOPE/W by Geo-Slope International was used to analyze the various stability models using the circular Bishop method.

A stability model was developed for the central valley Section `B`, as shown on Figure A4-1, where the dam height is a maximum of about 18 m. A review of the foundation stratigraphy indicated that the dam foundation could be conservatively modelled as a homogeneous sand despite the presence of frequent gravel layers in the foundation. The dam geometry was based on the preliminary design cross-section with upstream slopes of 2.5H:1V and downstream slopes of 3.5H:1V. As previously discussed, construction of the downstream berm was omitted, but a 4 m thick upstream berm was included to protect the upstream toe of the dam from rapid thawing and possible instability due to warm pond water during mine startup. Dependant on the dam foundation performance, a downstream berm may eventually be required, however, construction of the berm can be delayed without serious ramifications until the actual performance of the dam can be monitored by the foundation instrumentation.

For design, the minimum allowable factor of safety against failure for static conditions was taken to be 1.5.

.2 Soil Parameters

Standard proctor densities were measured for the proposed embankment fill as part of the borrow material evaluation. Maximum dry densities were measured at 1766 kg/m^3 and 1850 kg/m^3 for the typical sand material which will be used in the embankment. For an average value of 1808 kg/m^3 at 95% standard Proctor density and 13% moisture content, the estimated average bulk density of the embankment fill is 1940 kg/m^3 or 19.0 kN/m^3. During the recent field investigation, bulk densities were also measured in the various soil strata underlying the main dam embankment. These values are recorded on Figure A2-2. For the foundation soils underlying the base of creek valley, the average bulk density is estimated to vary from an average of 1760 kg/m^3 in the upper soils, to be between 1900 kg/m^3 and 2000 kg/m^3 in the lower layers. An average unit weight of 19.0 kN/m^3 was adopted in the analyses. Based on our experience with similar tailings at other mine sites, the dry density of the tailings is estimated at 1250 kg/m^3. With an average water content of about 40%, the estimated bulk density of the tailings is about 1730 kg/m^3 or 17.0 kN/m^3. The frictional strengths of the fine sands and tailings are estimated based on the typical grain size when compared to published values and experience with similar materials. The sand soils are all considered cohesionless. The soil parameters used in the stability analyses are summarized as follows:

SOIL TYPE	BULK UNIT WEIGHT (γ) (kN/m^3)	FRICTIONAL STRENGTH (ϕ) (Degrees)	COHESION (kPa)
Compacted Embankment Fill	19	34°	0
Foundation Sand	19	30°	0
Tailings	17	30°	0

.3 Static Analyses

To assess the effect of foundation pore pressures on the factor of safety, a number of parametric analyses were undertaken to relate the factor of safety to the pore pressure parameter r_u' which is defined as the ratio of excess pore pressure (u) to the effective stress ($\gamma'.d$) at a specific depth. From the equation in Section 7.2.3, the pore pressure parameter is calculated based on the pore pressure generated on thaw relative to the rate of thaw as follows:

$$r_u' = u/(\gamma'.d) = \frac{1}{1 + \frac{1}{2R^2}}$$

Where:

u	= excess pore pressure (kPa)
γ'	= submerged unit weight = $\gamma_t - \gamma_w$ (kN/m^3)
d	= depth (m)
R	= thaw consolidation ratio = $\alpha / 2 \sqrt{c_v}$
α	= rate of thaw (m/s$^{0.5}$)
C_v	= coefficient of consolidation (m^2/s)

The results of the slope stability analyses for the downstream slope are summarized on Figure A4-1. The upstream slope is contained by the tailings beach and stability is therefore not a concern by the time the foundation thaws. As shown, for the downstream slope, the minimum factor of safety for the downstream slope occurs at a r_u' value of approximately 0.55. The predicted r_u' value based on the rate of thaw ($\alpha = 6.2 \times 10^{-4}$ m/s$^{0.5}$) and the coefficient of consolidation ($c_v = 1 \times 10^{-6}$ m^2/s) of the material, as discussed in Section 7.2.3, is approximately 0.16 which results in a factor of safety in excess of 2.

.4 Seismic Analyses

The Mt. Nansen minesite lies within a seismically active area of the Canadian north in Seismic Zone 2, as defined by the National Building Code of Canada, 1990. The site lies within the Northern British Columbia (NBC) source zone between the McKenzie (MKZ) zone and the Denali Shakwak (DSK) zone.

A probabilistic assessment of the seismic risk at the site has been undertaken by the Pacific Geoscience Centre for the Mt. Nansen site (October 1988). Based on this assessment, a peak horizontal ground acceleration of approximately 10% g is expected with probability of exceedance no greater than 0.0021 per annum, or one in 475 years, while a peak horizontal acceleration of 12% g is expected with a return period of .001, or one in 1 000 years.

The dam foundation consists primarily of fine sands and hence, if sufficiently loose, can be potentially liquefiable. The governing parameters regarding the liquefaction of the sand are the thawed in situ density and the design earthquake in terms of magnitude (M) and acceleration (a_h). Current engineering practice to determine liquefaction susceptibility in foundation soils, requires the measurement of Standard Penetration Test (SPT) corrected blow counts ($(N_1)_{60}$) either by SPT or cone penetration methods. If an analysis of the blow count profiles with depth indicates that the foundation is potentially liquefiable then the residual strength for the material can be estimated and a corresponding factor of safety can be calculated. However, based on the current frozen nature of the foundation and the predictions from the thermal analysis related to future thawing, the following approach is recommended:

- Monitor the actual performance of the dam and foundation over the 3 years of operation.
- At time of mine closure review seismic stability including the design earthquake and SPT data from thawed zones (if any).
- Review liquefaction potential and, if required, include a downstream berm in the mine closure plan.

.5 Seepage Analyses

.1 General

The design of the main tailings dam is intended to minimize seepage of the pond water downstream of the dam into Dome Creek. This will ensure release of water from the tailings pond is controlled through treatment facilities and/or the emergency spillway and will also ensure that piping and loss of material does not occur at the downstream toe of dam due to uncontrolled seepage pressures.

The proposed dam embankment is expected to be constructed of fine to medium sand with a silt content typically varying from 10% to 40%. The foundation soils underlying the dam similarly consist of these type of sand materials. The fine grained sands are moderately pervious and a seepage barrier is therefore required. The presence of an ice saturated frozen foundation will provide an initial barrier against seepage, however, the design must account for the long term potential of the foundation thawing with a corresponding increase in foundation permeability.

The primary barrier to impede seepage through the dam will consist of the tailings which will be deposited on the upstream slope throughout the period of mine operation. The tailings to be stored in the impoundment are expected to have a high content of clay size particles with a corresponding low permeability. The thickness of tailings will increase throughout the mine operation as the level of required protection also increases due to higher pond levels and a greater potential for thawing in the foundation. A secondary seepage barrier is provided by inclusion of a geosynthetic clay liner on the upstream slope. The clay liner will provide seepage protection during the initial pond startup when water will be ponded against the upstream slope prior to tailings deposition. Seepage protection is also provided by the geosynthetic clay liner in the event foundation thawing occurs which may result in differential settlement and cracking of the dam. Cracking is not however considered a major concern due to the self healing nature of the cohesionless sand fill and the thick covering of tailings on the upstream slope.

The initial dam cross-section to be constructed does not include a downstream berm. In the event that seepage does develop and seepage pressures at the downstream toe are reason for concern, a downstream berm can be constructed with drains which can control seepage without piping or loss of material at the toe.

The limited seepage expected through the embankment and underlying foundation will be collected upstream of the seepage recovery dam where it can be returned to the tailings pond by pumping or released downstream if acceptable water quality is achieved. This small embankment for the seepage recovery dam will also be provided with a geosynthetic liner which will be keyed into the underlying permafrost. Details of the main dam, seepage recovery dam and geosynthetic clay liner are shown on Drawings B-5314-007 and -008.

.2 Seepage Analyses

Seepage analyses were carried out using the two dimensional finite element computer program SEEP/W by Geo-Slope International. The seepage model is shown on Figure A5-1 and the hydraulic parameters are listed below:

MATERIAL	HYDRAULIC CONDUCTIVITY (cm/s)	Kh/Kv
Bedrock	$1x10^{-7}$	1
Frozen Sand Foundation	$1x10^{-7}$	1
Thawed Sand Foundation	$1x10^{-3}$	1
Dam	$1x10^{-3}$	1
Tailings	$1x10^{-5}$	1

The design values of the hydraulic conductivity were based on laboratory testing and assumed values for similar type soils. For comparison, falling head permeability tests on recompacted samples of the foundation sand gave permeability values of $2x10^{-5}$ cm/s to $9x10^{-4}$ cm/s. All materials were assumed to be isotropic with the ratio of horizontal and vertical permeabilities equal to unity.

The seepage model consisted of a typical dam cross-section with dam crest set at elevation 1151.5 m, a pond elevation of 1149.7 m and a 50 m wide tailings beach. The downstream water elevation was set at 1136 m. The presence of a geosynthetic clay liner was not included in the analyses and therefore the estimated seepage quantities are expected to be conservatively higher than anticipated. The seepage analyses indicate that the phreatic surface will be low in the downstream portion of the dam, as shown on Figure A5-1, and quantities of seepage through the dam should also be low at approximately a maximum of 0.2 l/sec for the case of a frozen foundation and unfrozen dam over a 200 m wide seepage zone. At this rate of seepage, the seepage recovery pond should fill to the crest level after two months operation. Figure A5-1 also presents the variation of predicted seepage with depth of thaw under the dam. With about 16 m of thawed foundation, the maximum predicted seepage increases to about 2.4 l/s assuming a 200 m wide seepage zone (14 days capacity in the seepage recovery pond). It is important to note that these seepage rates are dependant on the development of a tailings beach on the upstream face of the dam.

.3 Geosynthetic Clay Liner (GCL)

The proposed GCL consists of Bentofix NW by Terrafix. The Bentofix NW is a sandwiched composite of two non woven geotextiles with an intermediate bentonite clay layer. The Bentofix is needle punched across its entire surface to bind the two geotextiles together. The needle punching laterally confines the bentonite to control swelling during hydration and also provides a high degree of internal shear strength. As a result, the Bentofix GCL's internal friction within the composite is relatively high at approximately 34° so that in the hydrated state, no potential plane of weakness exists within the composite.

.6 Instrumentation

An instrumentation system is required to adequately monitor the tailings dam performance both during and after mine operation. The proposed instrumentation plan should include thermistors, pneumatic piezometers and settlement pins in the area of the main tailings dam and seepage collection dam. Drawing B-5314-009 presents the proposed instrumentation details.

A series of six thermistor strings were installed during the recent field program at various depths up to 15 m in the foundation areas of the main dam and seepage recovery dam. The thermistors were supplied by R.S. Technical Instruments of Port

Coquitlam, British Columbia and were accurate to $\pm 0.2°C$. The thermistor cables are presently protected with steel casings at the ground surface at each test hole location. During construction of the dam, each cable should be extended to the toe of dam by splicing on additional cable and burying the cable horizontally in the embankment fill. An additional thermistor string with six thermistors spaced at 14 m intervals should be placed horizontally within the embankment fill approximately 5 m above the foundation base, as shown on Drawing B-5314-009. All thermistor strings should be labelled and extended to a suitable readout location near the downstream toe. Monitoring of the thermistors should be continued throughout construction as well as the subsequent mine operation period to measure temperature changes and possible thawing in the foundation soils. Monitoring of this data is extremely important as the inclusion of any future design measures is dependant on the presence or absence of permafrost in the foundation.

Pneumatic piezometers are also recommended at several locations as shown on Drawing B-5314-009. In total, fifteen pneumatic piezometers are proposed underlying and within the dam embankment. The pneumatic piezometers should consist of P-100 General Purpose piezometers as supplied by R.S. Technical Instruments. Eleven of these pneumatic piezometers are proposed along a central line down the centre of the creek valley. The pneumatic piezometers should be installed at depths ranging from 3 m to 10 m below foundation level and up to 2 m above foundation level. An additional two piezometers should be installed on each abutment with each set consisting of one piezometer 3 m below foundation level and the other piezometer 1 m above foundation level. Pneumatic piezometer tube lines should be buried in the embankment fill and extended laterally to the toe of dam. Monitoring of the pneumatic piezometers is only recommended in zones which are known to have thawed. Measurement of pneumatic piezometers in frozen zones is not required as these instruments will not reliably measure pore pressures until the surrounding soils thaw. Once thawing has occurred, the pneumatic piezometers will, however, allow measurement of the foundation pore pressures which are caused due to thaw-consolidation and will also allow measurement of any seepage pressures present in the embankment. Monitoring of the pneumatic piezometers therefore need only be initiated after pond filling has commenced and the temperature regime indicates thawing is occurring.

Settlement pins are also required along the crest to monitor settlements after completion of the embankment construction. The settlement pins should consist of 25 mm diameter iron bars at least 1.5 m long which are driven into the embankment fill along the downstream edge of the dam crest. The settlement pins should be positioned across the crest as shown on Drawing B-5314-009. The frequency of survey measurements for the settlement pins will depend on the performance history of the dam and should be reviewed as settlement data is collected.

.7 Borrow Assessment

Approximately 100 000 m^3 of Zone 1 sand fill is required for construction of the tailings dam embankment. The borrow areas which have been identified for construction use in the dam embankment include three sand ridges on the south valley slope upstream of the dam (Borrow Areas A, B and C, see Drawing B-53147-003) and the operating spillway excavation (Borrow Area D) on the north valley slope located downstream of the dam. It is estimated that these borrow areas should be sufficient to provide the necessary quantities for dam construction. The primary borrow material from each of these areas is a fine to medium grained silty sand. The sand is generally frozen below the depth of active zone and thawing of any frozen materials will be required prior to compaction.

The silt content for the sand borrow material typically varies from 10% to 40%. Two standard Proctor density tests were carried out in accordance with ASTM D698 which showed maximum dry densities varying from 1766 kg/m^3 to 1850 kg/m^3. The optimum moisture content to achieve the required maximum dry density varied from 13.3% to 12.8% respectively for each of these two tests. (See test results on Figure A2-5 in Appendix II.)

Borrow Area A contained a surficial layer of fine sand to a depth of 6.25 m which is expected to be suitable for embankment construction. The sand is at or below optimum moisture content in the upper 3 m and then increases above optimum moisture content up to 20% below the 3 m depth. The lower strata in this area, below 6.25 m depth, as observed in DH95-17, consist of silt with sandy zones. Moisture contents in this strata are generally much higher at 30% and use of this zone is most likely not feasible due to the additional drying and conditioning of the wet material which is anticipated prior to compaction.

Borrow Area B is located further upstream on another sand ridge. Similar to Area A, test holes DH95-18 to DH95-20 generally showed a trend towards increasing moisture content with depth. For the most part moisture contents were generally within the 15% to 20% range in the fine sands. Silt layers were observed at various depths which may not be useable in the main dam embankment. It is expected the thickness of useable material may vary from about 9 m on the outer edges of this ridge tapering to less than 2 m towards the south valley slope.

Borrow Area C indicated the presence of significant silt layers in test holes DH95-21 to DH95-23. It is anticipated that the use of these silt soils will not be practical due to high moisture contents which were evident after thawing samples in the field. An upper layer of suitable sand borrow was, however, observed to a depth of 2.4 m near the edge of the deposit.

In addition to the borrow areas upstream of the dam, it is anticipated that a significant quantity of borrow will be obtained from the operating spillway excavation on the north valley slope in the vicinity of test hole DH95-26. It is expected that this excavation will be widened on either side to include a portion of the sand ridge which parallels Dome Creek. Test hole DH95-27 was drilled east of the spillway excavation on this sand ridge. Both of these holes in this area indicate good potential for sand borrow to depths up to 9 m and possibly deeper. Similar to other areas, there is a trend towards increasing moisture content with depth. Again, moisture contents were observed to be less than 20%, except in zones near the ground surface where moisture contents are less than 10%.

Operation of the borrow areas will require that the sand borrow be excavated in a manner to allow thawing prior to placement and compaction. In some cases, it is anticipated that ripping will be required to accelerate the thawing process. Operation of several borrow areas simultaneously will also maximize the amount of thawed material suitable for placement. Excavation should also be carried out in a manner which will mix dry soils with wet soils. This may best be carried out by sloping excavations in order to mix soils from different layers. The deeper sands are otherwise expected to be wet of optimum and drying will be required prior to compaction. The use of a dozer or grader is recommended to blade materials across the embankment fill or in situ in borrow areas to allow sufficient aeration and drying.

3. HYDROLOGY AND WATER MANAGEMENT

.1 Meteorological Stations

The long term meteorological station located closest to the site is Carmacks which has available about 22 years of data. The station is located about 60 km to the east of the site at an elevation of 524 m, or about 600 m to 800 m below the minesite elevation. Other long term stations reviewed as part of this study are listed in Table 1 along with their location with respect to the site.

Table 1 - Meteorological Stations

Station Name	Station Number	Location	Record Interval	Years of Record
Carmacks	2100300	60 km east	1963-1991	29
Whitehorse	2101300	180 km southeast	1942-1990	49
Pelly Ranch	2100880	80 km north	1951-1990	40
Mayo Airport	2100700	185 km north-northeast	1925-1990	66
Haines Junction	2100630	145 km south	1945-1984	40
Burwash Airport	2100182	135 km southwest	1966-1991	26
Aishihik	2100100	80 km southeast	1943-1966	24
Braeburn	2100167	90 km southeast	1974-1991	18

.2 Annual Precipitation

Precipitation at Mt. Nansen occurs primarily as snow from November to the end of April. On average approximately 40% of the annual precipitation falls as snow which corresponds to the typical snowpack of 1 m as observed at the site. Some rainfall occurs as early as April, but the majority of rainfall occurs from June to August.

An analysis of regional weather stations indicates that the average annual precipitation is consistent from about Burwash Landing, northeast to Mayo and as far south as Whitehorse and north to Dawson. Although most of these stations are located at elevations substantially lower that the minesite, increases in precipitation with elevation are not expected at this site given the local rolling topography and the overwhelming effect of the St. Elias range in removing moisture from air masses moving to the northeast. Precipitation does however, begin to significantly increase further northeast of Mayo.

For the reasons mentioned above, Carmacks monthly precipitation data was taken as representative of the site. Mean annual precipitation values on a monthly basis are summarized in Table 2 along with a 200-year return period wet year and a 10-year return period dry year.

.3 Lake Evaporation

The closest station for which lake evaporation data is available is Pelly Ranch (formerly Fort Selkirk) which is located about 80 km north of the minesite at an elevation of 454 m. The lake evaporation data from this station was taken as representative of the site.

There are 17 to 28 years of evaporation data available at Pelly Ranch, depending on the month (May and September have missing data). The data is shown in Table 2. During the summer months of May through August, evaporation exceeds average precipitation indicating a soil moisture deficit. In the wettest month (July) lake evaporation is about twice the precipitation.

Table 2 - Summary of Monthly Precipitation and Evaporation

MONTH	PRECIPITATION			EVAPORATION
	Average Year (mm/month)	10-Year Return Period Dry Year	200-Year Return Period Wet Year	Average Monthly (mm/month)

		(mm/month)	(mm/month)	
January	17.5	13.0	26.5	0.0
February	12.0	8.9	18.2	0.0
March	6.8	5.0	10.3	0.0
April	6.6	4.9	10.0	0.0
May	19.6	14.5	29.8	105.0
June	33.7	25.0	51.2	121.3
July	53.8	39.9	81.7	111.2
August	38.5	28.5	58.5	80.0
September	30.0	22.2	45.6	36.6
October	18.7	13.9	28.4	0.0
November	17.9	13.2	27.2	0.0
December	14.9	11.0	22.6	0.0
TOTAL	270	200	410	454

.4 Short Duration Storms

Weather data has been collected at Carmacks station since 1964. For the years of 1964 to 1984 rainfall data is available in IDF format from Atmospheric Environmental Services which gives annual extreme rainfall for durations from 5 minutes to 24 hours. The IDF format is based on a moving window average over the duration of the rainfall in question. From 1984 to present this level of detailed rainfall information is not available and only once daily measurements are available to determine annual "daily" extreme rainfall. The daily data is available from 1964 to present and thus represents a significantly longer period of record. To account for differences in sampling techniques between the IDF and daily data methods, the daily data is typically scaled up by a sampling factor of 1.13.

The longer term extreme daily data from Carmacks was compared to other stations in the region. The results showed that the Carmacks extreme daily data was low and thus a scaling factor of 1.4 was applied to represent site conditions. Since the longer term daily data is more reliable, the shorter duration IDF data was scaled to match the 24-

hour IDF to the daily data. A frequency analysis was done using this data to determine 20 year and 200 year data for durations of 5 min to 24 hour data as shown in Table 3.

The 24-hour Probable Maximum Precipitation (PMP) was determined from the Carmacks daily data using the Hershfield method. This value was scaled by 1.4 to obtain the 24-hour PMP for the site. Individual Hershfield analyses were carried out for the IDF data for each duration and these were scaled up to match the IDF 24 hour PMP to long term daily 24 hour PMP. The PMP values are shown in Table 3.

For design of the diversion ditch and emergency spillway and to provide estimates of required dam freeboard over the spillway invert, a short duration storm for the 20 year and 200 year return period and the PMP event was required. The rainfall runoff model OTTHYMO was used to determine peak flow in the diversion ditch and the inflow hydrograph for pond routing. Based on the estimated time of concentration of the catchment, a storm of 6 hours was selected. Six hour storms were assembled for each event with an interval of 5 minutes using rainfall data as shown in Table 3 incorporating an alternating block method.

Table 3 - Short Duration Rainfall Data

DURATION	RAINFALL (mm)		
	20 year	200 year	PMP
5 min	8.8	12.9	22.1
10 min	12.0	17.8	41.2
15 min	14.3	21.2	48.1
30 min	16.8	24.1	57.8
1 hr	18.8	26.9	86.1
2 hr	21.7	30.0	101
6 hr	26.0	35.5	124
12 hr	31.9	44.0	146
24 hr	35.7	49.2	170

.5 Diversion Channel and Emergency Spillway Design

.1 General

For operating conditions the recommended level of design is a 20 year return period for the diversion channel and 200 year return period for the emergency spillway. By designing the diversion channel for only the 20 year event, the required channel size and erosion protection are reduced. BYG have indicated that in order to minimize storage requirements for excess water in the tailings pond, water treatment will be performed if water quality standards are not achieved. Therefore, any flows larger than the 20 year return period will be routed through the tailings pond which lowers downstream channel requirements. The general layout and section details of the diversion channel and spillways are shown on Drawings B-5314-003 and B-5314-010.

.2 Diversion Channel

To divert the flow of Dome Creek around the tailings pond, a diversion channel could be constructed on the north abutment of the dam, as shown on Drawing B-5314-003.

Water from the diversion would enter the emergency spillway channel downstream of the dam centreline. If required during mine operation, it would be possible to construct an outlet from the channel into the tailings pond to allow water into the pond if a water deficit were to occur.

A catchment area of approximately 2.3 km^2 contributes flow to the diversion channel. The 20 year peak flow in the diversion channel was calculated to be 6.4 m^3/s. The channel depth is shown in Table 4 for 0.1% and 6% gradients. The channel was assumed to have a 3 m wide invert and 2H:1V sideslopes.

The material underlying the diversion channel is a silty sand which will require protection against erosion. The flat gradient section of the diversion upstream of the dam centreline will require a 300 mm thick layer of Zone 2 gravel consisting of 75 mm minus pitrun gravel, while the steeper section will require a heavier Zone 3 riprap up to 300 mm diameter with a 300 mm thick bedding layer of Zone 2 gravel. Recommended gradations for riprap and bedding materials are provided in Section 9.4 and are summarized on Table 5. The heavier riprap should be extended 10 m upstream of the grade change to prevent scour in the transition from tranquil to supercritical flow. As an alternative, an appropriate geotextile filter cloth (Terrafix 600R or equivalent) could be substituted for the bedding material underlying the riprap.

Due to the permeability of the underlying material, water is expected to seep out of the channel. The amount of seepage is estimated to be in the order of 4500 m^3/month during the summer when the ground is not frozen. An impermeable liner, which would reduce the seepage to almost zero, may be cost effective when considering water treatment costs. The liner could be installed at a later date if pond inflow is deemed too large based on first year operating data.

In the spring, when snow-melt begins and rainstorms may occur, excessive ice, snow and snow drifts must be cleared from the channels to allow for the safe diversion of surface runoff. Every spring and fall, the channel must be checked for vegetation build-up and subsidence due to frost action. Either of these may reduce the effectiveness of the drainage channels and therefore must be monitored.

The present design shows a temporary diversion channel which bypasses the emergency spillway channel. It is anticipated that dam construction will be carried out coincidentally with the emergency spillway excavation which will provide a significant amount of borrow for the dam. A temporary diversion channel is therefore required during the period of construction. Riprap placement and activation of the emergency spillway channel may be delayed till the end of dam construction as the reservoir is initially empty and excess flood flows could be stored in the reservoir if necessary until riprap could be placed. The temporary diversion channel is also subject to erosion during major flows. Construction should therefore be scheduled so as to minimize the operation period for this channel.

.3 Emergency Spillway

The purpose of the emergency spillway is to prevent overtopping of the dam crest during peak inflow. The design level of flood protection is equivalent to the 200 year inflow to which freeboard has been added. To determine peak outflow, the 200 year inflow was routed through the pond to account for storage effects. The reservoir volume elevation curve used in the assessment is shown on Drawing B-5314-006 and the stage discharge curve was determined using a backwater computer model assuming critical flow occurs at the spillway transition from a 0.1% gradient to a 2% gradient.

The calculated emergency spillway flow is 3.7 m^3/s assuming the diversion channel is breached during the event and a starting water level at the spillway invert is 1149.7 m. The flow and channel depths for the emergency spillway are shown in Table 4 for various gradients. Downstream of the intersection with the diversion channel the design flow is 6.4 m^3/s which is equivalent to the diversion channel spillway 20 year peak flow. The channel was assumed to have a 5 m wide invert and 2H:1V sideslopes.

Table 4 - Diversion ditch and Emergency Spillway Hydraulics

Channel Description	Return Period (yrs)	Flow (m^3/s)	Slope (%)	Water Velocity (m/s)	Flow Depth (m)	Channel Depth (m)
North Diversion (3m width)	20 20	6.4 6.4	0.1-0.2 6	0.9 3.2	1.4 0.5	1.4 0.5
Emergency Spillway (5 m width) u/s of intersection with diversion	200 200	3.7 3.7	0.1-0.2 3	0.9 2.3	0.8 0.5	1.1 0.8
Emergency Spillway (5 m width) d/s of intersection with diversion	200 200	6.4 6.4	6 10	2.7 3.0	0.5 0.4	0.8 0.7

Note: Emergency Spillway depth includes 0.3 m freeboard.

The material underlying the emergency spillway is also a silty sand which will require protection against erosion. The flat gradient section of 0% to 0.2% will require a 300 mm thick layer of Zone 2 gravel, while the steeper sections of 2% to 10% will require either Zone 3 or Zone 4 riprap with a bedding layer of Zone 2 gravel or a geotextile filter fabric as shown on Table 5 and Drawing B-5314-010. The heavier riprap should be extended 10 m upstream of the grade change, to prevent scour in the transition from tranquil to supercritical flow. An appropriate geotextile filter cloth (Terrafix 600R or equivalent) could be substituted for the bedding material.

Table 5 - Recommended Diversion Channel and Emergency Spillway Riprap and Filter Sizes

Channel Description	Return Period (yrs)	Slope (%)	Riprap Zone	Riprap				Filter Size
				d15 (mm)	d50 (mm)	d100 (mm)	Thickness (mm)	
North Diversion (3 m width)	20 20	0.1-0.2 6	Zone 2 Zone 3	* 130	* 200	* 300	300 450	no filter 300 mm Zone 2 Gravel or Geotextile**
Emergency Spillway (5 m width) u/s of intersection with diversion	200 200	0.1-1.02 2-3	Zone 2 Zone 3	* 130	* 200	* 300	300 450	no filter 300 mm Zone 2 Gravel or Geotextile**
Emergency Spillway (5 m width) d/s of intersection with diversion	200	6-10	Zone 4	200	300	500	750	300 mm Zone 2 Gravel or Geotextile**

* Denotes (75 mm minus)
** Geotextile fabric should consist of Terrafix 600R or equivalent

The routing of the 200 year inflow produced a maximum pond level of 1150.5 m. Adding a freeboard of 0.4 m to this value results in a dam crest elevation of 1150.9 m. The effective protection of this dam crest level with zero freeboard is 50% of the PMF which is reasonable given the size of the dam and the downstream and financial consequences of failure.

.4 Closure Spillway

For mine closure, a permanent spillway should be constructed in rock on the south abutment. For longterm abandonment, the design flood discharge is assumed to be 100% of the Probable Maximum Flood (PMF). Assuming a spillway invert width of 5 m, channel sideslopes in rock of 0.5H:1V, an invert elevation of 1149.2 m (final tailings level) and PMF conditions, a routed maximum pond level of 1151.3 m was calculated. Including freeboard for waves and wind setup a dam crest of up to 1151.8 m may be required. This exceeds the current planned dam crest elevation of 1150.9 m by 0.9 m indicating that the dam would require raising at closure. Depending on the final level of the tailings it may be acceptable to reduce the invert of the spillway thereby reducing the requirement to raise the dam crest at closure. Alternatively a 10 m wide spillway could be constructed which results in a PMF pond level of 1150.8 m requiring a final dam crest of 1151.3 m, reducing the amount of the dam raise at closure to 0.4 m. The final design of the closure spillway should be determined when the final orientation of the tailings and the settled dam height is known. The analysis should include provisions for wave protection and overtopping.

.6 Water Balance

A combined water and tailings balance was carried out to determine the final tailings level and the amount of excess water that would require treatment. Because the tailings impoundment clearly operates with an excess of water for the average year and given that the diversion channel could be breached if more water were required, a dry year analysis was not done.

1149.7 m based on storing a minimum two-month mill water circulation volume of 30 000 m^3/month. During the winter there is a net loss of pond water due to minimal inflow and losses to tailings voids. In an average year, winter losses are estimated to be 2000 m^3/month. The results for the average year monthly water balance are shown in Appendix VI.

.3 200 Year Water Balance

If a 200 year wet year is added in year 2, with the remaining years being average, results indicate that, with the diversion in place, there will be 298 360 m^3 of excess water over the 3 year mine life. The maximum water level of 1149.7 m is unchanged since it is based on the tailings level plus 2 months volume of mill water circulation. The results for the 200 year monthly water balance are also shown in Appendix VI.

SECTION III

Ref: Screening Report Section 6.1

Also Ref: Water Licence Application Section 6

OPERATIONAL MONITORING PLAN

(addendum to Water Licence Application Section 6.)

6.11 Seepage from Pit Floor Through Brown McDade Adit

Seepage water from the old Brown McDade adit will be monitored and tested on a monthly basis. The seepage will be tested for pH, Arsenic and flow rate.

6.12 Thermistors

Thermistors will be installed with sufficient cable to allow the thermistor string to terminate at the downstream toe of the dam. The thermistor strings will be monitored on a monthly basis.

6.13 Piezometers

The piezometers will be installed with sufficient tubing to allow for extension to the downstream toe of the dam. The piezometers will be monitored when the thermistor strings show that the ground around the piezometer has thawed. The piezometers will then be monitored on a monthly basis.

6.14 Settlement Pins

Settlement pins will be driven into the downstream crest of the tailings dam. The settlement pins will protrude 50 mm from the dam crest. The settlement pins will be surveyed on a monthly basis .

6.15 Thermometer

Daily maximumand minimum temperatures will be recorded at the mine site. Comparison of this data with the Carmacks station will aid in long term prediction of temperature variances.

6.16 Rainfall Gauge

Precipitation will be recorded on a daily basis fopr comparison with Carmacks station and to determine runoff inflows.

Snowpack

SECTION IV

Ref: Screening Report Section 6.1
Screening Report Section 6.4

Also Ref: Tailings Impoundment
Final Design Report Section 5

WASTE ROCK ARD AND WASTE ROCK MANAGEMENT PLAN

4) Variability within each rock type grouping will be assessed using frequency and cumulative distribution plots of Maximum Potential Acidity (MPA) or Percent Sulphide (%$S^{=}$), Neutralization Potential (NP), Net Neutralization Potential (Net NP) and Neutralization Potential Ratio (NPR).

5.9.3.3 Waste Management Plan

The waste management plan will ensure that all handling of waste rock is supervised by a qualified geologist or mining engineer. The plan will dictate the following;

1) All individual rock types will be identified during mining based on a combination of location, visual characteristics, Au assay, ABA and other suitable analyses (depending on location).

2) Detailed records will be maintained for all materials distributed during mining and left on surface. The information will be logged according to quantities, waste characteristic and disposal locations.

3) The waste rock disposal logs will be reviewed by the mine manager on a regular basis to ensure that waste rock with a potential for acid generation are handled so as to eliminate or minimize risk associated with ARD from the waste dump.

5.9.3.4 Schedule

The Waste Rock ARD program will commence with development of the pit prior to the commencing of milling. The samples will be logged on a regular basis as development of the pit proceeds. The primary focus of this program will be to ensure that the construction materials do not have any potential for ARD. Most of the samples and ABA data will be collected during the first year of operation based on the current mine plan.

5.9.3.5 Report Submission

A report will be prepared to present the data 60 days after the first phase of pit development is complete and milling has commenced. This report will assess the potential for ARD occurring in each rock type grouping and compare the data with the waste rock characterization work conducted as part of the IEE.

5.9.3.6 Decision Criteria

The criteria used to assess the potential for ARD from waste rock will be based on the characteristics and quantities of each rock type identified in the above report. The Waste Rock ARD program will verify the assessment completed as part of the IEE.

The decision criteria with therefore consist of drawing comparisons with the IEE predictions. If the waste rock exhibits substantially higher potential for ARD, it may be necessary to implement sub-aqueous disposal either in a flooded pit. Alteratively materials with a low potential for ARD, may require blending and backfilling with non-acid generating waste rock.

95% percentile

Write here